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Experimentation and stochastic modelling for reliability evaluation of an existing bridge

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ABSTRACT: *Bridge structures in the north of Algeria are at risk from aging, leading to structural deterioration from service demands from increasing traffic and heavier loads, from aggressive environmental attack and other physical mechanisms. The paper present the information from the in situ assessment of reinforced concrete structures in marine environment. The test procedure used and the results are presented. These results will serve as the basis for the application of the probabilistic model, in an attempt to evaluate the service life of these structures. In this paper the assessment of reliability profiles for reinforced concrete bridges exposed to chloride attack is discussed. The stochastic service lifetime of an existing RC highway bridge is estimated. The durability investigations of the structures are presented.*

Keywords: *Reliability-bridge-durability-Stochastic modelling- reinforced concrete.*

I. INTRODUCTION

An important problem with the analysis of corrosion of reinforcements used in concrete structures is the calculation of corrosion initiation time. The periodic inspection of highway bridges in Algeria plays a critical role in ensuring the safety, serviceability, and reliability of bridges. There are two significant reasons why we should be interested in reliability evaluation of a RC highway bridge. The first one is to improve our understanding of the actual loading environment and the corresponding bridge responses. There are several significant gaps in our understanding of the actual service and damage-capacity limit state loading effects, especially intrinsic ones, and how these loading effects change with time as a bridge ages and deteriorates. With this knowledge, Stochastic modelling for reliability evaluation, better design, construction, and maintenance practices can be initiated. The second reason for evaluating reliability would be to explore whether the information from assessment of the service lifetime of existing reinforced concrete bridges may properly complement the current practice of visual bridge inspection, provide an objective measurement of the state-of-health, and most importantly, alert responsible officials in the case of (approaching or occurred) failure. It is therefore essential to assess the service life of concrete bridge decks and to develop optimal strategies for inspection and repair. Recognizing the variability in the design, condition, and operating environments of bridges would provide for inspection requirements that better meet the needs of individual bridges to improve both bridge and inspection reliability.

Corrosion of the reinforcement is the main reason for deterioration of concrete bridges. Therefore, stochastic modelling of the corrosion process is an important aspect of the evaluation of the service life time. Although the theory of reliability analysis has been well-established [6, 10], there are only limited successful real-life examples of the use of structural performance defined in reliability theory terms for decision-making. The chloride penetration into concrete matrix is paramount to corrosion initiation, several studies have been focused on understanding and modeling the phenomenon. The most important threat to durability was found to be corrosion of reinforcement due to chloride ingress, mainly in older structures with relatively low concrete cover to the reinforcement [21]. However, under various assumptions, this phenomenon can be simplified to a diffusion problem [17]. There are other analytical solutions to the Fick's second law based on the Mejlbro function [13]. The performance criteria are defined as limit states. The limit state is the border that separates desired states from the undesired or adverse states in situations, acceptable to the owner, which a structure may be subjected to during its lifetime (DuraCrete 1999) [7]. DuraCrete's final report includes models for predicting corrosion initiation due to chloride ingress and due to carbonation as well as models for propagation of corrosion and subsequent cracking and spalling (DuraCrete R17, 2000) [8]. Using the DuraCrete methodology it is possible to quantify the reliability of a structure with respect to predefined limit states that concern durability [18]. The effect of the random variable on the corrosion of the reinforced concrete bridge corrosion initiation risk is studied with respect to penetration of chloride salts.The main objective of this paper is to assess reliability of RC highway bridge using probabilistic model for the durability analysis of concrete structures in marine environment, and understand how the model probabilistic parameters influence durability analysis. The goal is to provide engineers with an aid that helps to understand the impact their decisions have on the durability performance. Another goal essential is the use of performance based criteria in the specification of parameters for the evaluation of concrete durability of bridge.

II. DETERMINISTIC AND PROBABILISTIC MODELS FOR DURABILITY DESIGN

Reinforcement corrosion in concrete is the predominant factor in the premature deterioration of reinforced concrete structures. For service life prediction to be possible, the progress of deterioration mechanisms likely to be contributing to loss of serviceability must be modelled. Modelling explicitly the various uncertainties through a probabilistic-based method leads to reliable models for the prediction of service life of reinforced concrete structures. The model presented by Tuutti [17] is frequently used for representing the deterioration process. Most of the models are diffusion models,which means that all of them considered diffusion as the main transport mechanism, but they do not necessarily model the actual physical and chemical processes. The service life of corrosion-damaged reinforced concrete structures is idealized here as a two-phase process: an initiation stage, in which chlorides penetrate the concrete and reach the reinforcement in sufficient quantities to initiate corrosion, and a propagation stage, in which distinct levels of damage build up are attained.

