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The Assessment of the Reliability of Potentially Deteriorated Reinforced Concrete Elements with Bayesian Networks

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ABSTRACT: The concrete deterioration caused by corrosion is a complex physical, chemical and mechanical process. The modeling of this process is subjected to significant uncertainties, which are based on a simplistic model representation of the actual physical process and limited information on material, environmental and loading characteristics.The present work proposes a generic framework for the estimation of structural reliability of a potentially corroded reinforced concrete element. This framework combines structural reliability analysis and Bayesian networks. Thereby uncertainties by model parameters, but also additional information, provided by measurements, monitoring and inspection results, are considered.

1. INTRODUCTION

One of the major deterioration mechanisms in reinforced concrete (RC) structures is corrosion of the reinforcing steel. This process causes effects such as cracking, spalling, or delamination of the concrete and also leads to a reduction in the reinforcement cross-section and a loss of bond strength (Bertolini et al., 2004). These changes are accompanied by a decrease of the load bearing capacity and structural reliability of the corresponding structural elements. Therefore RC structures may be inspected and maintained in order to control the decrease in structural reliability. However, it is an ongoing challenge to direct these activities within the large portfolios of RC structures cost effectively.

A vast amount of research did result in a resonable understanding of the phenomena that cause RC deteriotation and in corresponding probabilistic representation of the initiation of corrosion damage. The representation of corrosion progression and how a system of partly corroded reinforcement-bars affect the loadbearing capacity, the structural reliabilty and herewith the principal performance criterion of a RC element represents still an interesting research challenge.

In this paper a generic framework for stochastic modeling of the structural reliability of deteriorating RC elements is presented. Thereby a combination of structural reliability analysis (SRA) and Bayesian networks (BNs) is used to model the process from environmental exposure towards time-dependent component reliability.

2. DEGRADATION AND RELIABILITY MODELS

For the degradation of concrete structures, several models have been developed to provide methods to estimate the duration of time during which RC structures maintain a desired level of functionality. Service life models such as DuraCrete (2000), LIFECON (2003), or fib Bulletin 34 (2006) provide valuable information about the durability characteristics of concrete structures.

The basic approach of such a service life model is based on Tuutti (1982), where the service life is subdivided to two phases, initiation and propagation. While the models for the initiation phase are well documented, there is a lack of information for the propagation phase. Additionally, models for both phases are developed separately, such that connections from the initiation phase to the propagation phase cannot be made in terms of a unified model. This is unsatisfactory in scope of a holistic view of the service life and the identification of optimal decisions during the management of deteriorated RC structures.

In the remainder of this paper the initiation and propagation models proposed by DuraCrete (2000) and the model for effects of corrosion suggested by Val and Melchers (1997) and modified by Stewart (2004, 2009, 2012), are going to be utilized.

2.1. *Chlorde induced corrosion*

A frequent cause of reinforcement steel corrosion is contamination by chloride. To initiate the corrosion process, the chloride content at the surface of the reinforcement has to reach a certain threshold value (Bertolini et al., 2004). The transport of chloride in concrete towards the reinforcement can be represented in a simplified way by use of Fick's second law of diffusion :

$$\frac{\partial C_{\mathrm{cl}}}{\partial t} = D_{\mathrm{cl}} \frac{\partial^2 C_{\mathrm{cl}}}{\partial x_{\mathrm{cl}}^2} \tag{1}$$

where C_{cl} is the concentration of chloride ions at distance x_{cl} from the concrete surface after time t of exposure to chlorides and D_{cl} the chloride diffusion coefficient. The obtained solution of the partial differential equation is:

$$C_{\mathrm{cl}}(x_{\mathrm{cl}}, t) = C_{s,\mathrm{cl}}^{(e_e)} \left(1 - \mathrm{erf}\left(\frac{x_{\mathrm{cl}}}{2\sqrt{D_{\mathrm{cl}} \cdot t}}\right)\right) \tag{2}$$

where $C_{s,\mathrm{cl}}$ is the surface concentration of chlorides and $\mathrm{erf}(\cdot)$ denotes the error function. According to DuraCrete (2000) D_{cl} can be calculated as:

