Durability Assessment of Large Surfaces Using Standard Reliability Methods

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Abstract: It is well recognized that initiation limit states defined in (ISO 13823, 2008) may be of uttermost importance for serviceability as well as ultimate limit states of civil engineering structures. However, practical applications of durability assessments may be difficult as basic variables influencing structural durability are often random quantities with a considerable spatial variability that should be considered as random fields. Application of common discretisation techniques may be rather cumbersome and require a considerable amount of input data. A simplified probabilistic model for spatial variation is thus proposed to allow for durability analysis of large surfaces using efficient reliability methods such as FORM/SORM. The technique is applied in the example of carbonation of concrete where spatial variation of the carbonation depth and concrete cover is considered. It appears that the failure probability increases with the size of surface exposed to unfavourable environmental influences. Optimisation study further indicates that the total costs primarily depend on the thickness of concrete cover, design service life, and the surface area exposed to the deterioration. However, the optimum concrete cover and optimum reliability index seem to be almost independent of the size of the surface area.

Keywords: durability, random fields, discretization, FORM.

1. Introduction

Durability is becoming an important issue of structural design. General principles on the probabilistic approach to verification of structural durability are provided in the new international standard (ISO 13823, 2008). The document is based on the fundamental principles provided in (ISO 2394, 1998), (ISO 19338, 2003) and (EN 1990, 2002). Materials of other international organisations such as CEB, fib, RILEM and findings in scientific publications have also been taken into account.

(Holicky, 2011) indicates that due to limited experience with the operational use of (ISO 13823, 2008), additional studies focused primarily on models of material deterioration, acceptance criteria, and theoretical models of basic variables are required. Difficulties in practical applications may arise particularly when basic variables influencing structural durability have a considerable spatial variability (e.g. for large surfaces concentrations of unfavourable agents or diffusion properties of construction materials). In probabilistic analyses the spatial variability is normally described by random fields. Application of common discretisation techniques, see e.g. (Allaix et al., 2009), may be rather cumbersome and may require a considerable amount of input data.

In the present study a simplified probabilistic model for spatial variation is thus proposed to allow for durability analysis of large surfaces using efficient reliability methods such as FORM/SORM. The

5th International Conference on Reliable Engineering Computing (REC 2012)
Edited by M. Vořechovský, V. Sadílek, S. Seitl, V. Veselý, R. L. Muhanna and R. L. Mullen

ISBN 978-80-214-4507-9. Published by Ing. Vladislav Pokorný − LITERA

technique is applied in the example of carbonation of concrete where spatial variation of the carbonation depth and concrete cover is considered.