II.1 Initiation and propagation phase models

In the 1970's Fick's laws were introduced by Collepardi [3] for describing the ingress of chlorides into concrete.

$$J = -D \frac{\partial c}{\partial x} \qquad (1)$$

where:

J : ionic flow, kg/m²s;

D : diffusion coefficient, m²/s ;

c : chloride concentration in solution, kg/m³;

x : distance, m.

To describe the diffusion process, the particle density must be function of space and time. Starting with Fick's 1st law and considering the law of mass conservation, Fick's second law is obtained.

$$\frac{\partial c}{\partial t} = D \frac{\partial^2 c}{\partial x^2} \qquad (2)$$

where t is time in seconds.

However, in simple cases, for pre-determined border conditions and with a constant diffusion coefficient, the solution for (2) is given by (3)

$$C(x,t) = C_i + (C_s - C_i)\left\{1 - erf\left(\frac{x}{2\sqrt{D.t}}\right)\right\} \qquad (3)$$

where:

C_i : initial chloride content, kg/m³;

C_s : surface chloride content, kg/m³;

erf : error function;

x : distance, m;

D : diffusion coefficient, m²/s ;

t : time, s;

$C(x,t)$: chloride content at depth x and time t, kg/m³.

$C(x,t)$: C_s for x=0 and t > 0, and also $C(x,t)$: C_i for x>0 and t = 0.

The time for the chloride content to reach a predefined concentration (usually the critical chloride concentration) at depth x. Considering that $C(x,t) = C_{CR} - C_i$

$$t = \left[\frac{x}{(2.erfc^{-1}(\frac{C_{CR} - C_i}{C_s - C_i}))} \right]^2 / D \quad (4)$$

where,

C_{CR} : critical chloride content, kg/m³.

Laboratory testing [11, 12] and results from existing structures showed that the concrete's age dependency of the coefficient obeys a straight line in a double logarithmic co-ordination system. This meant that the diffusion coefficient could be written as a power function.

$$D(t) = D_0 \cdot \left[\frac{t_0}{t} \right]^{\alpha} \quad (5)$$

where:

$D(t)$: time dependant diffusion coefficient, m²/s ;

t_0 : time when reference diffusion coefficient D_0 was measured, s;

t: time when D(t) is valid, s;

D_0 : reference diffusion coefficient at time t_0, m²/s ;

α : parameter to be determined by regression analysis of test results.

Corrosion of reinforcement starts when the concentration of ions reaches the critical concentration C_{CR} level. The time of reinforcement corrosion initiation T_i is obtained as follows by setting the value of equation (3) to C_{CR} for the reinforcement surface ($x=d_c$ [Thickness of the concrete cover]):

$$T_i = \frac{d_c^2}{4D} (erfc\,(\frac{C_s - C_{CR}}{C_s - C_0}))^{-2} \quad (6)$$

The time by which concentration of chlorides on reinforcement surface reaches the critical level C_{CR} is calculated by considering variations of diffusion coefficient to be a time history and integrating equation (5) to the desired time limit and putting the result into the diffusion equation.

$$T_i = \left[\frac{(1-n)C^2}{4D_0 t_0^n} (erfc \left(\frac{C_s - C_{CR}}{C_s - C_0} \right)^{-2}) \right]^{\frac{1}{1-n}} \quad (7)$$

The propagation time is the time from corrosion initiation until a specified level of corrosion induced damage state is attained. Based on this phenomenon, the probability of corrosion induced concrete cracking $p_c(t)$ at time t can be considered as follows:

$$p_c(t) = P[\sigma_c(t) \geq \sigma_T] \quad (8)$$

where $\sigma_c(t)$ is the stress asserted by the expansive corrosion products and σ_T , is the minimum stress required to cause the cracking of concrete cover.