$$D_{\mathrm{cl}} = k_{e,\mathrm{cl}}^{(e_e)} \cdot k_{t,\mathrm{cl}} \cdot k_{c,\mathrm{cl}}^{(t_{\mathrm{cur}})} \cdot D_o^{(w/c)} \cdot \left(\frac{t_{o,\mathrm{cl}}}{t}\right)^{n_{\mathrm{cl}}^{(e_e)}} \tag{3}$$

where $k_{e,\mathrm{cl}}$ is the environmental parameter, $k_{t,\mathrm{cl}}$ is a test method parameter, $k_{c,\mathrm{cl}}$ is the executions parameter, D_o is the empirical diffusion coefficient, t_o is the reference time and n_{cl} the age factor; depending on the exposure events: the exposure environment e_e, the curing time t_{cur} and the water-cement ratio w/c. Each parameter can be expressed as a random variable, the corresponding distributions and parameters that are used in this study are documented in the Appendix.

For the onset of corrosion, the limit state can be assumed as the probability that the critical chloride concentration C_{crit} is reached at the depth of the reinforcement denoted by d_c.

$$p_{f,\mathrm{cl}} = \pi[C_{\mathrm{crit}} - C_{\mathrm{cl}}(x_{\mathrm{cl}} = d_c, t) \leq 0] \tag{4}$$

2.2. *Propagation of corrosion*

After the depassivation of the reinforcement has occurred and the passive layer broke down, the so-called propagation phase starts and the reinforcement steel starts to corrode.

The corrosion rate V_{corr} is usually expressed as the penetration rate and is measured in [mm/yr].

A simplified propagation model based on Nilsson and Gehlen (1998) is used in the DuraCrete (2000) model. Hereby, the basic assumption is to represent the corrosion rate as a product of material parameters and local influencing factors:

$$V_{\mathrm{corr}} = \frac{m_o}{\rho} \cdot F_{Cl}^{(P_{\mathrm{cl}})} \cdot F_{\mathrm{Galv}} \cdot F_{O_2} \tag{5}$$

where m_o is a constant for corrosion rate versus resistivity based on Faraday's law, ρ is the concrete resistivity, F_{Cl} is the chloride corrosion rate factor considering the chloride induced corrosion P_{cl}, F_{Galv} the galvanic effect factor, and F_{O_2} the oxygen availability factor.

In the model of Nilsson and Gehlen (1998), the resistivity ρ is a function of the concrete properties, the temperature and the moisture conditions in the concrete cover (DuraCrete, 2000):

$$\rho = \rho_o \left(\frac{t_{\mathrm{Hydr}}}{t_{o,\mathrm{r}}}\right)^{n_{\mathrm{r}}} k_{t,\mathrm{r}} \cdot k_{c,\mathrm{r}} \cdot k_{T,\mathrm{r}}^{(T)} \cdot k_{RH,\mathrm{r}}^{(RH)} \cdot k_{Cl,\mathrm{r}}^{(p_{f,\mathrm{cl}})} \tag{6}$$

where ρ_o is a potential concrete resistivity for a reference environment, t_{Hydr} the time of hydration, t_o a reference time, n_{r} the age factor, $k_{t,\mathrm{r}}$ a test method parameter, $k_{c,\mathrm{r}}$ the execution parameter, $k_{T,\mathrm{r}}$ the temperature factor depending on the temperature T, $k_{RH,\mathrm{r}}$ the humidity factor depending on the relative humidity RH and $k_{Cl,\mathrm{r}}$ the chloride factor depending if chloride induced corrosion P_{cl} occur; the corresponding distributions together with the parameters can be found in the Appendix.

2.3. *Effects of corrosion*

If corrosion is initiated, the consequences are a reduction in the cross section of the load carrying reinforcement steel, increase in bar diameter resulting from the volumetric expansion of the corrosion products, and a change in the mechanical properties of the reinforcement steel and the concrete. These effects do not only lead to serviceability issues of corresponding structural components, but may also affect its structural reliability and therefore the safety of the component. Correspondingly the corrosion affects the reinforcement itself and the surrounding concrete. In the remainder of this paper only the effects of corrosion on the reinforcement steel will be considered.

Local or pitting corrosion is only associated with chloride induced corrosion. According to Gonzalez et al. (1995) the corrosion rate for pitting corrosion is four to eight times higher than the average penetration p_{av} on the surface of a reinforcement bar. This ratio between maximum and average corrosion penetration is called pitting factor and denoted by $R_{\text{pit}} = p_{\max}/p_{\text{av}}$. Stewart (2004) proposes, that the pitting factor R_{pit} can be treated as a random variable modeled by a Gumbel distribution.