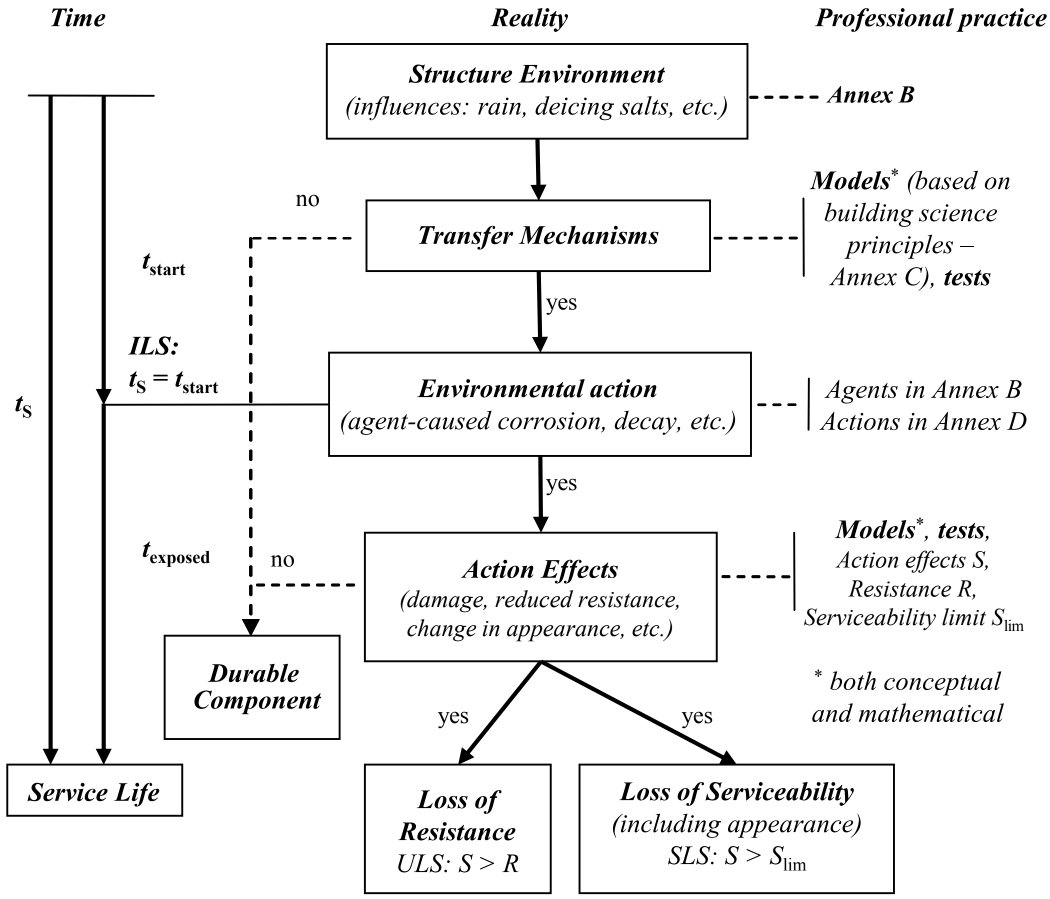

Figure 1. Limit state method for durability (accepted from (ISO 13823, 2008)).

2. Concept of Limit States

(ISO 13823, 2008) formulates the principles of limit state methods for durability. The key steps of deterioration modelling and reliability verification using the concepts of limit states are indicated in Figure 1. It provides a very general scheme that may be modified considering actual conditions of an investigated structure. It should be noted that Figure 1 is a result of many discussions and amendments made during the development of (ISO 13823, 2008).

The three vertical strands in Figure 1 indicate a time axis (on the left), reality (in the middle) and professional practice (on the right). The time axis is split into two parts by the point denoted as the Initiation Limit State (ILS). It corresponds to the point in time when environmental actions have turning point (for example the beginning of reinforcement corrosion or decays of construction materials).

The environmental effects may in general be combined with the action effects (the middle part of Figure 1). Resulting effects may then lead to the loss of resistance (bearing capacity) or to the loss of serviceability (excessive cracking or deformations). These limit states - ULS and SLS - are indicated in the lower part of Figure 1. However, an important question of load combination rules is not covered in (ISO 13823, 2008).

3. Verification of the Service Life

The fundamental durability requirement is represented by a simple condition that the predicted service life t_{SP} should be greater than the design service life t_D with a sufficient degree of reliability. Difficulties are obviously linked to the term „sufficient reliability". It is well recognised that the service life t_S is dependent on a number of basic variables and is consequently a random variable having a considerable scatter. The document (ISO 13823, 2008) thus provides a probabilistic formulation of this criterion:

$$\mathrm{P}[t_S < t_D] < P_{\mathrm{target}} \tag{1}$$

where P_{target} denotes the target probability of the service life t_S being less than the design service life t_D. Commonly the design service life t_D is a deterministic quantity (for example 50 or 100 years) specified in advance.

4. Verification of the Limit States

The probabilistic formulation of the limit state conditions is similar to a case of the service life. For an arbitrary point in time $t \le t_D$ the following condition should be satisfied:

$$P_f(t) = \mathrm{P}[\mathrm{Z}(t) < 0] = \mathrm{P}[R(t) - S(t) < 0] < P_{\mathrm{target}} \tag{2}$$

where $P_f(\cdot)$ denotes the failure probability; $\mathrm{Z}(\cdot)$ = reliability margin; $R(\cdot)$ = resistance; and $S(\cdot)$ = action effect. The basic probabilistic condition for the serviceability can be written analogously as:

$$P_f(t) = \mathrm{P}[\mathrm{Z}(t) < 0] = \mathrm{P}[S_{\mathrm{lim}} - S(t) < 0] < P_{\mathrm{target}} \tag{3}$$

where S_{lim} is the limit value of the serviceability indicator (for example of the crack width or deflection). The initiation limit state may be verified in accordance with Eqs. 2 or 3 depending on particular conditions.

5. Assessment of the Service Life

The probabilistic assessment of the predicted service life t_{SP} is schematically shown in Figure 2 adopted from (ISO 13823, 2008). Figure 2 describes monotonously varying action effects $S(t)$ and resistances $R(t)$. The horizontal axis denotes the time t and the vertical axis in the upper part denotes the resistance $R(t)$ and action effect $S(t)$, in the lower part the probability $P_f(t)$. Probability distributions of the variables $R(t)$ and $S(t)$ are indicated by probability density functions.

