Update of Service Life Design with Monitoring Data from Corrosion Sensors

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ABSTRACT: This paper gives guidance on how to incorporate additional information from monitoring action into a full-probabilistic service life design procedure with respect to durability. The incorporation of data will be outlined by means of a case study in which data from corrosion sensors (in this case Anode Ladders) installed into parking decks of a major parking garage is being used to update the a-priori service life design with respect to chloride-induced reinforcement corrosion. Chloride-induced corrosion is in most cases the governing deterioration process for parking structures and of high economical relevance as consequences can be costly. During the design stage both the environmental impact and the material resistances are unknown and will thus be incorporated with correspondingly high variation coefficients, leading to a great extent of uncertainty for the a-priori design. Integration of monitoring data during the use of the structure by means of a Bayesian Update will lead to a deeper understanding of the structural condition and gradually to a higher degree of certainty of the service life design and thus enable the owner to schedule the optimal point-of-time for intervention measures.

1 INTRODUCTION

With recent advances in both monitoring techniques and monitoring data acquisition and -processing, monitoring of structures is expected to gain increased importance for different fields of application. One major advantage of monitoring is that the information can be used for an economic long-term planning of maintenance action over service life. Besides using the monitoring data to evaluate the structural behavior within the observation period this data can also be used to improve the accuracy of the prediction of the structure's long-time behavior and therewith enables a life cycle management of concrete structures.

Especially in the field of infrastructure systems this issue becomes more important, as the environmental impact can be severe and the budget resources available for the maintenance of the public bridge stock are generally scarce. Basically, the same applies for private owners of concrete structures, which are responsible for the maintenance over the service life. As the increased need for tools to assist with the management of large building stocks goes along with recent achievements in the field of deterioration modeling, new generations of life cycle management systems employing full-probabilistic deterioration models for the condition prognosis are under development. A further field of application of these software tools is the sector of public-private-partnership projects (PPP). Within PPP projects the construction company is not only responsible for the planning and construction phase, but also for the maintenance of the structure over a period of normally twenty to thirty years.

2 STRUCTURE OUTLINE

The following case study, a major parking garage attached to a football stadium, cp. Fig. 1, is intended to demonstrate the use of corrosion monitoring data as an element of predictive life cycle management. By the time of completion in 2005 it was Europe's largest parking garage with 9.800 parking spaces in total.

Figure 1. View of the parking garage during construction.

In the case of this parking garage, a very slender construction was chosen. The four parking decks of 125 m length and 152 m width each, were constructed as prestressed concrete slabs of only 20 cm height supported on concrete columns without any additional downstand beams.

According to German standards, reinforced or prestressed concrete parking decks have to be carried out with a coating system that prevents chloride ingress into both cracked and uncracked concrete. However, taking the considerable costs for the coating of 270,000 m² parking deck area and the comparably short life time of coating systems into account, alternative corrosion protection concepts were discussed. As the parking garage is only in use on match days, lower chloride loads can be assumed compared to other parking structures. In the end, owner, construction company and local building authorities agreed upon a local coating of cracked concrete areas in combination with a durability concept for the uncracked areas. This concept comprised reliability calculations, installation of corrosion monitoring devices and a tight inspection schedule to detect additional cracking in time. As the parking decks were constructed as continuous slabs, cracking due to bending was only to be expected close to the columns. Therefore, coating was limited to these tension zones, whereas no coating was applied to surface areas under compression, Fig. 2. For the uncracked area it has been agreed upon to use a service life design.

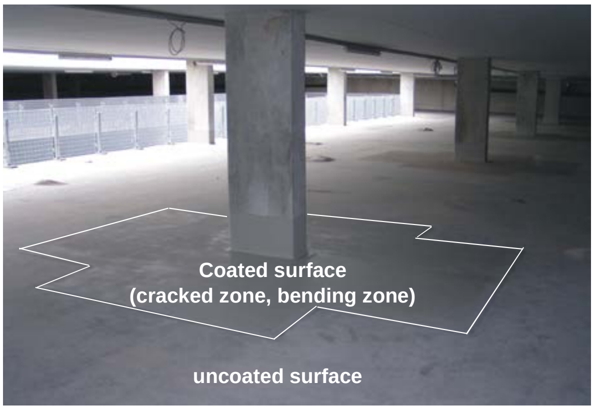


Figure 2. Coating of concrete surfaces solely in the bending zone. Surfaces under compression are left uncoated and visually checked for cracking.