For simplicity, a hemispherical form of pits is assumed. The radius of the pit, $p_{\max}$, at time t_{corr}, can be estimated as: $p_{\max}(t_{\text{corr}}) = V_{\text{corr}} \cdot t_{\text{corr}} \cdot R_{\text{pit}}$. The pit configuration is used to predict the cross sectional area of the pit, denoted by A_{pit}.

2.4. Structural Reliability Analysis

Structural reliability analysis (SRA) is generally used to estimate the probability of adverse events related with the performance of structures that is here defined with the event $g(\mathbf{Z}) \leq 0$. Here, $\mathbf{Z}$ describe a set of random variables $Z_1, \ldots Z_n$ which influence the performance of a structure.

The probability of failure p_f is equal to the probability that an undesired performance will occur and is expressed thought an integral of the form:

$$p_f = \pi(g(\mathbf{Z}) \leq 0) = \int \cdots \int_{g(\mathbf{Z}) \leq 0} f_{\mathbf{Z}}(\mathbf{z}) d\mathbf{z} \qquad (7)$$

where $f_{\mathbf{Z}}(\mathbf{z})$ describe the complete joint probability density function, for the set of random variables $\mathbf{Z}$.

3. BAYESIAN NETWORK APPROACH

3.1. Transformation

In order to provide a generic framework for stochastic modeling of the structural reliability of deteriorating RC elements, each physical model, explained in Section 2 is transformed in a BN. Every parameter of the model is represented as a variable or node in the single model (SM) network. Also the exposure events are represented as nodes. The edges denote the certain causal relationships between different variables corresponding to the physical models.

Based on the Eqs. (2) to (4), the BN for chloride induced corrosion can be represented as shown in Figure 1.

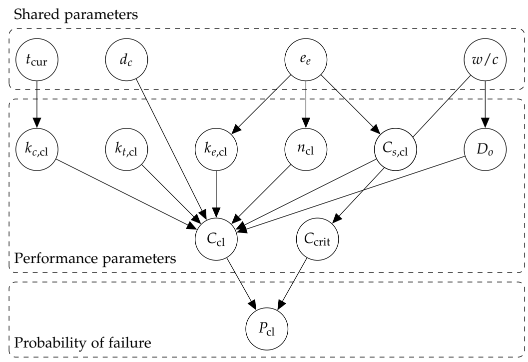

Figure 1: Bayesian network for corrosion.

A coupled model (CM) over the whole physical process is constructed. Therefore, the SMs have to be connected; shared and dependent random variables from the different BNs must be singled out and replaced with the corresponding nodes or BNs. In certain circumstances some additional nodes must be added to the CM.

- Shared parameters are denoted parameters that occur in multiple places across one or more networks. Where the notation parameter describes a node in which the values are not determined by others.
- Dependent parameters are similar to shared parameters for a SM, but with the difference that the parameter itself depends on an other BN. In a CM these parameters are part of the coupled BN and link the different models to an overall consistent model representation, starting by the edification of the RC structure and ending by reaching a critical limit state.
- In some cases it is beneficial to add extra nodes to the existing BNs, to extract some extra information from the network and on the other hand to simplify the existent model.