III. MODELLING PARAMETERS CORROSION AS RANDOM VARIABLES

III.1 Duracrete model

The European Union project, Duracrete (2000) [8], proposes an expression similar to equation (3) which considers the influence of material properties, environment, concrete aging and concrete curing on the chloride diffusion coefficient.

$$C(x,t) = C_s \left\{ 1 - erf \left(\frac{x}{2\sqrt{k_e k_t k_c D_0 (\frac{t_0}{t})^{nDt}}} \right) \right\} \quad (9)$$

where:

k_e is an environmental factor;

k_t is a factor which considers the influence of the test method to measure the diffusion coefficient D_0 ;

D_0 is the chloride migration coefficient measured at defined compaction, curing and environmental conditions;

k_c is an influence factor for concrete curing;

t_0 is the reference period to measure D_0 and nD is the age factor.

The lifetime assessment resulting from this approach is better than the one provided by equation (3) because it accounts for type of concrete, w/c ratio, environmental exposure (submerged, tidal, splash and atmospheric), aging and concrete curing. In addition, the strength of the Duracrete approach lies in considering the randomness related to chloride penetration.

III.2 Probabilistic Modelling

The durability design focus using reliability-based design approach is to represent the Fickian model in the form of a limit state function. For this study, the limit state is initiation limit-state described by the depassivation of reinforcing steel in concrete.

The initiation limit-state will be represented by two functions: $C(x,t)$ function which represents the action effect and; C_{CR} representing the critical chloride content, and which when exceeded leads to steel deppasivation. The initiation limit-state condition is represented by Equation.

$C(x,t) = C_{CR}$ for $t = t_1$.

The relationship between $C(x,t)$ and C_{CR} is modelled mathematically by a function of basic variables $G(.)$.

$$H = C_{CR} - C(x,t) = G(x, k_e, k_t, k_c, D_0, t_0, nD, C_{CR}) \quad (10)$$

$$H = C_{CR} - C_s\left\{1 - erf\left(\frac{x}{2\sqrt{k_e k_t k_c D_0 (\frac{t_0}{t})^{nDt}}}\right)\right\} \quad (11)$$

All the variables in the limit state function are represented as stochastic quantities. The statistical information for each parameter is exploited to provide improved uncertainty estimates of the output, which is expressed in terms of the probability limit state function failure (P_f)(Equation 12).

$P_f = P[C_{CR} - C(x,t) \geq 0]$.

IV. RELIABILITY ANALYSIS FOR CONDITION ASSESSMENT OF EXISTING CONCRETE STRUCTURES

The example considers a two concrete column subject to chloride ingress. The structure is assumed to be a load-carrying part of an existing bridge situated in Algeria seawater. Figure 1 shows that the column is subject to severe weather.

a) Column (A) b) Column (B)

Figure 1. Column subject to chloride ingress of an existing bridge

IV.1Modelling of Stochastic Variables

IV.2 Condition assessment of Column (A) and Column (B)

IV.3 Concrete cover measurements and Chloride profiles

An extensive measurement routine of the concrete cover was performed on both columns. Chloride profiles were measured on two distinct surfaces of each column: the underside and the lateral side. The measured profiles are indicated in Table 1, where the second row indicates the time of measurement (time in years from construction to measurement). The measurement uncertainty related to each chloride measurement is modelled by a normally distributed variable with expected value 0 and a standard deviation 0.02. The concrete cover is assumed log-normally distributed with a coefficient of variation 0,15.

Table 1.Measured chloride profiles.

Profile N° 1 "Column (A)"		Profile N° 2 "Column (B)"	
32.5 years		32.5 years	
Concrete cover (mm)	Content of chloride ($\%$)	Concrete cover (mm)	Content of chloride ($\%$)
1.5	0.97	1.5	1.24
4.5	1.23	4.5	1.30
7.5	0.88	7.5	1.02
10.5	0.81	10.5	0.91
13.5	0.74	13.5	0.82
16.5	0.69	16.5	0.73
19.5	0.61	19.5	0.66
22.5	0.51	22.5	0.52
25.5	0.49	25.5	0.41
28.5	0.41	28.5	0.38
31.5	0.31	31.5	0.29
34.5	0.19	34.5	0.22
37.5	0.10	37.5	0.11
40.5	0.05	40.5	0.06
43.5	0.04	43.5	0.05
46.5	0.03	46.5	0.04
50	0.01	50	0.01

The expected values and the standard deviation are given in Table 2.

