Fracture Mechanics of Concrete and Concrete Structures -
Assessment, Durability, Monitoring and Retrofitting of Concrete Structures- B. H. Oh, et al. (eds)
 ISBN 978-89-5708-181-5

Reliability based maintenance planning of RC bridges considering spatial variability of corrosion-induced crack width

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ABSTRACT: Corrosion of reinforcement is the main deterioration mechanism of concrete structures. Although models for initiation phase of corrosion is well developed but there is little in the literature regarding analytical models for propagation phase, including cover crack width. Furthermore considering spatial variability of deterioration has the advantage of estimation of probability and extent of deteriorated area which is a more valuable indicator in infrastructure maintenance planning and management. In this paper some mechanistic analytical models in literature has been reviewed and then using random fields the probability and extent of failure of corrosion affected concrete structures under some serviceability limit state, e.g. critical crack width, is estimated. Finally expected life cycle costs of inspection and repair intervention is calculated with event based Monte Carlo method for various inspection intervals and acceptable extent of deterioration.

1 INTRODUCTION

Corrosion induced deterioration of RC structures, especially bridges due to frequently applied deicing salts, is the main challenge of civil asset managers worldwide. In most Bridge Management Systems condition ratings are based on visual inspections and then Markov chain models used to predict the future bridge conditions. There is a high degree of uncertainty in deterioration mechanisms and also inconsistency exists in inspectors' judgments. This was the motivation for most researchers in the past decade to apply reliability methods for evaluation of deterioration of RC bridges. In the past decade proposed reliability based maintenance management systems mechanistic deterioration models in probabilistic framework are utilized to account for temporal variations of strength and loads (Frangopol et al. 1997).

Although considerable research is accomplished regarding reliability based maintenance planning of bridges but limited research exists in literature considering spatial variability of corrosion. The fact that in concrete structures, due to the spatial variability of workmanship and environmental factors, the material and dimensional properties are not homogeneous and consequently corrosion damage has a spatial variability motivated some researchers to study the effects of spatial variation on reliability models (Li et al. 2004, Vu & Stewart 2005, Karimi et al. 2005, Sudret 2008). These works revealed the utility of considering spatial variability of corrosion parameters in prediction of extent and likelihood of corrosion induced damage in RC structures. Most of these models assume that "failure" occurs when corrosion is initiated or use empirical models for prediction of crack width increase with time. In propagation phase after initiation of surface cracking their width will increase with time to a limit that spalling of concrete is prone. In recent years several efforts have been made to model analytically the propagation phase of the corrosion.(Liu & Weyers 1997, Li et al. 2006, El Maaddawy & Soudki, 2007). In this paper the model proposed by Li et al. (2006) is used for crack initiation and propagation with time.

Extent of damage is an important factor in actual repair and maintenance strategy selection. On the other hand serviceability limit states, e.g. cracking, delamination, spalling, etc., in contrast to strength limit states occur earlier in service life of a bridge and demand more for repair and maintenance interventions. So extent of cracked, spalled or delaminated area (length) seems an appropriate indicator of bridge condition and reliability.

In this paper a two-dimensional spatially variable time-dependent reliability analysis is developed to predict the likelihood and extent of cracking for RC deck top surface exposed to deicing slats. Spatial variability of parameters is modeled using random fields, discretized with midpoint method. This model will consider the random spatial variability of concrete material properties, concrete cover and surface chloride concentration. For repair and maintenance optimization purpose a method considering acceptable levels of extent of damage and inspection and maintenance intervals yielding to minimum life cycle costs is introduced.

2 CORROSION MECHANISM

2.1 *General*

Corrosion of steel in reinforced concrete structures is one of the major causes of their deterioration over time. In bridge decks ingress of chlorides from de-icing salts through the concrete cover deactivates the natural protective oxide layer formed around the reinforcements by the strong alkalinity of pore solution. Once the protective layer has dissolved, if chloride concentration exceeds a threshold value and enough oxygen and moisture are present, corrosion is initiated. In propagation phase since the corrosion products have a volume of three to six times greater than the original steel tensile stresses within the concrete increase which in turn result in longitudinal cracking and spalling at the surface.