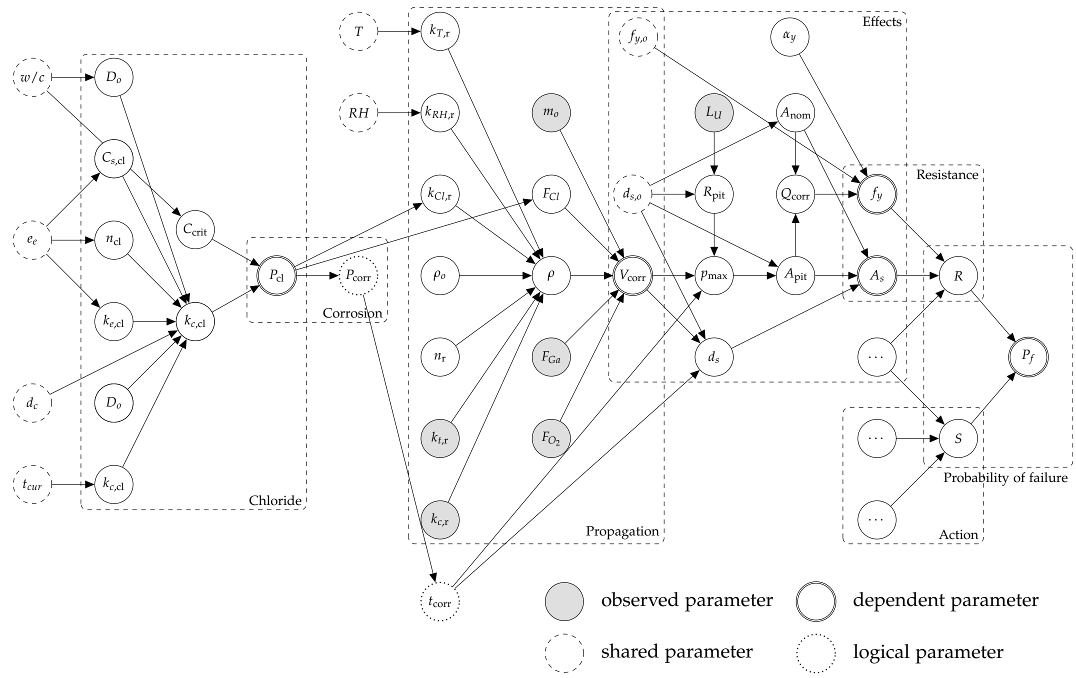

Figure 2: Coupled model for degradation of concrete, caused by corrosion.

Figure 2 shows the CM for the structural reliability of a deteriorating RC element. This model combines the probabilistic models for: chloride induced corrosion, propagation of corrosion, effects of corrosion and structural reliability. Constant variables from the physical model are modeled as uniform distributed nodes and set evidence at the initial value, here illustrated as shaded nodes. Nodes denoted by "…" represent additional nodes that are necessary to compute the structural reliability.

3.2. Simplification

In order to describe the physical process of corrosion, it is not necessary to represent every random variable in the model as explicit as a node in the BN. Instead parts of the BN can be reduced to a single random variable, containing the information of the other ones. For the information transformation from several nodes to one specific node SRA is used.

Consider a Bayesian network $\mathscr{B}$ with the probability measure $\pi_{\mathscr{B}}$ over the outcome space of a set of random variables $\mathbf{X} = \{\mathbf{Y}, \mathbf{Z}, F\}$, where $\mathbf{Y} = \{Y_1, \dots, Y_j\}$ denote a set of shared parameters with the set of children $\mathbf{Z} = \{Z_1, \dots, Z_k\}$. According to Section 2.4, the set of random variables $\mathbf{Z}$ influences the performance of a structure. An undesired performance will occur if $g(\mathbf{Z}) \leq 0$, which is equal to the probability of failure F and it implies an unsafe structure. The joint probability measure for this network is given as:

$$\begin{aligned} \pi_{\mathscr{B}}(\mathbf{X}) &= \pi(Y_1, \dots, Y_j, Z_1, \dots, Z_k, F) \\ &= \prod_{i=1}^{j} \pi(Y_i) \prod_{i=1}^{k} \pi(Z_i|\mathrm{pa}(Z_i))\ \pi(F|\mathrm{pa}(F)) \end{aligned} \tag{8}$$

Where $\pi(F|\mathrm{pa}(F))$ can be expressed as the indicator function $\mathbf{1}_{[g(\mathbf{Z})\leq 0]}$. Hence it follows:

$$\begin{aligned} \pi_{\mathscr{B}}(\mathbf{X}) &= \pi(Y_1, \dots, Y_j, F(\mathbf{Z})) \\ &= \prod_{i=1}^{j} \pi(Y_i) \int\limits_{g(\mathbf{Z})\leq 0} \prod_{i=1}^{k} \pi(Z_i|\mathrm{pa}(Z_i))\, dz_1 \dots dz_k \end{aligned} \tag{9}$$

The integral in Eq. (9) can be expressed as:

$$\pi(F|\mathbf{Y}) = \pi(g(\mathbf{Z}) \leq 0\,|\mathbf{Y}) \tag{10}$$

Consequently, Eq. (8) can be rewritten as:

$$\pi_{\mathscr{B}}(\mathbf{X}) = \prod_{i=1}^{j} \pi(Y_i)\ \pi(g(\mathbf{Z}) \leq 0\,|\mathbf{Y}) \tag{11}$$

Which describes the simplified form of the Bayesian network $\mathscr{B}$, where the second part of Eq. (11) can be solved by using SRA.