Table 2.Standard deviations of the diffusion parameters considering fitting and measurement uncertainty.

	C_s ($(\%)$ chloride of dry concrete)		D (mm^2/year)	
	$E(C_s)$	σ	E(D)	σ
Profile N° 1	1.12	0.02	13	0.7
Profile N° 2	0.98	0.02	13	0.7

IV.4 Critical chloride content

The critical content of chloride depends on the w/b ratio and the depth of the concrete cover. In the present study the critical content of chloride is assumed normally distributed (C_{cr}=1.38 kg/m³) and a coefficient of variation=0,10. The characteristics of the other random variables of the model are given in Table 3.

Table 3. Input parameters for the model

Parameters	Mean	Std. Dev	Distribution
k_e	1	0.1	Normal
k_t	0.85	0.2	Normal
k_c	1	-	Deterministic
t_0	0,078 years taken equal to 28 days	-	Deterministic
n	0.25	-	Deterministic

In order to count exactly the concentration of surface chloride it is necessary to consider operation conditions and the construction environment. In the present example the concentration of surface chloride D_0 (Mean=1,2.10^{-12} m^2/s) is assumed log-normally distributed with a coefficient of variation 0,57 [5].

IV.5 Probabilistic approach

The probabilistic approach is based on the Monte Carlo Simulation (MCS). When a simulation method is used for calculating the probability of failure, the failure function is calculated for each outcome. If the outcome is in the failure region, then the contribution to the probability of failure is obtained. The probability of failure is estimated by the following expression:

$$P_f = \frac{1}{N}\sum_{J=1}^{N} I\left[G(r_j, s_J\right] \quad (13)$$

N :number of simulation;

$I\left[G(r_j, s_J\right]$:indicator function;

$G(r_j, s_J)$: limit state equation;

s represents the environmental load;

r is the resistance of the concrete against chloride penetration.

IV.6 Impact of the concrete cover depth

From Figure 2, the effect of concrete cover on service life is clearly noticeable. Small increases in concrete cover vary significantly the performance of the concrete throughout the service life. This effect is even greater at earlier ages, for example 10 years, where the difference in probability of failure between a concrete cover of 25 mm and 50 mm is almost 45 %.

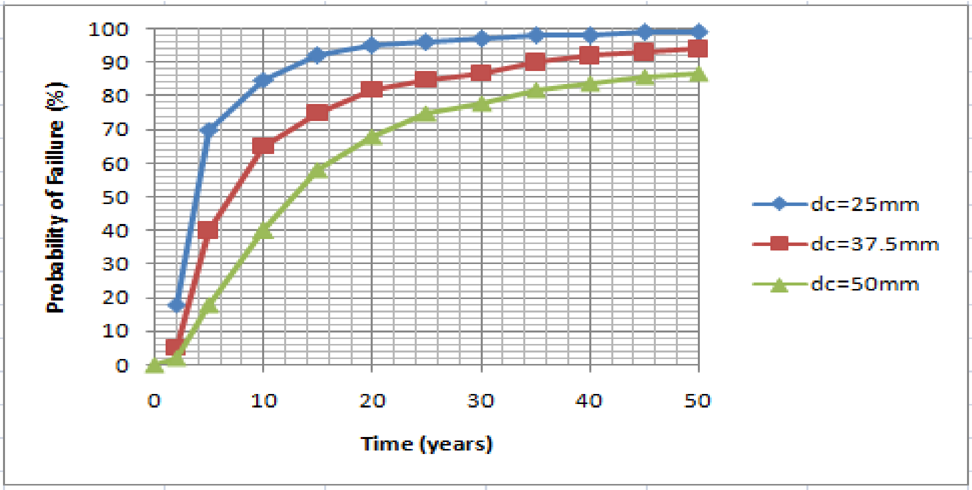

Figure 2 . Effect of various concrete covers (d_c) with time.