Obviously the failure probability $P_f(t)$ is an increasing function of time t. The predicted service life t_{SP} follows from the relationship:

$$P_f(t_{SP}) = P_{target} \tag{4}$$

However, there are no recommendations concerning the target probability P_{target} provided in (ISO 13823, 2008) and this open question may cause difficulties in the effective use of the document.

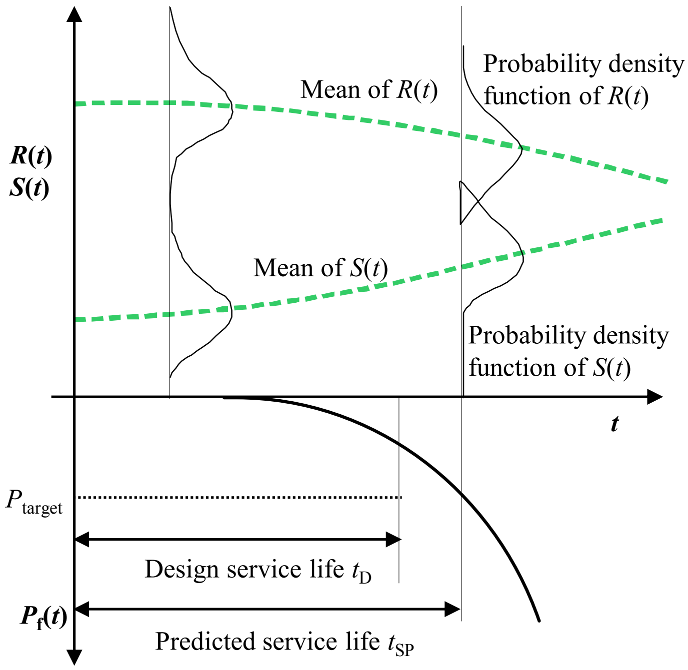

Figure 2. Probabilistic assessment of the service life.

6. Target Reliability Level

Target reliability level, indicated by the target probability P_{target} or reliability index β_{target}, depends in general on the definition of the service life time, whether the critical durability requirement concerns the ultimate limit state, serviceability limit state or initiation limit state and what are the consequences of their infringement (Holicky, 2011). Table I provides indicative intervals for the target reliability.

Table I. Indicative values of the target probability and reliability index.

Limit state	P_{target}	β_{target}
Ultimate limit state - ULS	~ 10^{-4}	~ 3.7
Serviceability limit state - SLS	0.01 to 0.1	1.3 to 2.3
Initiation limit state - ILS	0.05 to 0.2	0.8 to 1.6

The target probability P_{target} and reliability index β_{target} given in Table I represent indicative values only. They are based on the target values recommended in (ISO 2394, 1998) and (EN 1990, 2002). It should be mentioned that (ISO 2394, 1998) indicates an additional dependence of the target values on relative costs of safety measures (required to increase the reliability level). This aspect should be also considered when specifying target reliability level for durability requirements. Specification of the appropriate reliability level remains, therefore, one of the most important open questions.

7. Simplified Model for Spatial Variability

(Faber and Rostam, 2001) suggested that a large surface exposed to deterioration effects should be analysed as an assembly of elementary surfaces rather than a whole structure. Probabilistic characteristics of the variables influencing the deterioration should then include also the spatial variability of the variables among elementary surfaces. For instance several studies focused on reinforced concrete structures, see e.g. (Vu and Stewart, 2005) or (Stewart and Mullard, 2007), reveal that the elementary surface may be a square with the side length varying from 1 to 3 meters. For steel structures the size of an elementary surface may correspond to a size of inspected areas (e.g. 3 m), (Straub, 2004).

The present study is based on the following assumptions:

- The basic (random) variables influencing a given limit state can be divided into random fields $\mathbf{W}(x,y)$ (e.g. some material properties) and variables attaining a single value for the whole structure $\mathbf{X}(t)$ (e.g. some environmental influences).
- Random fields $\mathbf{W}(x,y)$ are homogeneous and can be approximated by N elementary surfaces of the same size. Values of the random fields in each elementary surface, $W_{i,j}$ for $i = 1..n_W$ (number of the random fields) and $j = 1..N$ (number of elementary surfaces), are independent, identically distributed variables (having an appropriate probability distribution based on available data). To simplify the notation, vector of the values of the random fields in an elementary surface j is hereafter denoted as $\mathbf{W} = W_i$ for $i = 1..n_W$.
- Some of the variables $\mathbf{X}(t)$ can be time-dependent. Then they are either monotonously decreasing (when favourably influencing durability) or monotonously increasing (when unfavourable). Consequently the failure probability is monotonously increasing with time.