In case of the SM for chloride induced corrosion, the random variable of interest is the probability of chloride induced corrosion P_{cl}. All other random variables are not explicitly necessary to describe for the structural reliability computation. So the information of the material, environmental, execution, and test variables are collected and treated in the node for the probability of failure. Only the input parameters have to be modeled beside P_{cl}.

The disadvantage of this simplification is that information of the model values, such as the age factor n_{cl}, are no longer available. The huge advantage is that those values need no longer to be modeled and so a discretization of those continuous random variables is omitted. Beyond this, SRA can be used to calculate the probability of failure, which leads to more accurate values for the node P_{cl} as any discretization of the unsimplified BN. Also the property that the node P_{cl} only can reach two states, allows to transmit those outcome values without any loss of information to an other model (or node).

The same principle as discussed previously can be used to simplify the SM for chloride induced corrosion. However, V_{corr}, A_s and f_y are also only intermediate results for the structural reliability. Using the idea of simplification as before, these values can be included in R, the random variable for the resistance of a RC element or even included in P_f, the probability of failure of the structure. This leads to a two-phase model, where the first phase is the failure of corrosion and the second phase the failure of the system according to Tuutti (1982), but now coupled through a probabilistic model based on a BN.

3.3. *Temporal probabilistic model*

The last step to provide a sophisticated model for the deterioration process of RC structures evolving in time is to "unroll" the previous discussed CM over the service life of the structure.

A dynamic Bayesian network (DBN) is just another way to represent stochastic processes using a DAG. To model domains that evolve over time, the system state represents the system at time t and is an assignment of some set of random variables $\mathscr{V}$. Thereby the random variable X_i itself is instantiated at different points in time t, represented by X_i^t and called template variable. To simplify the problem, the timeline is discretized into a set of time slices with a predetermined time interval δ. This leads to a set of random variables in form of $\mathscr{V}^0, \mathscr{V}^1, \ldots, \mathscr{V}^t, \ldots, \mathscr{V}^{\mathscr{T}}$ with a joint probability distribution $\pi(\mathscr{V}^0, \mathscr{V}^1, \ldots, \mathscr{V}^t, \ldots, \mathscr{V}^{\mathscr{T}})$ over the time $\mathscr{T}$, abbreviated by $\pi(\mathscr{V}^{0:\mathscr{T}})$. This distribution can be reparameterized by using the chain rule for probabilities.

To unroll the CM over the service life of the structure, the properties of DBNs are used. That lead to a so-called dynamic coupled model (DCM). Key of this procedure is to model the DBN efficiently, which includes the simplification and optimization methodologies introduced in Section 3.2, to decrease the size of the network.

After the simplification of the CM and the subdivision of the shared parameters into initial and temporal values, the BN can be expanded over time, which is shown in Figure 3.

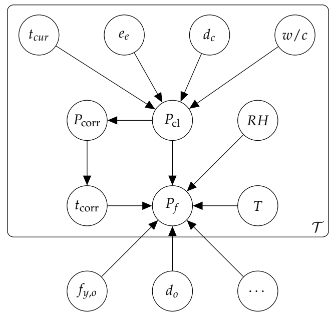

Figure 3: Dynamic Bayesian network for degradation of concrete caused by corrosion, given as plate model.

This proposed DCM framework, based on SRA and BNs, allows to couple the probabilistic models for the initiation and propagation of corrosion. Thereby uncertainties by model parameters, but also additional information, provided by measurements, monitoring and inspection results, can be considered.

4. APPLICATION: BENDING BEAM

4.1. *Structural configuration*

The structural configuration is assumed to be a simply supported RC beam with a rectangular cross-section. Therefore, the ultimate flexural capacity M_u of the RC beam can be used to describe the resistance R of the structure. The resistance R can be expressed in terms of ultimate flexural capacity M_u

for a simple supported RC beam:

$$R = M_u = A_s f_y \left(d - \frac{A_s f_y}{1.6 f_c b} \right) X_m \qquad (12)$$

where A_s is the cross-sectional area of the reinforcement, f_y the yield strength, d the effective depth, f_c the compressive strength of concrete, b the beam width, and X_m describes the model uncertainties.