3 SERVICE LIFE DESIGN

3.1 General

In contrast to load design, durability design has traditionally been carried out according to merely empirical “deemed to satisfy” rules defining minimum concrete cover, maximum water-cement-ratios or cement types to be used. With the fib Model Code for Service Life Design (2006) a step is made towards a performance-oriented durability design, which includes full probabilistic deterioration models taking actual environmental loads and material resistances into account. In analogy to the load design the precondition for the application is a model in which the load is confronted with the resistance. This confrontation is carried out in the so-called limit state equation, which can be evaluated full-probabilistically. For durability considerations, a time-dependent formulation of the limit state equation is used, allowing for the monitoring of changes of system reliability with time.

From the probabilistic prognosis using models as mentioned above requirements for specific performance based concrete characteristics can be deducted. According to German standard DIN EN 1992-1-1:2011, the concrete slabs had to be classified as XD3 – surface subject to chloride attack, alternatingly wet and dry. For concrete surfaces which are directly driven upon, the German standards demand the application of a crack-bridging coating system or alternative concepts to avoid depassivation of the reinforcement due to chloride ingress. As mentioned above the owner targeted to limit the coating to cracked concrete areas at the exposed surface. For the uncracked areas the durability was ensured by increasing the material resistance towards chloride ingress in order to avoid the depassivation of the reinforcement over service life. Such probabilistic models with respect to depassivation due to carbonation or chloride ingress have already been applied successfully over the last decade for various structures. In order to verify the modelling results and gradually improve the reliability of the prognosis, additional corrosion sensors were installed in the park decks to monitor the chloride ingress with time. The data attained from these devices can be incorporated into the service life design via Bayesian Update to improve the condition prognosis.

The target service life of the structure and the relevant limit states and corresponding safety indices for the different structural elements were defined individually, taking the significance of the structural element under consideration and their accessibility during use into account. For ultimate limit states, reliability indices are given in national standards. However, for elements of minor structural relevance, lower safety indices can be agreed upon. For the considered car deck, the serviceability limit state of reinforcement depassivation and a corresponding target reliability index of $\beta = 1.0$ at the end of the target service life T_{sl} (50 years) has been agreed upon. This comparably low reliability index was considered acceptable, as the corrosion monitoring devices allowed for a steady control of chloride ingress and the identification of possible corrosion risks before depassivation.

3.2 Model

To carry out a service life design for reinforcement corrosion, a model is needed to describe chloride ingress into the concrete and predict the point of time at which a critical chloride threshold value at reinforcement level is exceeded and depassivation takes place. In the following the relevant equations are introduced briefly. For further information see the fib Model code (2006).

Reinforcement embedded in concrete is protected against corrosion due to the high alkalinity of the pore solution. A thin iron oxide layer, the so-called passive layer is formed on the steel surface. Among others the passive layer can be destroyed locally by ingress of chlorides from deicing salt if the chloride concentration exceeds a critical concentration C_{crit}. For parking decks it is generally assumed that reinforcement corrosion occurs as soon as depassivation takes place.

The applied model for chloride ingress into the concrete is based on the 2nd law of diffusion assuming that diffusion is the dominant transport mechanism. This is a good assumption for concrete surfaces constantly submerged, however for an intermitting chloride impact the chloride concentration close to the surface varies with time of exposure due to wetting and drying action. For that reason, an adaption of the diffusion-based model became necessary. According to Gehlen (2000) the limit state of chloride induced depassivation can be described by means of equation 1:

$$g\left(C_{crit.}, C(x = d_c, t)\right) = C_{crit.} - \left\{ C_{S,\Delta x} \cdot \left[1 - \mathrm{erf} \frac{d_c - \Delta x}{2 \cdot \sqrt{D_{RCM,0} \cdot k_e \cdot k_{t} \cdot \left(\frac{t_0}{t}\right)^a \cdot t}} \right] \right\} \tag{1}$$

$C_{crit.}$:	critical corrosion-inducing chloride content at depth of reinforcement [wt.-%/cem.]
$C_{S,\Delta x}$:	substitute chloride content at depth Δx (intermitting chloride impact) [wt.-%/cem.]
d_c:	concrete cover depth [mm]
Δx:	depth of convection zone layer [mm]
$D_{RCM,0}$:	rapid chloride migration coefficient [mm^2/a]
k_e:	factor considering temperature impact on effective chloride diffusion coefficient [-]
k_t:	transfer variable (test method) [-]
t_0:	time of reference for testing, here: 28d [a]
a:	ageing exponent [-]
t:	time of exposure up to the target service life T_{sl} [a]

The dimensionless correction factor k_e accounts for the influence on the actual onsite temperature and is given by

$$k_e = \exp\left(b_e\left(\frac{1}{T_{ref}} - \frac{1}{T_{real}}\right)\right) \tag{2}$$

where T_{real} is the onsite temperature, T_{ref} is the reference temperature, and b_e is a regression variable (all in [K]).