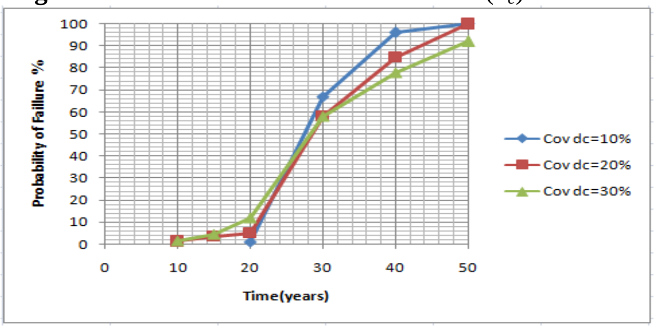

Figure 3. Influence of concrete cover scatter on the probability of failure.

The influence of the scatter on the concrete cover measurements on the probability of failure can be observed in figure 3. The initiation of corrosion in the case of Cov (d_c) = 10% starts late from 20 years in contrast to other cases of Cov. When t = 25years there is almost total convergence (inflection point) in the failure rates of different coefficients of variation of the concrete cover depth C. The change rate of failure for the two cases (Cov=10% and 30%) is reversed after 25 years. A low failure rate in the case of Cov (d_c) = 30% after the point of reversal of the two curves. This explains why in the case of Cov (d_c) = 30% (high dispersion values of the coating), the initiation of corrosion starts faster than in the case of Cov (d_c) = 10%, this is due to low coating thicknesses in some points of the surface. While in another point of the surface, the thickness of the coating is greater, hence the delay to reach 100% of the rate of failure as compared to the case of Cov (d_c) = 10%. In conclusion, the concrete cover depth is significantly influence the rate of failure of a reinforced concrete element in a marine environment. The formwork and reinforcements implementation must respect the standards code in order to reduce cover uncertainties.

IV.7 Impact of the Diffusion coefficient

Figure 4 shows the importance of the diffusion coefficient as a parameter for service life design. The differences between $1{,}0.10^{-12}$ m^2/s and 15.10^{-12} m^2/s are significant. An increase of one order of magnitude in the diffusion coefficient results in an improvement of the performance by at least 90 % at 50 years. This effect is still large at earlier ages, for example 10 years, where the difference in probability of failure is almost 55 %.

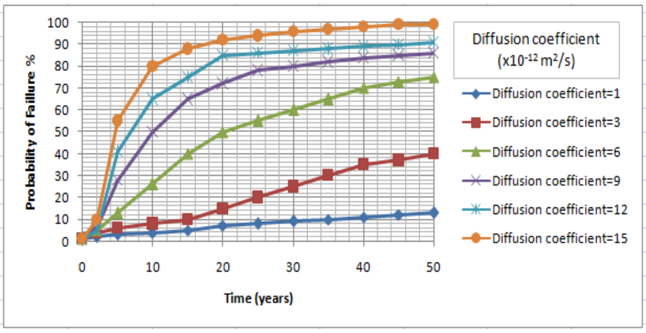

Figure 4. Effect of various diffusion coefficient with time.

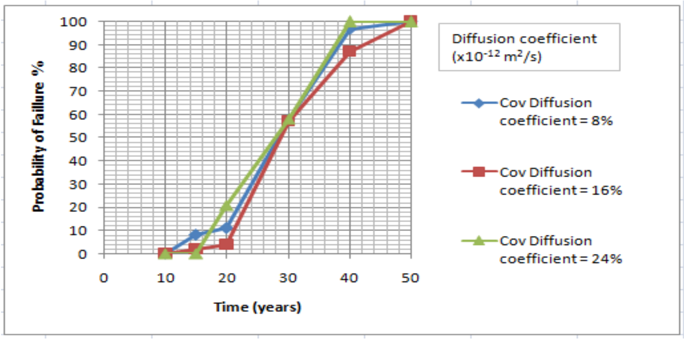

Figure 5 . Influence of diffusion coefficient scatter on the probability of failure.

We have varied the coefficient of variation of the parameter D_0 following three cases (Cov D_0= 8%, 16% and 24%). The results are shown in the figure 5. For t = 30 year the three curves have the same failure rate. It was also noticed at the beginning and the end of the initiation of corrosion, the two extreme values of Cov D_0 (8% and 24%) yield higher than the reference Cov failure rate (16 %). This analysis shows that it is important to limit the amount of chlorides in concrete (penetration by diffusion), playing on the w/c ratio, and generally on the implementation that gives the concrete a certain compactness.