The nominal resistance in RC design is obtained from the design condition (Eurocode, 2012):

$$\frac{1}{\gamma_m} R_k = \alpha \gamma_G G_k + (1-\alpha) \gamma_Q Q_k \qquad (13)$$

where γ_m, γ_G and γ_Q are the corresponding partial safety factors for the resistance and for the load, and α describes the ratio between permanent loads G_k and variable loads Q_k and is defined in the range from 0 to 1.

The time period is taken as 50 years. According to Eurocode (2012), a target reliability index of $\beta_t = 3.8$ should be reached, assuming no deterioration of the RC structure. This can be ensured by using a suitable set of partial safety factors ($\gamma_m = 1.4, \gamma_G = 1.35, \gamma_Q = 1.5$) and a permanent to variable load ratio of $\alpha = 0.5$.

4.2. *Results*

The following sections, show some results and capabilities of the proposed DCM framework. Due to the complexity and vast amount of data, only few simplified results will be presented here.

Therefore, it is assumed, that no evidence is given for the considered system, meaning all exposure parameters treated as unknown and equally likely. Hence, the quantitative values are indicative and not necessarily realistic.

The proposed framework allows an analysis for the whole process of corrosion, i.e. the initiation process, the propagation process and the mechanical performance of a RC structure that depend on each other. This leads to the result that the qualitative service life model proposed by Tuutti (1982) actually can be represented as quantitative model.

Figure 4 shows a schematic of the typical service life modelling approach based on Tuutti (1982), represented by the solid line. The DCM framework is shown by the dashed line. Here, the end of service life depends on the defined limit state; for example, the reliability index β falling below an acceptable reliability index β_{acc}.

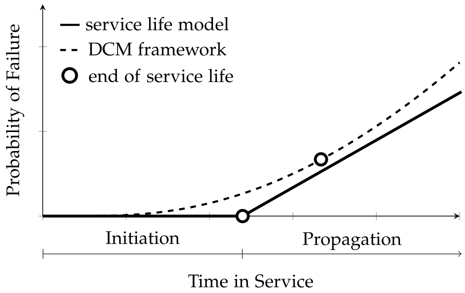

Figure 4: Comparison of service life models.

In the DCM framework, a conventional initial phase is no longer outlined, which describes the period during the depassivation of reinforcement, because of the fact that even in the first weeks the probability of corrosion onset is considered. This assumption has been confirmed under experimental conditions and field conditions.

Instead, the initiation phase in the DCM describes the period of time where no significant loss of structural performance can be expected. This criteria is related to the general requirements concerning the safety of the RC structure, in terms of the probability of failure, as shown in Figure 5. This period of time depends not only on the model for chloride induced corrosion but rather on the whole DCM, which includes also the propagation and the effects of corrosion.

The probabilistic models for the case of corrosion initiation and propagation, caused by chloride penetration, are functions of a number of random variables, discussed in Section 2. The simplified models in Section 3.2 have reduced the amount of random variables to a set of shared parameters.

These parameters can also be called initial condition indicators and are used in the probabilistic model as prior estimations for the probability that the RC structure is in a condition state at some defined period of time during its service life.

To make existing service life models more accurate and realistic, the proposed framework allows, on the one hand, to deal with uncertainties by the input parameter of the model and on the other hand to consider dependency in time.

For example, the bar diameter d_s can be used to provide a prior estimated of the structural perfor-