3.3 General Input Variables for Service Life Design

The input variables to describe the resistance of the reinforced concrete basically take the concrete cover and the concrete composition into account. As a binder a CEM I with 50 kg/m³ fly ash addition was chosen due to the beneficial aging behavior with respect to chloride ingress (comparably high ageing coefficient). Furthermore, a comparably low effective water-cement ratio of 0.45 was selected to achieve a high chloride ingress resistance due to the rather low concrete porosity. The chloride migration coefficient at an age of 28 days was determined to $D_{RCM,0} = 11.1 \cdot 10^{-12}$ m²/s within compliance tests. The nominal concrete cover was originally set to 45 mm, but the statistical evaluation of quality control measurements carried out after completion rendered an average concrete cover of $\mu = 50.3$ mm and a standard deviation of $\sigma = 9.0$ mm.

An overview of all quantified data for the calculation is given in Table 1 incorporating also data from compliance testing and quality control.

Table 1. Quantification of input variables for the service life design after completion.

variable		unit	distribution*	mean value	standard deviation	reference
$C_{crit.}$		[wt.-%/ cem.]	BetaD $0.2 < C_{Crit.} < 2$	0.6	0.15	cp. Gehlen (2000)
$C_{S,\Delta x}$		[wt.-%/ cem.]	LogND	2.0	1.0	cp. Gehlen (2000)
d_c		[mm]	LogND	50,3	9	quality control
Δx		[mm]	BetaD $0 < \Delta x < 50$	8.9	5.6	cp. Gehlen (2000)
k_e	b_e	[K]	ND	4,800	700	
	T_{ref}	[K]	C	293	-	
	T_{real}	[K]	ND	282	8	measurement data
$D_{RCM,0}$		[mm²/a] ([10^{-12}m^2/s])	ND	350 (11.1)	70 (2.2)	compliance testing
k_t		[-]	C	1	-	
A(t)	a	[-]	BetaD $0 < a < 1$	0.6	0.15	cp. Gehlen (2000)
	t_0	[a]	C	0.0767	-	
t		[a]	C	variable	-	$1 < t < T_{sl} = 50$ a

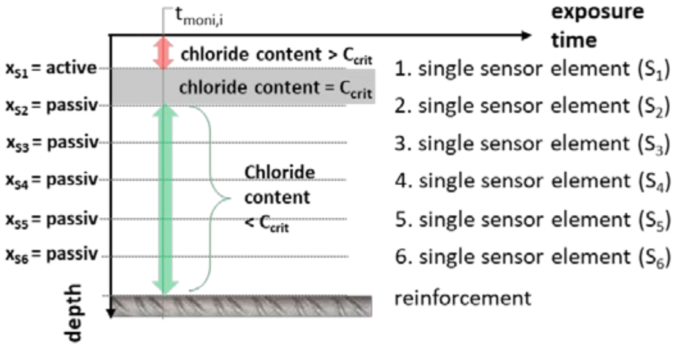
*ND: normal distribution; C: constant; BetaD: beta-distribution; LogND: log-normal distribution

To determine the system reliability over the service life the limit state equation (Equation 1) has to be evaluated, taking the respective quantities for variables into account, cp. Table 1. Based on the results of compliance testing for both the concrete cover and the concrete properties, the reliability index with respect to depassivation was determined to be appr. $\beta = 1.1$ at the end of the target service life T_{sl} (50 years). The calculations were carried out with the software package STRUREL, cp. RCP (1995).

4 CORROSION MONITORING OF PARKING DECKS (UNCRACKED AREAS)

About 30 corrosion sensors (Anode Ladders) were placed in the parking decks. These sensors are located in the uncoated surface areas and allow for a constant control of the ingress of the depassivation front toward the reinforcement. The readout of monitoring data and its interpretation takes place in regular intervals (currently once a year). Since the quantification of input parameters and the prediction of the depassivation is associated with several uncertainties, the monitoring system is intended to give additional information on the actual structural behaviour and hence help to reduce uncertainties associated with the assumptions made.