2.2 *Initiation Phase*

Numerous studies have found that the penetration of chlorides through concrete can be best represented by a diffusion process if the concrete is assumed to be relatively moist. In this case, the penetration of chlorides is given empirically by Fick's second law of diffusion if the diffusion is considered as one-dimensional in a semi-infinite solid; this is expressed as Equation 1:

$$\frac{\partial C}{\partial t} = D\frac{\partial^2 C}{\partial^2 x} \tag{1}$$

Crank's solution to Fick's second law of diffusion is represented as Equation 2 (ACI 365 1R-00):

$$C(x,t) = C_0 + (C_{sa} - C_0)\left[1 - erf\left(\frac{x}{2\sqrt{D_a t}}\right)\right] \tag{2}$$

In general, the chloride concentration profiles obtained under different climates are used in mathematical models for obtaining parameters of Equation 2. A plot of chloride concentration vs. penetration depth can often be closely described by Crank's solution to Fick's second law of diffusion. The curve-fitting results in these parameters: apparent diffusion coefficient, D_a, apparent surface chloride concentration, C_{sa} and the original chloride concentration, C_0 then $C(x,t)$ express the chloride content in depth of x at time t. Apparent diffusion coefficient (D_{ap}) reflects the influence of all possible transport mechanisms that have contributed to the chloride profile and it is the mean value of the actual coefficient over the period between the initial exposure to the chloride-laden environment to sampling.

The corrosion of reinforcements is initiated when the chloride content, in steel bars embedment depth, X_c, exceeds a threshold value, $C_{cr,}$ which depassivates the steel embedded in the concrete provided that sufficient moisture and oxygen are present. The probability of corrosion initiation with time can be expressed with the following limit state:

$$g(.) = C_{cr} - C(X_c, t) \tag{3}$$

The influence of several factors such as concrete mix proportions, cement type, C_3A content of cement, materials incorporated, w/c ratio, relative humidity, and temperature are amongst the factors contributing to variation of governing parameters, e.g. D_a, C_{cr} , within a large range (Dupart 2007, Kong et al. 2002) but in general the uncertainty of governing parameters can be handled with random variables. Although accurate statistical distributions cannot be derived without sufficient experimental data from existing structure, Dupart (2007) concluding of various measurements in the literature for various environmental and workmanship classifications made some general propositions. The descriptive parameters of random fields and random variables in Table 1 are based on such propositions (Dupart 2007, Stewart & Mullard 2008).

Table 1. Statistical Description of Variables.

Variable	$\theta x = \theta x$ (m)	μ	COV	Distribution
Concrete Cover (mm)	3.5	50	0.2	Normal
Concrete Comprssive Strength, fc, MPa	3.5	35	0.2	Lognormal
Surface Chloride Concentration (kg/m^3)	3.5	3.5	0.6	Lognormal
Critical Chloride Concentration (kg/m^3)	-	0.9	0.19	Uniform
Reinforcement Size (mm)	-	12	0.15	Normal
Workmanship Coefficient (kw)		0.87	0.06	Normal
Porous Media around Reinforcement (d0), µm	-	12.5	-	-
α rust	-	0.57	-	-
ρ_{rust} (kg/m^3)		3600	-	-
ρ_{steel} (kg/m^3)		7850	-	-
Poisson coefficient (ν)		0.18	-	-

2.3 *Propagation Phase*

Considerable research has been undertaken on corrosion-induced cracking process, with perhaps more numerical and experimental investigations than analytical studies(Chen & Mahadevan 2008, Vu et al.

2005). For reliability calculations a closed form mathematical model is of paramount importance. In this regard some models for the prediction of time to crack initiation of corroding reinforced concrete (RC) structures have been developed (Liu & Weyers 1997, El Maaddawy & Soudki, 2007) but little has been on crack width growth with time. Some empirical models based on mathematical regression of experimental results exist in the literature. For example Vu et al. (2007) developed an empirical model to predict the time for a crack to propagate to a width of 1 mm. This model is based on accelerated corrosion tests in laboratory.