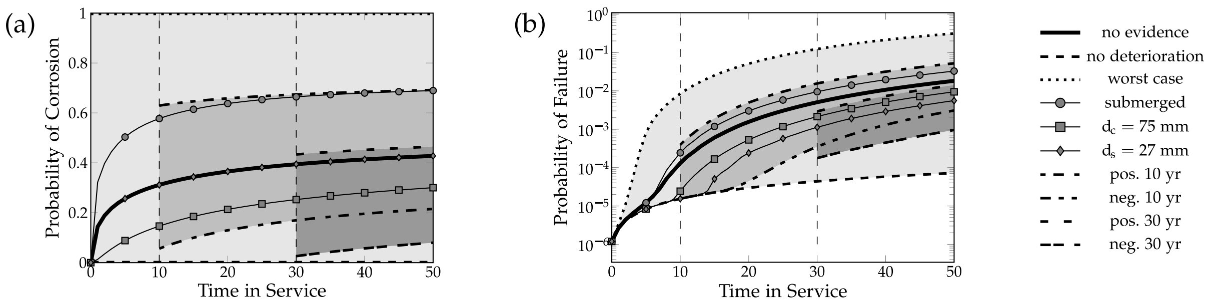

Figure 5: Probability of (a) corrosion and (b) failure over the service life.

mance. However, additional information about the bar diameter affects only the probability of failure, as shown in Figure 5. In contrast, the effects of the concrete cover d_c or the surrounding environment influences the probability of corrosion and probability of failure. The exact estimation of those parameters for an RC structure is not always possible. Hence, using the DCM framework allows to take those uncertainties into account.

During service life several parameters may change and/or the RC structure evolves differently in time as expected. Hence, new information about the condition of the RC structure have to be collected during the service life. One way is to monitor the RC structure and continuously collect data about the condition states. Another option is to do selective inspections of the condition of interest during the service life.

For instance, half-cell potential measurement (HCPM) is a widely recognized and standardized non-destructive method for assessing the corrosion state of the reinforcement in RC structure. The quality of the HCPM is affected by different factors (e.g. moisture, cracks, etc.). Hence, the interpretation of a HCPM should be combined with other measurements and information.

Based on these assumptions, Faber et al. (2006) proposed that whether the probability of corrosion given by HCPM, is observed or not, is stated $\pi(I_{\mathrm{hc}}|p_{f,\mathrm{cl},y}) = 0.9$ and $\pi(I_{\mathrm{hc}}|\bar{p}_{f,\mathrm{cl},y}) = 0.24$, where the inspection is expressed through the probability of an indication of corrosion initiation I_{hc}, given that corrosion occurs $p_{f,\mathrm{cl},y}$, or given that corrosion does not occur $\bar{p}_{f,\mathrm{cl},y}$.

Figure 5 shows the influence of HCPM on a RC structure, performed after 10 and 30 years in service with corresponding positive and negative test indications. If no HCPM is performed the prediction is still based on the prior estimations for the models. After a inspection the test result can be considered in the DCM. Hence, the prediction of the structural safety can be updated.

5. CONCLUSIONS

The present paper proposes a DCM framework, based on SRA and BNs, which enables the coupling of probabilistic models for the initiation and propagation of corrosion. Within this framework uncertainties of model parameters and additional information, provided by measurements, monitoring and inspection results, can be considered.

The basic features of the framework are illustrated along an indicative example. Another possible feature of the proposed framework is the study of the impacts of parameters, models and decisions on the results by sensitivity analysis. Such a study might deliver important insights for the further development of the models in particular and might direct future research needs in the area in general.

Using the presented simplification of the SMs, the critical part of discretizing a continuous random variable for BNs can be reduced to a minimum amount or even eliminated. Hence, the DCM can be expanded over a period of time and updated when new information becomes available.

Keeping it feasible for the beginning, only one element of a RC structure that is exposed to the process of corrosion is considered. However, the deterioration of concrete caused by corrosion is strongly related to spatial and temporal variability. This property can be modeled by different approaches and will be necessary for a holistic contemplation of the system.