This simplified model for spatial variation is assumed to yield conservative results compared to standard techniques such as discretization at the centre of gravity or discretization by spatial mean proposed by (VanMarcke, 1983). The failure probability at the elementary surface p_{f} can be obtained from the following relationship:

$$p_{\text{f}}[\mathbf{W},\mathbf{X}(t)] = \text{P}\{\text{Z}[\mathbf{W},\mathbf{X}(t)] < 0\} \tag{5}$$

The failure probability at a whole surface can be written as:

$$P_{\text{f}}(t) = \text{P}\{n_{\text{deg}}[\mathbf{W},\mathbf{X}(t)] \,/\, N \geq \alpha_{\text{lim}}\} = \text{E}_{\mathbf{X}(t)}\{\text{P}[n_{\text{deg}}(\mathbf{W}|\mathbf{x}(t)) \,/\, N \geq \alpha_{\text{lim}}]\} \tag{6}$$

where $n_{\mathrm{deg}}(\cdot)$ denotes the number of elementary surfaces for which $[Z(\cdot) < 0]$; α_{lim} the limiting value of the deterioration level $\alpha = n_{\mathrm{deg}} / N$; $E(\cdot)$ expectation operator; and $\mathbf{x}(t)$ values of the variables $\mathbf{X}(t)$.

Given $\mathbf{x}(t)$, values of the reliability margin in each elementary surface are statistically independent. The probability of occurrence of ν "failed" elementary surfaces out of N is thus given by the binomial distribution, see (Faber and Rostam, 2001), (Malioka et al., 2011) and (Sýkora and Holický, 2011):

$$\mathrm{P}\{n_{\mathrm{deg}}(\mathbf{W}|\mathbf{x}(t)) = \nu\} = \mathrm{f}_{\mathrm{binom}}\{\nu, N, p_{\mathrm{f}}[\mathbf{W}|\mathbf{x}(t)]\} \tag{7}$$

where $\mathrm{f}_{\mathrm{binom}}(\cdot)$ is the probability density function of a binomial distribution. Note that the number $n_{\mathrm{deg}} = \nu$ actually represents the probability of ν successes out of N independent trials with the probability of success p_{f}. The failure probability (6) can then be modified as:

$$P_{\mathrm{f}}(t) = \mathrm{E}_{\mathbf{X}(t)}\{1 - \mathrm{F}_{\mathrm{binom}}[N\alpha_{\mathrm{lim}}, N, p_{\mathrm{f}}(\mathbf{W}|\mathbf{x}(t))]\} \tag{8}$$

where $\mathrm{F}_{\mathrm{binom}}(\cdot)$ is the cumulative distribution function of the binomial distribution.

The use of $\mathrm{F}_{\mathrm{binom}}$ significantly decreases computational demands since the assessment of spatial variability simply reduces to evaluation of the cumulative distribution function of the binomial distribution. Note that the binomial distribution may be approximated by a normal distribution for, say, $N > 50$. The expectation in Eq. 8 can be carried out by the FORM/SORM methods, see e.g. (Wen, 1990).

The limiting value α_{lim} should be specified by an owner, preferably using cost optimisation and previous experience. As an example (Fitch et al., 1995) suggested $\alpha_{\mathrm{lim}} = 0.12$ for corrosion-induced cracking of reinforced concrete bridges while $\alpha_{\mathrm{lim}} = 0.2$ was considered in a general study by (Faber and Rostam, 2001).

It is emphasized that the proposed model of spatial variability may be oversimplified when the random fields need to be associated with different areas for which their values can be considered as independent. In such a case it would be necessary to modify Eqs. 5, 7 and 8. However, it is foreseen that the proposed approximation can be applied in a number of practical cases.

8. Numerical Example

8.1. Deterioration Model

The initiation limit state can be well illustrated by the carbonation of concrete. The limit state may be defined as a simple requirement that the carbonation depth $S(t)$ (action effect) is less than the concrete cover R (resistance). Note that it may be more suitable to define the failure considering an indicator that can be verified by visually (such as crack width).

A large, vertical concrete surface is investigated. Concrete cover R and inverse carbonation resistance under natural carbonation conditions $R_{\mathrm{NAC},0}^{-1}$ are assumed to be spatially variable. The size of an element is assumed to be 0.5 m in accordance with (Vu and Stewart, 2002) and (Malioka, 2009). The variables $\mathbf{X}$ and the model uncertainty of action effect K_S are time-independent. Notation and probabilistic models of the basic variables are given in Table II.