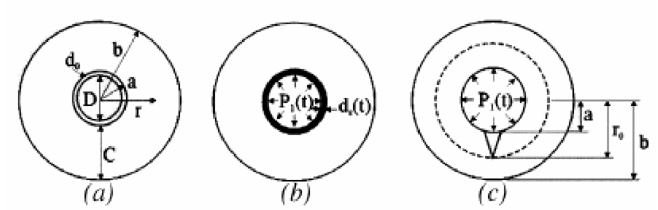

Figure 1. Schematic Representation of Cracking Process [Li et al 2006].

In this paper the analytical model developed by Li et al (2006) will be used for simulation of corrosion-induced crack width variation with time. The merit of this model is that it is directly related to the factors that affect the corrosion such as concrete geometry and property and corrosion rate.

In this model as shown schematically in Figure.1 Concrete with embedded reinforcing steel bars is modeled as a thick-wall cylinder, where d_0 is the thickness of the annular layer of concrete pores (that is, a pore band) at the interface between the steel bar and concrete, D is the diameter of steel bar, and Xc is the concrete cover. The inner and outer radii of the thick-wall cylinder are a = (D + 2d0)/2 and b = Xc + (D + 2d0)/2. When the steel bar corrodes in concrete, its products fill the pore band completely and a ring of corrosion products forms, the thickness of which, ds(t) (Fig.1), can be determined from Equation. 4 as below (Liu et al. 1997):

$$d_s(t) = \frac{W_{rust}(t)}{\pi(D+2d_0)}\left(\frac{1}{\rho_{rust}} - \frac{\alpha_{rsut}}{\rho_{st}}\right) \quad (4)$$

where α_{rust} is a coefficient related to the type of corrosion products, ρ_{rust} is the density of corrosion products, ρ_{st} is the density of steel, and $W_{rust}(t)$ is the mass of corrosion products. $W_{rust}(t)$ increases with time and can be determined from Equation. 5 (Liu et al. 1997):

$$W_{rust}(t) = \left(2\int_0^t 0.105(1/\alpha_{rust})\,\pi D \times i_{corr}(t)dt\right)^{1/2} \quad (5)$$

where $i_{corr}(t)$ is the corrosion current density (in $\mu A/cm^2$). In general formation of rust products on the steel surface will reduce the diffusion of the iron ions away from the steel surface resulting in reduced corrosion rates with time. For example Vu & Stewart (2005) suggested the following time dependent equation suggested for $i_{corr}(t)$:

$$i_{corr}(t) = i_{corr}\,(1) \times 0.85(t - T_i)^{-0.3} \quad (6)$$

where T_i is the time to corrosion initiation and i_{corr} (1) is the corrosion current density in the first year after corrosion initiation which is based on concrete quality and can be calculated using Equation. 7 as below:

$$i_{corr}(1) = \frac{27(1-w/c)^{-1.64}}{X_c} \quad (7)$$

According to Li et al. (2006) the growth of the ring of corrosion products (known as a rust band) exerts an outward pressure on the concrete at the interface between the rust band and concrete. Under this expansive pressure, the concrete cylinder undergoes three phases in the cracking process: 1) not cracked; 2) partially cracked; and 3) completely cracked. In the Phase 1 (no cracking), the concrete can be considered to be elastically isotropic so that the theory of elasticity can be used to determine the stress and strain distribution in the cylinder For a partially cracked concrete cylinder, cracks are considered to be smeared and the concrete to be a quasi-brittle material, so that the stress and strain distribution in the cylinder can be determined based on fracture mechanics When the crack penetrates to the concrete surface, the concrete cylinder fractures completely. Knowing the distribution of stress and strain, the crack width at the surface of the concrete cylinder can be determined as below:

$$w_c = \frac{4\pi d_s}{(1-\nu_c)(a/b)^{\sqrt{\alpha}} + (1+\nu_c)(b/a)^{\sqrt{\alpha}}} - \frac{2\pi b f_t}{E_{ef}} \quad (8)$$