IV.8 Impact of the Critical chloride content

The average critical chloride thresholds were varied from 0.06 % to 0.2 % by weight of concrete with a 10 % Cov. After 50 years, the influence in the probability of failure between the maximum value and the minimum value is 20%. However, at 10 years, the difference is large being approximately 45 %. This suggests that with time, the effect of the different critical chloride thresholds become less significant. This is due to the fact that with time, ever more parts of the structure begin to deteriorate and corrode.

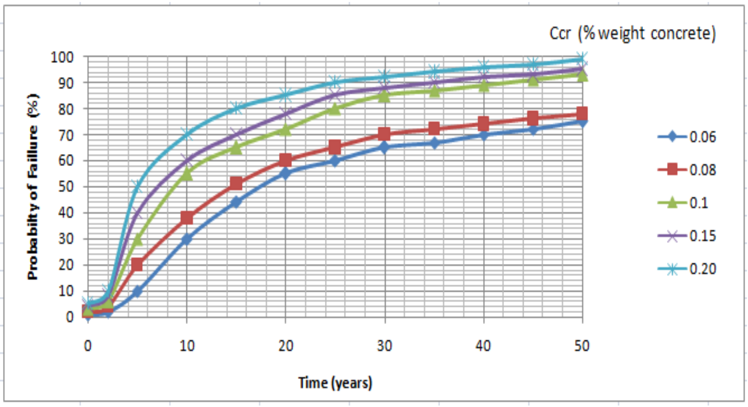

Figure 6 . The effect of different critical chloride thresholds with time.

IV.9 Impact of the Surface chloride content

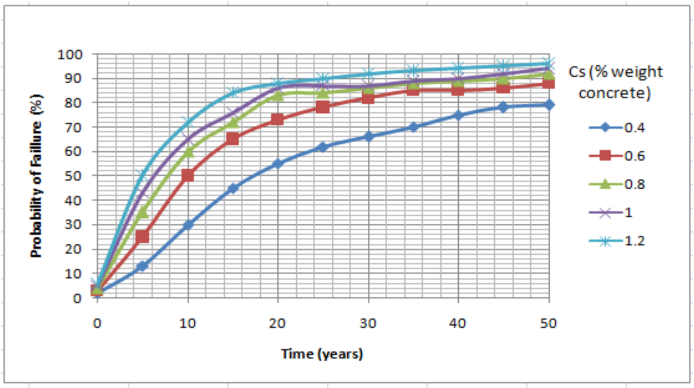

Figure 7. The effect of different surface chloride content on the probability of failure with time.

After 50 years, the influence in the probability of failure between the maximum value and the minimum value is 18%, however, at 10 years, the difference is large, approximately 42 %. This is probable obvious because the lower the surface chloride content, longer time is needed for the chloride to penetrate the concrete cover in sufficient quantities to initiate corrosion. Therefore, at 50 years, the values are closer to one another than at 10 years.

IV.10 Impact of the Age factor

Figure 8 indicate that the age factor influences the performance significantly even more than the concrete cover or the diffusion coefficient. After 50 years, the influence in the probability of failure between the maximum value and the minimum value is 95%.
At 10 years, the difference is still very large, approximately 90 %.

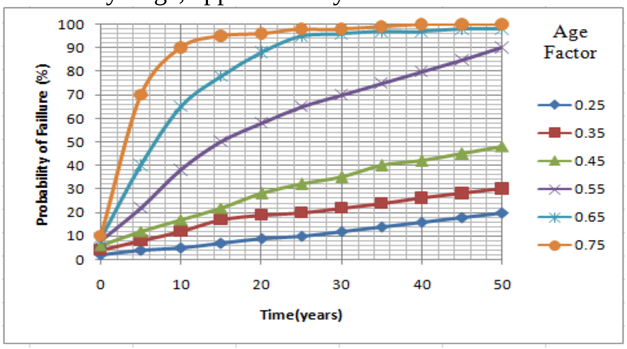

Figure 8 . The effect of age factors of diffusion coefficient on the probability of failure with time.