Given values $\mathbf{x}$ and k_S, the failure probability at an elementary surface is determined as follows:

$$p_{\mathrm{f}}(\mu_R, t|k_S, \mathbf{x}) = \mathrm{P}[R(\mu_R) - k_S\, S(R_{\mathrm{NAC},0}^{-1}, t|k_S, \mathbf{x}) < 0] \tag{9}$$

where μ_R denotes the mean of the concrete cover (nominal value – study parameter).

The point-in-space carbonation depth is described in accordance with (fib, 2006):

$$S\left(R_{\mathrm{NAC},0}^{-1}, t \middle| rh_{\mathrm{real}}, c_{\mathrm{s}}, b_{\mathrm{w}}\right) = \sqrt{t}\sqrt{2\left(\frac{1-rh_{\mathrm{real}}{}^{5}}{1-rh_{\mathrm{ref}}{}^{5}}\right)^{2.5}\left(\frac{t_{\mathrm{c}}}{7}\right)^{b_{\mathrm{c}}} R_{\mathrm{NAC},0}^{-1} c_{\mathrm{s}}}\left(\frac{0.0767}{t}\right)^{\frac{(p_{\mathrm{SR}} tow)^{b_{\mathrm{w}}}}{2}} \tag{10}$$

where t is time in years. Note that in Eqs. 9 and 10, the values of the random fields R and $R_{\mathrm{NAC},0}{}^{-1}$ are denoted by capital letters while values of the random variables and deterministic quantities are denoted by small letters.

Table II. Probabilistic models of the basic variables.

Type	Variable / random field	Symbol	Distribution	Unit	μ_X	V_X	Ref.
Random fields	Concrete cover	R	Beta (lower bound = 0, upper bound ≈ 3μ_R)	mm	μ_R	0.35	(Holický and Holická, 2006) (fib, 2006)
	Inverse carbon. resistance under natural carbonation	$R_{\mathrm{NAC},0}{}^{-1}$	Gamma	[(mm^2/year) / (kg/m^3)]	2×10^4	0.5	
Random variables	Relative humidity	RH_{real}	Beta	-	0.71	0.18	nearest weath. station
	CO_2 concentration	C_{s}	normal	kg/m^3	8.2×10^{-4}	0.12	(fib, 2006)
	Regression coefficient	B_{w}	normal	-	0.45	0.37	(fib, 2006)
	Model uncertainty	K_S	LN	-	1	0.1	-
Determ. variables	Refer. relative humidity	rh_{ref}	-	-	0.65	-	(fib, 2006)
	Curing period	t_{c}	-	day	5	-	-
	Regression coefficient	b_{c}	-	-	-0.57	-	(fib, 2006)
	Prob. driving rain	p_{SR}	-	-	0.4	-	nearest weath.
	Time of wetness	tow	-	-	0.27	-	station

The model for relative humidity is based on daily mean values. Probability of driving rain for vertical surface (facing to the west here) is determined from the distribution of wind directions during rain events. Time of wetness is assessed from the average number of days per year for which daily precipitation total exceeds 2.5 mm (in this case 100 days per year). Contrary to the recommendations of (fib, 2006), the regression coefficient b_{c} is considered here to be deterministic since numerical experience indicates that its variability is negligible.

The considered model for the carbonation depth has been calibrated against extensive measurements on cooling towers (unprotected external concrete) described by (Holický and Holická, 2006). Basically, the presented model leads to a similar mean value and somewhat lower coefficient of variation and skewness as compared to the measurements.

8.2. Reliability analysis

The limiting deterioration level $\alpha_{\mathrm{lim}} = 0.15$ is considered. The failure probability $P_{\mathrm{f}}(t)$ is obtained from Eq. 8 by integration over K_S and **X**. In Figure 3 the failure probability $P_{\mathrm{f}}(t)$ is indicated for $\mu_R = 25$ mm and $N = 1$, 20, and 100. In addition results based on a hypothetical assumption according to which correlation amongst the elementary surfaces is neglected and all the random variables are spatially variable, are plotted for the number of elementary surfaces $N = 100$.

It appears that the failure probability depends significantly on the number of elementary surfaces. The assumed model predicts significantly lower failure probabilities for $N = 1$ than for $N = 10$ or 100. Similarly as concluded by (Stewart, 2004), it follows that the spatial variability should be appropriately considered particularly when analysing large surfaces.