where ν_c is Poisson's ratio of concrete and α (<1) is the tangential stiffness reduction factor. According to Li et al. (2006) it is assumed that the residual tangential stiffness is constant along the cracked surface, that is, on the interval [a, r0], and represented by αE_{ef} , where E_{ef} is the effective elastic modulus of concrete which can be calculated as per Equation 10 where Φ_{cr} and E_c represent creep coefficient and elastic modulus of the concrete (Liu et al. 1997)

$$E_{ef} = \frac{E_c}{1+\varphi_{cr}} \quad (9)$$

In Equation. (9), the key variables are the thickness of corrosion products d_s, which is directly related to the corrosion rate (i_{corr}), and the stiffness reduction factor α, which is related to stress conditions and concrete property and geometry. Equation 8 which is used in this paper has been verified by both numerical and experimental results (Li et al. 2006). In using this equation one needs to calculate time dependent variables α and d_s. The former is determined solving simultaneous equations derived in Li et al.(2006) which are not repeated here for briefness purposes, while the later is calculated according to Equation 4.

2.4 *Effect of Concrete Quality*

Various mechanical properties of concrete are usually correlated to compressive strength of concrete, a parameter which can be easily measured in practice.Refer to ACI 318-05 the concrete tensile strength and modulus of electricity is related to compressive strength as:

$$f_t = 0.53\sqrt{f_c'} \text{ (MPa)} \tag{10}$$

$$E_c = 4600\sqrt{f_c'} \text{ (MPa)} \tag{11}$$

The other important influencing parameter in studying corrosion in RC structures is the chloride diffusion coefficient. Several models have been developed to consider the influence of mix proportions on chloride diffusion coefficient (Bamforth & Price 1996 & Papadakis et al. 1996). But Stewart and Vu (2000) comparing reported data in literature concluded that the model developed by Papadakis et al. (1996) appears to be the best fit to available literature which is represented as:

$$D = D_{H_2O} 0.15 \frac{1+\rho_c \frac{c}{w}}{1+\rho_c \frac{w}{c} + \frac{\rho_c}{\rho_a}\frac{a}{c}} \left(\frac{\rho_c \frac{w}{c} - 0.85}{1+\rho_c \frac{w}{c}} \right)^3 \tag{12}$$

In this model *a/c* is the aggregate-to-cement ratio, ρ_c and ρ_a are the mass densities of cement and aggregates respectively and D_{H2O} is the chloride diffusion coefficient in an infinite solution (=1.6×10^{-5} cm^2/s for NaCl). The water-cement ratio is estimated from Bolomey's formula for Ordinary Portland Cement (OPC) concretes as below:

$$\frac{w}{c} = \frac{27}{f_c' + 13.5} \tag{13}$$

where f_c' is the concrete compressive strength of a standard test cylinder in MPa.

3 RANDOM FIELD

3.1 *General*

Spatial variability of physical properties includes systematic spatial variation, e.g. variation of the mean value and standard deviation which is easily handled considering corresponding random variable, and random spatial variation. Random spatial variability of continuous media can be represented by the use of random fields (Chryssanthopoulos & Sterritt, 2002). In the case of a large surface a 2D random field can be used (Li et al. 2004, Stewart et al. 2006), while in the case of beam elements a 1D random field will be more appropriate[Engelund & Sorensen1998,Karimi et al. 2005 & Sudert 2008].

A simple model for a random field is a homogenous isotropic Gaussian field, where the random variables have a Gaussian distribution that does not change with direction or location therefore the interdependency between two random variables defined at two points depends only on the distance between them.

The correlation function ρ(τ) determines the correlation coefficient between two elements separated by distance (τ) and is representative of the spatial correlation between the elements. As the distance between correlated elements becomes smaller the correlation coefficient approaches unity as defined by the correlation function, and likewise as the distance increases the correlation coefficient reduces. The Gaussian (or squared exponential) correlation function used herein is defined as:

$$\rho(\tau) = exp\left[-\left(\frac{|\tau_x|^2}{{d_x}^2} \right) - \left(\frac{|\tau_y|^2}{{d_y}^2} \right) \right] \tag{14}$$

where $d_x=\theta_x/\sqrt{\pi}$; and $d_y=\theta_y/\sqrt{\pi}$.