The corrosion sensor consists of six sensor elements located at different depths within the concrete cover. Corrosion of a single sensor element can be determined by means of half-cell potential and corrosion current measurements. With this information, the ingress depth of the depassivation front x_{crit} can be assessed to be between the last active x_{active} and the first passive x_{passiv} sensor element at any given monitoring time t_{moni}, cp. Fig. 3.



$$x_{activ}(t_{moni}) \leq x_{crit}(t_{moni}) < x_{passiv}(t_{moni})$$

Figure 3. Information of the chloride penetration depth linked to the critical corrosion initiating value derived from the anode ladder data.

The statistical evaluation of x_{active} and x_{passiv} for all sensors leads to a cumulative frequency of active and passive sensor depth and thereby outlining the uncertain depth in which the depassivation front is located (Fig. 4). For the parking decks in this case study, the analysis was carried out for the first time after two years of exposure (Fig. 5).

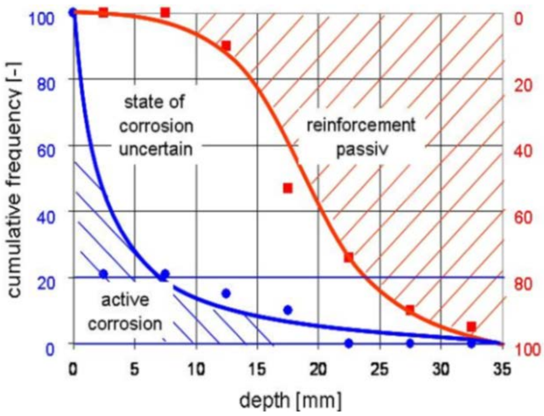


Figure 4. Evaluation of corrosion sensors after two years of exposure.

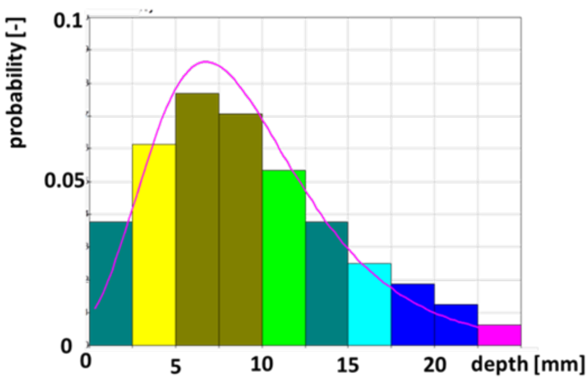


Figure 5. Probability distribution of the depassivation front after 2 years of exposure by using information from Anode Ladders.

The information of active sensor elements (x_{active}, lower bound of depassivation depth) and passive sensor elements ($x_{passive}$, upper bound) can be described statistically. By assuming a uniform distribution of x_{crit} within these boundaries, the location of the depassivation x_{crit} front can be evaluated statistically by means of mean value (Equ. 3) and standard deviation (Equ. 4), Sodeikat et. al (2006). Subsequently the probability distribution of the depassivation front can be determined as shown in Figure 5.

$$m = \frac{x_{activ}(t_{moni}) + x_{passiv}(t_{moni})}{2} \tag{3}$$

$$s = \frac{x_{activ}(t_{moni}) + x_{passiv}(t_{moni})}{\sqrt{12}} \tag{4}$$

The information on the depassivation depth x_{crit} derived from monitoring data can be incorporated into the service life calculation by an additional equality constraint, cp. (Equation 6).

$$x_{crit}(t_{moni}) = erf^{-1}\left(\frac{C_{S,\Delta x} - C_{crit.}}{C_{S,\Delta x}}\right) \cdot 2 \cdot \sqrt{D_{Eff,C}(t_{moni}) \cdot t_{moni}} + \Delta x \tag{6}$$

By adding a further equality constraint for each monitoring event, the reliability index over service life (here: 50 years) can be updated by means of the Bayesian update. The quantification of the monitoring variable x_{crit} after 2, 4 and 6 years is given in Table 2. The monitoring results after 6 years showed that no additional sensor elements turned active, i.e. the quantification of x_{crit} after 4 and 6 years is identical.