IV.11 Impact of the environmental factor k_e

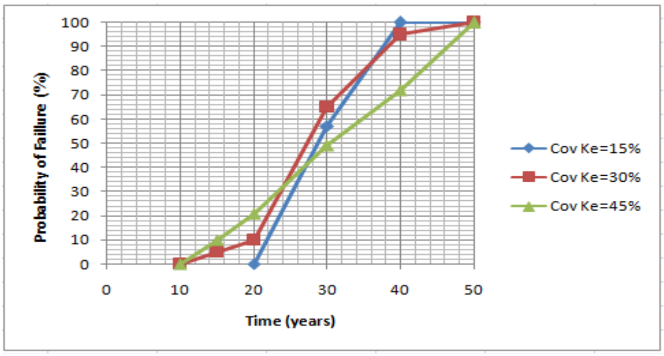

Figure 9 . The effect of the environmental factor coefficient on the probability of failure with time.

From Figure 9, the variant of coefficient of variation of the environmental conditions parameter k_e (temperature, humidity, ..) has an inversely proportional impact with the lifetime of the examined item. For low coefficients of variation, the initiation of corrosion begins slowly and spreads rapidly and conversely for large values of the coefficients of variation. It is concluded that the corrosion rate depends strongly on the moisture content of the concrete as it directly influences the conductivity and the diffusion of oxygen.

IV.12 Impact of the parameter k_t

Similarly for the parameter k_t (taking into account the strength of the concrete), the fluctuation of the coefficient of variation has an inversely proportional impact with the lifetime of the item examined. For small values of coefficients of variation, initiation of corrosion begin slowly and spreads rapidly and conversely for large values of the coefficients of variation. In practice, one must ensure that the induction period is the longest possible. This step can be lengthened by providing sufficient thickness of concrete cover and selecting a mixing formula for decreasing the permeability of concrete.

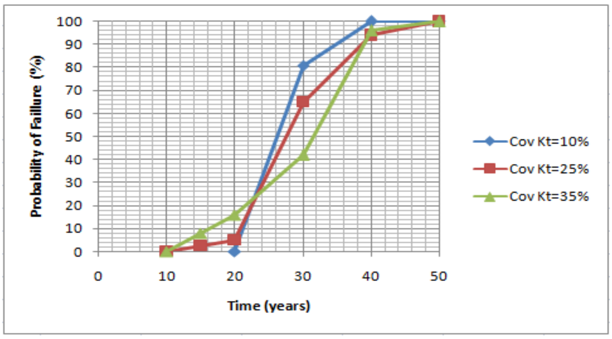

Figure 10 . The effect of the coefficient k_t on the probability of failure with time.

IV.13 Effect of the corrosion rate and C_{CR}

The critical content of chloride and the corrosion rate depends on factors like the w/c ratio and the thickness of the concrete cover d_c. Therefore a parameter study is performed in the following to investigate the effect of this dependency. Considering the corrosion propagation phase the parameter of interest is the corrosion rate. The corrosion rate depends on the depth of the concrete cover and the *w/c* ratio, see figure 11. The present value is chosen considering w/c =0.4 and d_c =50 mm. The corrosion rate is assumed normally distributed and the expected value of R_{corr} is 25 µm/year corresponding to an expected value of T_{corr} , E[T_{corr}]=500 years. The coefficient of variation is estimated to 0,50. Chloride-induced corrosion is normally associated with localised attack. The present values, are average corrosion rates. It is noted that the critical content of chloride in figure 9 is given as % chloride of the weight of binder.

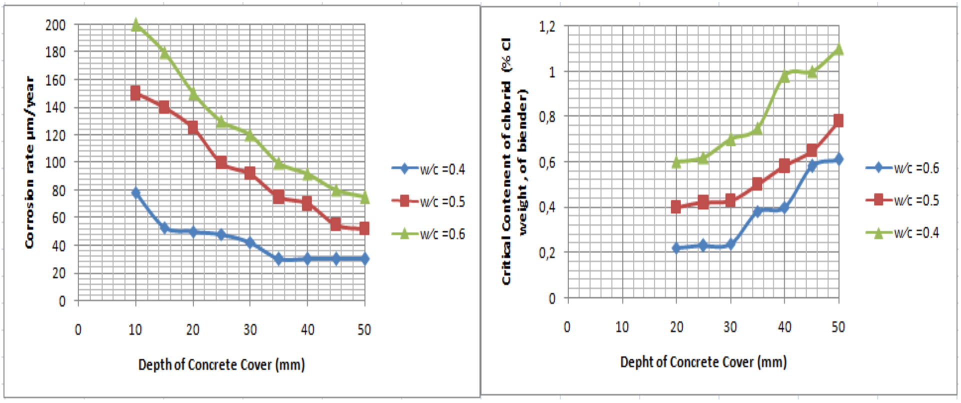

Figure 11. Relation between *w/c* ratio, depth of concrete cover and the corrosion rate and critical content of chloride respectively.