θ_x *and* θ_y are the scales of fluctuation for a two dimensional random field in x and y directions, respectively (VanMarcke 1984); and $\tau_x = x_i - x_j$ and $\tau_y = y_i - y_j$ are the distances between centroid of element i and element j in the x and y directions, respectively.

The scale of fluctuation for a 1D random field is defined as per Equation. 15 (VanMarcke, 1984 , Sudert & Der Kiureghian, 2000):

$$\theta = \int_{-\infty}^{+\infty} \rho(\tau) d\tau \tag{15}$$

The parameters of the Gaussian random field are the mean value μ , the standard deviation σ and the correlation length d_x and d_y .

Various methods of discretization of random fields have been proposed (Sudert & Der Kiureghian, 2000). In midpoint method the random field needs to be discretised into *N* elements of identical size and shape. The random field within each element is represented by a single random variable defined as the value of the random field at the centroid of the element and this value is assumed to be constant within the element.

Following the discretisation the covariance function can be replaced by a $n \times n$ covariance matrix C, for which the (i, j) th element is given by Equation 16:

$$C_{i,j}=\rho_{i,j}\times\sigma^2 \qquad (16)$$

The matrix C is a symmetric completely positive matrix and the values on the diagonal refer to the autocorrelation and are equal to the variance of the Gaussian variable. The eigenvalue problem of the covariance matrix is C:

$$C\psi_j=\mu_j\psi_j \qquad (17)$$

A discretised random field, given by a vector *p* of length *n*, can be represented by the KL expansion in the form of Equation 18:

$$H(x) = \overline{H(x)} + \sum \psi_j \sqrt{\mu_j}\zeta_j \qquad (18)$$

where $\overline{H(x)}$ denotes the mean, ζ_j are uncorrelated standard normal (zero mean and unit variance) random variables. The mean $\overline{H(x)}$ and the eigensolutions μ_j and ψ_j are deterministic. The randomness of the field is only included in ζ_j.

If the distribution type is non-Gaussian first a transformation into the Gaussian space is performed using the *Nataf* model. (Nataf,1962). In this paper regression formulas developed by Li and Der Kiureghian (1993) will be used for the mapping of the correlation coefficients to Gaussian space.

4 DESCRIPTION OF METHOD

4.1 *hypothetical bridge specification*

The extent of distress is an appropriate criterion in decision making about repair and maintenance strategy in practice. A spatial time-dependent reliability analysis is developed for a hypothetical RC bridge deck with 12 m length and 10 m width exposed to de-icing salts. The analysis considers corrosion initiation and then propagation of corrosion-induced upper cover cracking until a crack width of 0.3 mm is reached, which is the prescribed limit crack width in Duracrete (2000) and ACI-318-05. A 2D homogenous Gaussian random field is applied to consider the spatial variability of concrete compressive strength. This means that related properties of concrete, e.g. chloride diffusion coefficient, concrete tensile strength and concrete effective modulus of elasticity, water-cement ratio and corrosion density rate follow the variation of the same field of compressive strength of concrete. Furthermore concrete cover depth and surface chloride concentration are represented with random fields to account for spatial variations of these parameters.

According to Stewart and Mullard (2008) the scale of fluctuation of these random fields is approximately 3.5 m ($d_x = d_y$ = 2.0 m). In Table (1) full statistical description of random fields, random variables and deterministic variables are illustrated.

The 2D random field is discretized into square elements of size Δ=0.5 m, resulting in 480 elements. Then whole deck can be considered as a series system of elements in serviceability reliability analysis:

$$G_{sys}(X, t) = \min\left(G_j(X, t) - w_{lim}\right) \qquad j = 1 \text{ to } 480 \qquad (19)$$

In this formulation failure means that at least in one element the crack width is greater than w_{lim}=0.3 mm. The vector *X* represents random variables of every element and *t* is the age of the concrete deck.

The developed code in MATLAB carries out Monte Carlo Simulation in the space of independent standard normal variables of discretized random fields. Figure. 2 represents a random realization of spatial variability of concrete cover depth as a Normal random field.