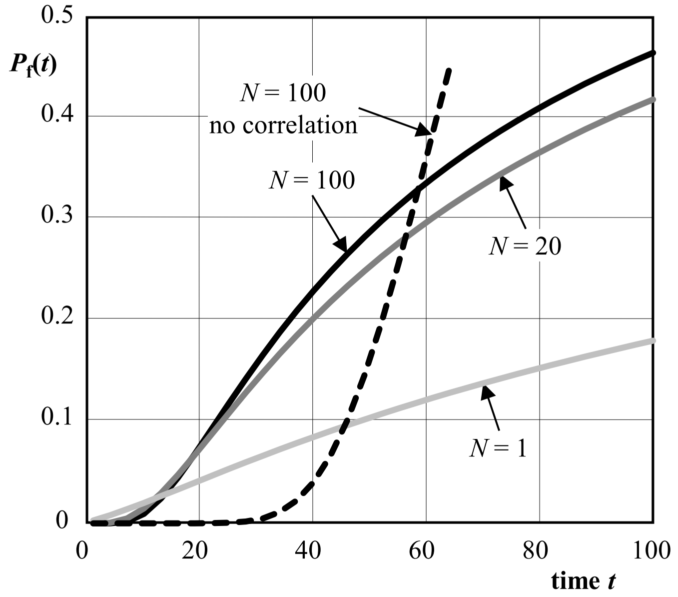

Figure 3. Variation of failure probability $P_{\mathrm{f}}(t)$ with time t for $\mu_R = 25$ mm and $\alpha_{\mathrm{lim}} = 0.15$.

Further Figure 3 shows that misleading results may be obtained when the correlation amongst the elementary surfaces is neglected. In this case, the failure probability is very low for $t < 30$ years and then significantly increases.

Figure 3 can be used to assess the service life t_{SP} defined by Eq. 4 for a specified target probability P_{target}, the mean of concrete cover μ_R and number of elementary surfaces N. If for example $P_{\mathrm{target}} = 0.15$, then the mean $\mu_R = 25$ mm corresponds to $t_{\mathrm{SP}} \approx 30$ years for $N = 20$ and 100, but for $N = 1$, $t_{\mathrm{SP}} \approx 80$ years is estimated. Obviously, the service life t_{SP} appears to be significantly dependent on the number of elementary surfaces and on the target probability.

9. Probabilistic Optimisation

Methods of probabilistic analysis may be effectively used for the specification of the target reliability level and durability assessment, (Holicky, 2009) and (Holicky, 2011). The total costs of execution and repair of the structure due to failure (infringement of the initiation limit state) can be expressed as a function of the mean μ_R (decisive parameter):

$$C_{\text{tot}} = C_0 + C_1\,\mu_R + \text{E}[C_{\text{f}}] \tag{11}$$

where C_0 denotes the initial costs independent of μ_R; C_1 the cost of a unit of μ_R; and $\text{E}[C_{\text{f}}]$ = expected expenses related to the durability failure given by:

$$\text{E}[C_{\text{f}}] = C_{\text{f}} \int_t \pi_{\text{f}}(\tau)\,\text{d}\tau \tag{12}$$

where C_{f} denotes a present value of the expected expenses related to the durability failure; q annual discount rate (around 0.03); and $\pi_{\text{f}}(\cdot)$ the discounted conditional failure rate given by:

$$\pi_{\text{f}}(t) = P_{\text{f}}(t)' \,/\, \{[1 - P_{\text{f}}(t)](1 + p)^t\} \tag{13}$$

where $P_{\text{f}}(t)'$ is the time derivative of the failure probability given in Eq. 8. Standardised total cost is considered as:

$$\kappa_{\text{tot}} = [C_{\text{tot}} - C_0] \,/\, C_1 = \mu_R + C_{\text{f}} \,/\, C_1 \int_t \pi_{\text{f}}(\tau)\,\text{d}\tau \tag{14}$$

The optimum mean $\mu_{R,\text{opt}}$ may be determined from:

$$\partial\kappa_{\text{tot}} \,/\, \partial\mu_R = 0 \tag{15}$$

Note that within the realistic domain of μ_R from 20 to 70 mm, Eq. 15 may not have a practical solution and the minimum of the total costs may not be attained.