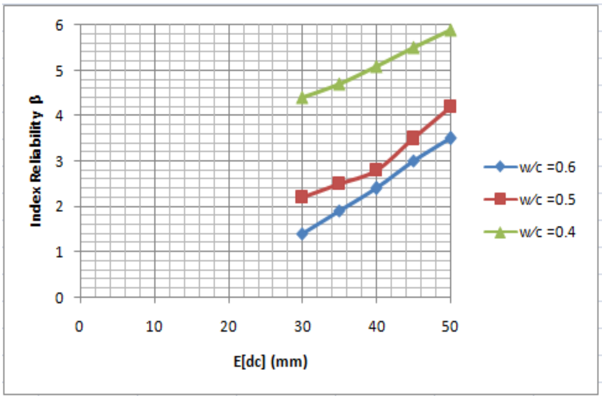

Figure 12. Reliability index for different choices of the mean value of concrete cover and the w/c ratio.

Figure 12 shown that the reliability index increases rapidly with decreasing w/c ratio. In addition it is seen that the reliability index increases with increasing mean value of the concrete cover. The fact that the diffusion parameters, C_s and D_0 depend on the w/c ratio and the depth of the concrete cover is not taken into account.

V. MODELLING AND ESTIMATION OF THE SERVICE LIFETIME

As an introduction a method for the evaluation of the Service Lifetime is presented, see e.g. Thoft-Christensen [19]. The service life time of a reinforced concrete bridge is in this paper defined as the initiation time for corrosion of the reinforcement [19, 20]. On basis of Relation (6) outcomes of the corrosion initiation time has been performed on basis of the following data by simple Monte Carlo simulation (100000 simulations) (calculated using the programme Comrel Version 8.00). A Weibull distribution W(x; μ ,k, ε), where μ = 63.67, k=1.81 and ε =4.79 is used to approximate the distribution of the simulated data. The design corresponding to the data above can be improved in different ways e.g. by increasing the cover d_c or reducing the diffusion coefficient D_0. The serviceability failure probability P_f as function of E[d_c] (in mm) and E[D_0] (in mm^2/year) is illustrated in figure 13.

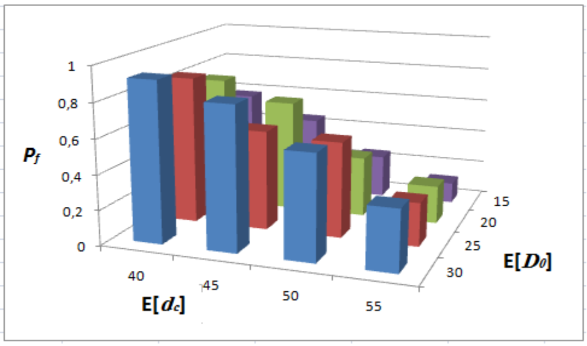

Figure 13. Serviceability failure probability P_f as function of E[d_c] and E[D_0].

The serviceability failure probability is then defined by $P_f = P(T_I - T_D \leq 0)$,where T_D is the design serviceability life time. As expected P_f decreases with decreasing values of E[D_0], and decreased with increasing values of E[d_c].

VI. CONCLUSION

An example considering a reinforced concrete Column subject to chloride ingress has been presented. The effect of random properties of reinforcement concrete cover, diffusion coefficient, surface chlorides concentration, and critical chlorides concentration were taken into account for calculating the corrosion initiation time. From the study, it was clear that the existing database for durability parameters required to be expanded in the future. Application of the described probabilistic and stochastic methodology is relevant for structures in marine environment that should have specified service lives, which is the case for structures such as bridges and wharfs. The comparison of the concrete cover values resulting from a probabilistic and deterministic analysis show that the first represent an increase between 10 to 15 % over the second. This increase of almost 12.5 mm, for a 50 years' service life, is seen to be very important for the long term durability of structures in marine environment

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