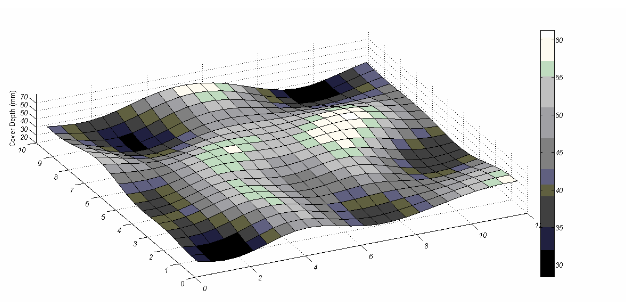

Figure 2. Random Realization of spatially variable cover depth of concrete deck.

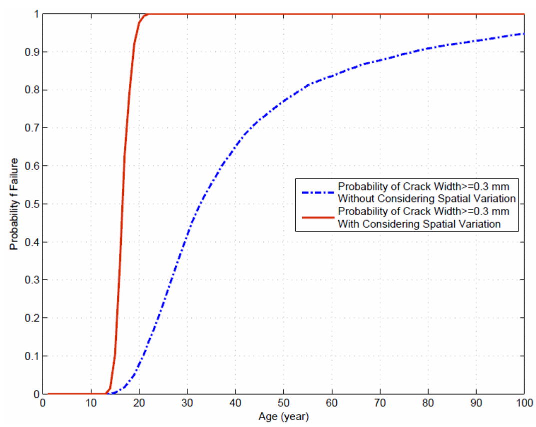

Figure 3. The effect of considering spatial variability on estimated probability of failure.

4.2 *Timing of the first repair*

In the hypothetical example for every discretized element the crack width was calculated 1000 times to simulate the statistical description of extent and likelihood of failure. According to Equation. 19 reliability analysis in the series system of these elements will result in the probability of failure in at least on element. This sophisticated analysis will calculate the probability of inception of deck failure but with due consideration to spatial variation of corrosion phenomenon parameters. As per Figure. 3 the probability of severe cracking considering the spatial variation is extremely higher than the case of considering the whole deck as one element. In the other words ignoring spatial variability will result to substantial underestimation of the probability of failure. For example in year 20 the simple reliability analysis predicts that there is just less than 10 percent probability of failure, while considering spatial variation will result in more than 90 percent of failure probability.

The result of this Monte Carlo analysis is depicted again in Figure 4 where a random realization of the crack width increase in the elements of the bridge deck system is represented. It is obvious that in every cycle of Monte Carlo simulation another realization may occur. The proportion of the failed area to whole deck area in every cycle of Monte Carlo simulation results in a random variable X which is representing extent of damage. In Figure 5 the simulated histograms and fitted density function of the extent of failed area of bridge deck are illustrated in 30, 50, 70 and 100 years of its lifetime.

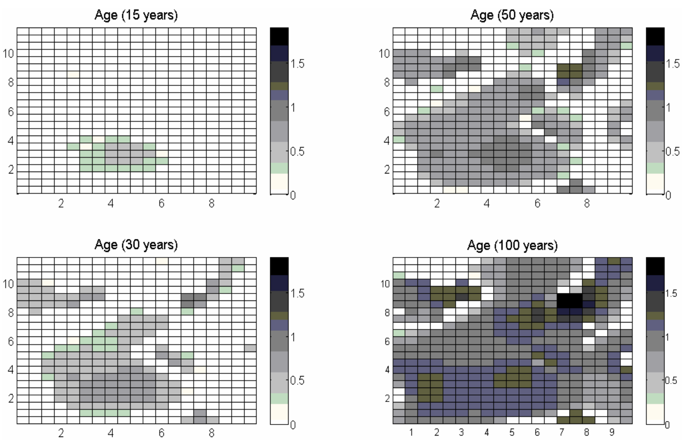

Figure 4. Random realization of spatial variation of crack width increase with time.