For the time being, this updating procedure has been carried out three times (after 2, 4 and 6 years of exposure). The effect of incorporating monitoring data derived from the Anode Ladders on the reliability index can be observed in Figure 6.

Table 2. Quantities of input variable from monitoring for update.

variable	unit	distribution*	mean value	standard deviation	reference
$x_{crit}(t_{moni,1})$	[mm]	GumbelMax	9.25	5.46	evaluated monitoring data
$t_{moni,1}$	[a]	C	2	-	after 2 years of exposure
$x_{crit}(t_{moni,2})$	[mm]	GumbelMax	14.1	7.56	evaluated monitoring data
$t_{moni,2}$	[a]	C	4	-	after 4 years of exposure
$x_{crit}(t_{moni,3})$	[mm]	GumbelMax	14.1	7.56	evaluated monitoring data
$t_{moni,3}$	[a]	C	6	-	after 6 years of exposure

*GumbelMax: Gumbel max distribution; C: constant

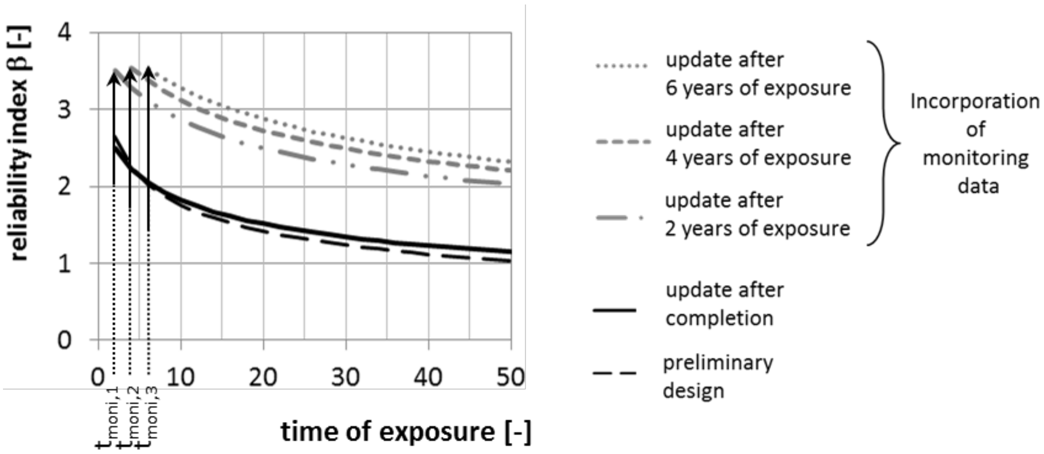


Figure 6. Calculated reliability indices (preliminary design and update after completion and by incorporation of monitoring data) vs. time of exposure.

It can be observed that the incorporation of monitoring data leads to a significant increase of the reliability index over service life. Apparently, monitoring devices itself do not lead to an increase of the structural resistance, but in the shown example the reduction of model uncertainties leads to the observed increasement. Based on the update results, no extra intervention will be necessary in the uncracked areas of the parking decks.

It should be noted that due to the open construction of this parking garage significant amounts of rainwater were able to enter the parking garage. As the deck surface has been smoothed after concreting, the anti-slip protection could not be guaranteed under these boundary conditions. Therefore, after 9 years of exposure the owner decided to coat the deck slabs completely to ensure traffic safety. This time span corresponds approximately to the service life of an average crack-bridging coating system. Therefore, it can be noted that despite this unforeseen development one coating cycle has been saved due to the chosen approach. The surface areas under which the corrosions sensors have been installed have not been coated in order to make sure that further information with respect to chloride ingress over the remaining service life of the parking garage can be collected.

5 CONCLUSION

In this paper a case study is presented in which data from corrosion sensors (in this case Anode Ladders) installed into parking decks of a major parking garage is being used to update the a-priori service life design with respect to chloride-induced reinforcement corrosion. Chloride-induced corrosion is in most cases the governing deterioration process for parking structures and of high economical relevance as consequences can be costly. During the design stage both the environmental impact and the material resistances are unknown and will thus be incorporated with correspondingly high variation coefficients, leading to a great extent of uncertainty for the a-priori design. Integration of monitoring data during the use of the structure by means of a Bayesian Update will led to a deeper understanding of the structural condition thus enables the owner to schedule the optimal point-of-time for intervention measures. The installation of monitoring devices in heavily exposed areas (‘hot spots’) can in some cases be an economic alternative to expensive protective measures.

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