6. APPENDIX

Table 1: Probabilistic parameters

Sym.	Description	Type	Mean	CoV	Unit	Ref.
D_o	diffusion coefficient					[1]
	w/c = 0.4	Norm	220.9	0.12	mm^2/yr	
	w/c = 0.45	Norm	315.6	0.1	mm^2/yr	
	w/c = 0.5	Norm	473.4	0.09	mm^2/yr	
$k_{e,\mathrm{cl}}$	environmental parameter					[1]
	Submerged	Gam	1.325	0.17	–	
	Tidal	Gam	0.924	0.17	–	
	Splash	Gam	0.265	0.17	–	
	Atmospheric	Gam	0.676	0.17	–	
$C_{s,\mathrm{cl}}$	surface concentration*					[1]
	Submerged	Norm	10.35	$(0.07, 0.06)$	$\%wb$	
	Tidal	Norm	7.76	$(0.18, 0.14)$	$\%wb$	
	Splash	Norm	7.76	$(0.18, 0.14)$	$\%wb$	
	Atmospheric	Norm	2.57	$(0.14, 0.16)$	$\%wb$	
n_{cl}	age factor					[1]
	Submerged	Beta	0.30	0.17	–	
	Tidal	Beta	0.37	0.19	–	
	Splash	Beta	0.37	0.19	–	
	Atmospheric	Beta	0.65	0.11	–	
$k_{c,\mathrm{cl}}$	execution parameter					[1]
	$t_{\mathrm{cur}} = 1\ d$	Beta	2.4	0.29	–	
	$t_{\mathrm{cur}} = 3\ d$	Beta	1.5		–	
	$t_{\mathrm{cur}} = 7\ d$	Det	1	–	–	
	$t_{\mathrm{cur}} = 28\ d$	Beta	0.8	0.13	–	
$k_{t,\mathrm{cl}}$	test method parameter	Norm	0.832	0.03	–	[1]
$t_{o,\mathrm{cl}}$	reference time	Det	0.077	–	yr	[1]
C_{crit}	critical concentration					[1]
	w/c = 0.3	Norm	0.9	0.17	$\%wb$	
	w/c = 0.4	Norm	0.8	0.13	$\%wb$	
	w/c = 0.5	Norm	0.5	0.2	$\%wb$	
m_o	rate / resistivity	Det	822	–	$\mu m\Omega/yr$	[1]
F_{Cl}	corrosion rate factor					[1]
	corrosion is true	sLN	5.71	1.84	–	
	corrosion is false	Det	1	–	–	
F_{Galv}	galvanic effect factor	Det	1	–	–	[1]
F_{O_2}	oxygen factor	Det	1	–	–	[1]
ρ_o	pot. concrete resistivity	Norm	77	0.16	Ωm	[1]
n_{r}	age factor	Norm	0.23	0.17	–	[1]
$k_{T,\mathrm{r}}$	temperature factor					[1]
	T > 20 ^oC	Norm	0.073	0.21	–	
	T = 20 ^oC	Det	1	–	–	
	T < 20 ^oC	Norm	0.025	0.04	–	
$k_{RH,\mathrm{r}}$	humidity factor	Norm				[1]
	RH = 50 %	sLN	1669	1.79	–	
	RH = 65 %	sLN	15.8	1.63	–	
	RH = 80 %	sLN	5.42	0.82	–	
	RH = 95 %	LN	2.94	0.14	–	
	RH = 100 %	Det	1	–	–	
$k_{Cl,\mathrm{r}}$	chloride factor	Norm	0.72	0.15	–	[1]
$k_{c,\mathrm{r}}$	execution parameter	Det	1	–	–	[1]
$k_{t,\mathrm{r}}$	test method parameter	Det	1	–	–	[1]
$t_{o,\mathrm{r}}$	reference time	Det	1	–	yr	[1]
R_{pit}	pitting factor					[2]
	$d_s = 10\ mm$	Gum	5.67	0.23	–	
	$d_s = 16\ mm$	Gum	6.23	0.24	–	
	$d_s = 27\ mm$	Gum	7.17	0.19	–	
d_s	bar diameter	Norm	$(10, 16, 27)$	0.01	mm	[3]
h	beam height	Norm	550	0.02	mm	
L	beam length	Det	5000	–	mm	
b	beam width	Norm	350	0.03	mm	
f_c	compressive strength	LN	$[25, 45]$	0.17	Mpa	[3]
d_c	concrete cover	LN	$[15, 85]$	0.3	mm	[3]
G_k	permanent loads	Norm	1	0.1	kN/m^2	
Q_k	variable loads	Gum	1	0.4	kN/m^2	
f_y	yield strength	LN	$[450, 600]$	0.05	Mpa	[3]
X_m	model uncertainty	LN	1.1	0.07	–	[3]

[1] DuraCrete (2000), [2] Stewart (2012), [3] JCSS (2008)
* with (CoV_A, CoV_ε), $C_{s,\mathrm{cl}} = A \cdot (w/c) + \varepsilon$, and $\varepsilon \sim N(1, \sigma_\varepsilon)$

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