Considering the above described initiation limit state, the standardised total costs κ_{tot} given by Eq. 14 are shown in Figure 4 assuming the design life time t = 40 years (typical for cooling towers), q = 0.03 and N = 100. In addition variation of the failure probability P_{f} with μ_R is also indicated. It appears that the optimum mean $\mu_{R,\text{opt}}$ considerably increases with increasing cost ratio C_{f} / C_1. More specifically, it follows that:

- For $C_{\text{f}} / C_1 = 10$ ("small" failure consequences or "high" unit costs), the optimum $\mu_{R,\text{opt}}$ is not attained in the practical range of μ_R.
- For $C_{\text{f}} / C_1 = 100$ ("medium" failure consequences and "medium" unit costs), the optimum mean is $\mu_{R,\text{opt}} \approx 29$ mm ($\beta_{\text{opt}}(\mu_{R,\text{opt}} = 29$ mm$) \approx 1.1$).
- For $C_{\text{f}} / C_1 = 1\,000$ ("high" failure consequences or "small" unit costs), then the optimum mean increases up to $\mu_{R,\text{opt}} \approx 43.5$ mm ($\beta_{\text{opt}}(\mu_{R,\text{opt}} = 43.5$ mm$) \approx 2.4$).

Additional parameter study reveals that the optimum mean concrete cover $\mu_{R,\mathrm{opt}}$ is nearly independent of the number of elementary surfaces N.

Variation of the optimum (target) reliability index β_{opt} (based on $\mu_{R,\mathrm{opt}}$) with the number of elementary surfaces N is shown in Figure 5 for the design life time $t = 40$ years, $q = 0.03$ and $C_f / C_1 = 100$ and 1 000. It follows that β_{opt} insignificantly increases with increasing N. In the first approximation the values $\beta_{\mathrm{opt}} \approx 1.1$ ($C_f / C_1 = 100$) and $\beta_{\mathrm{opt}} \approx 2.4$ ($C_f / C_1 = 1\ 000$) may be considered.

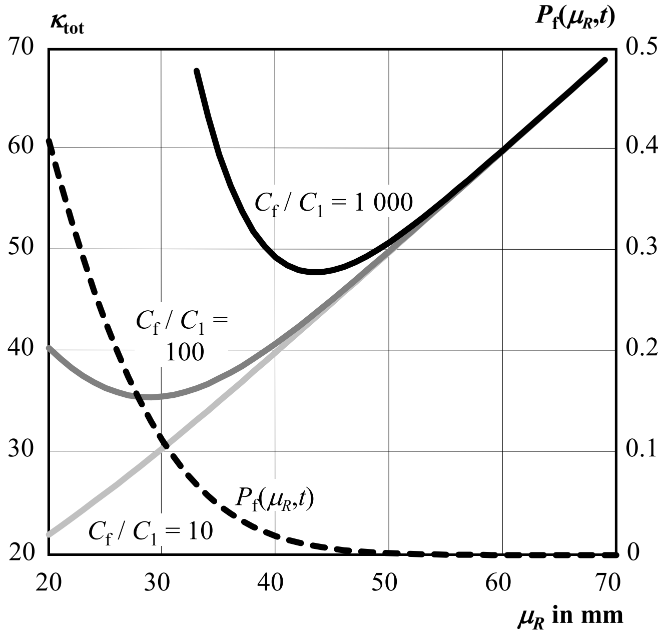

Figure 4. Variation of the total standardised costs κ_{tot} and failure probability P_f with the mean concrete cover μ_R for $q = 0.03$, $t = 50$ years and $N = 100$.

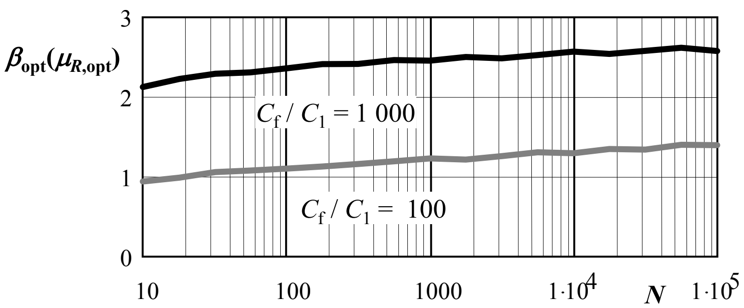

Figure 5. Variation of the optimum reliability index β_{opt} with the number of elementary surfaces N for $q = 0.03$, $t = 40$ years, and $C_f / C_1 = 100$ and 1 000.

10. Concluding Remarks

Structural durability is becoming an important part of structural design of buildings and other civil engineering works. It may be significantly affected the spatial variability particularly for large surfaces. Simplified model for deterioration of large surfaces, proposed here, seems to require less input data and significantly lower computational demands compared to random field techniques. It is foreseen that this model can be effectively used for optimisation studies when structural durability need to be assessed for various decision parameters.

Numerical example, focused on the carbonation of concrete, reveals that the failure probability increases with the size of surface exposed to unfavourable environmental influences. Optimisation study indicates that the total costs particularly depend on the thickness of the concrete cover, design service life, and the size of a surface area exposed to the deterioration. However, the optimum concrete cover and optimum reliability index seem to be almost independent of the size of the surface area. As a first approximation the optimum concrete cover of 30 mm and optimum reliability index of 1.1 may be recommended for the required design life of 40 years, discount rate 0.03 and the cost ratio $C_f / C_1 = 100$.

Further experimental data and appropriate models for the carbonation process, related model uncertainties and initial and failure costs are needed. Further research should be focused on the comparison of standard random field approaches with the proposed simplified model.

Acknowledgements

The study has been conducted in the framework of the research projects P105/12/G059 and P105/12/0589 supported by the Czech Science Foundation.

References

Allaix, D.L., Carbone, V.I. and Mancini, G. Recent advances in the modelling of space-variant structural properties. In H. Furuta, D.M. Frangopol & M. Shinozuka, editors. *Proc. ICOSSAR 2009,* Osaka, 13-17 September 2009. CRC Press/Balkema, Leiden, 2009.

EN 1990. *Eurocode - Basis of structural design.* CEN, Brussels, 2002.

Faber, M.H. and Rostam, S. Durability and Service life of Concrete Structures – the Owners Perspective. Anonymous *Proc. Int. IABSE Conf. on Safety, Risk, and Reliability - Trends in Engineering,* Malta, 21-23 March 2001. IABSE, 2001.

fib. *Model Code for Service Life Design.* fib, Lausanne, 2006.

Fitch, M.G., Weyers, R.E. and Johnson, S.D. Determination of end of functional service life for concrete bridge decks. *Transp. Res. Rec.,* (1490):60-66, 1995.

Holický, M. and Holická, N. Probabilistic optimisation of concrete cover exposed to carbonation. In K. Kovler, editor. *Proc. ConcreteLife'06,* Tallin, Estonia, RILEM Publications SARL, 2006.

Holicky, M. Forensic study of structures for durability. *Safety and Security Engineering III,* 1083-12, 2009.

Holicky, M. Forensic engineering of structures with durability problems. *Proceedings of the Institution of Civil Engineers-Structures and Buildings,* 164(1):19-25, 2011.

ISO 13823. *General principles on the design of structures for durability.* ISO, Geneve, Switzerland, 2008.

ISO 19338. *Performance and assessment requirements for design on structural concrete.* ISO, Geneve, Switzerland, 2003.

ISO 2394. *General principles on reliability for structures.* ISO, Geneve, Switzerland, 1998.

Malioka, V. *Condition indicators for the assessment of local and spatial deterioration of concrete structures (PhD thesis).* ETH, Zurich, 2009.

Malioka, V., Matsch, S. and Faber, M.H. Acceptance criteria for the spatial variability of the concrete cover depth in a Swiss tunnel. In M.H. Faber, J. Köhler & K. Nishijima, editors. *Proc. ICASP11,* ETH Zurich, 1-4 August 2011. CRC Press/Balkema, Leiden, 2011.

Stewart, M.G. Spatial variability of pitting corrosion and its influence on structural fragility and reliability of RC beams in flexure. *Structural Safety,* 26(4):453-470, 2004.

Stewart, M.G. and Mullard, J.A. Spatial time-dependent reliability analysis of corrosion damage and the timing of first repair for RC structures. *Engineering Structures,* 29(7):1457-1464, 2007.

Straub, D. *Generic approaches to risk based inspection planning for steel structures.* ETH, Zurich, 2004.

Sýkora, M. and Holický, M. Deterioration Model for Large Reinforced Concrete Structures. In C. Berenguer, A. Grall & C. Guedes Soares, editors. *Proc. ESREL 2011,* Troyes, France, 18 - 22 September 2011. CRC Press/Balkema, Leiden, 2011.

VanMarcke, E. *Random Fields: Analysis and Synthesis.* MIT Press, Cambridge MA, 1983.

Vu, K.A.T. and Stewart, M.G. Spatial Variability of Structural Deterioration and Service Life Prediction of Reinforced Concrete Bridges. Anonymous *Proc. 1st International Conference on Bridge Maintenance, Safety and Management,* Barcelona, 14-17 July 2002. IABMAS, 2002.

Vu, K.A.T. and Stewart, M.G. Predicting the Likelihood and Extent of Reinforced Concrete Corrosion-Induced Cracking. *Journal of Structural Engineering,* 131(11):1681-1689, 2005.

Wen, Y.K. *Structural load modeling and combination for performance and safety evaluation.* Elsevier, Amsterdam, 1990.