As another important result of time-dependent spatial reliability analysis is that the probability of exceeding the extent of failure from an acceptable threshold can be calculated. In Figure. 6 the results of such an analysis is represented for various acceptable limits of X. In risk-based maintenance management of bridges this can be a rational criterion in selecting optimum life cycle repair and maintenance strategies. For example if X=30% is the acceptable extent of failed area while there will be negligible risk prior to age 20 years it will increase dramatically and in age 30 years this probability will be about 95 percent. Such an instantaneous increase in calculated probability for every acceptable threshold X can be described in reference to low coefficient of variation of extent of failure in every time as presented in Figure 8 .

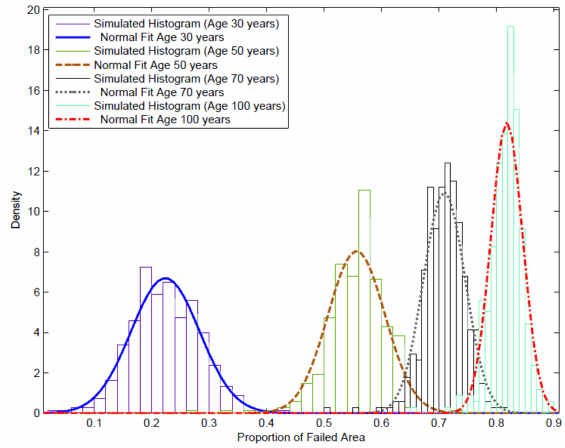

Figure 5. Distribution of extent of damaged area in different ages of service life of bridge deck.

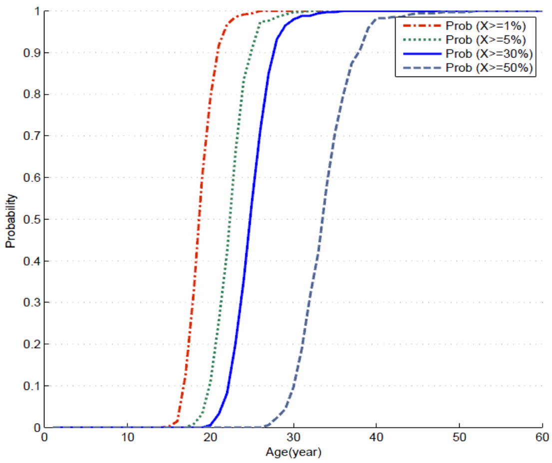

Figure 6. Probability of Exceeding of extent of cracked area from and acceptable threshold.

4.3 *Maintenance and inspection planning considering life cycle costs*

Based on spatial time dependent reliability analysis the propagation of the extent of damage with time can be simulated. As per above results the RC deck cannot meet designed service life without repair. In reality some areas of deck will undergo earlier deterioration and hence will be repaired first. The remaining parts of the deck and even the repaired areas will continue to deteriorate and likely will need repair at later times. It is assumed in this paper that bridge will be inspected on a routine interval times and in every inspection if the extent of damage is more than acceptable threshold it will be repaired. Furthermore it is assumed that repaired areas will be brought to undamaged, new condition. Decisions about selection of optimum inspection and maintenance strategy shall be based on minimum life cycle costs.

Repair costs are a function of damaged area (Stewart et al. 2006). In this paper an idealized linear function is considered:

$$C_{rep}=5000+2000\times\text{Damaged area. (€)} \quad (20)$$

And every inspection has a 1000 € cost.

Since the repair actions depends on the previously failed elements and the history of failures is not evident deriving a closed form formula for this problem is cumbersome In this paper an event-based Monte Carlo simulation is conducted. The discounted present value of repair and maintenance costs are calculated in every simulation run and expected of total cost is considered as the criterion of decision making. It should be noted that for simplicity other costs, especially user costs during repairs is not considered in the analysis. Furthermore this analysis is performed on an existing hypothetical bridge deck. Obviously this analysis in design stage can be conducted considering various design specifications and considering initial construction and design costs in life cycle cost analysis.

$$E(LCC) = C_{ins} + E(C_{rep}) \quad (21)$$

where: LCC=Life Cycle Costs; C_{ins}= inspection costs; C_{rep}=repair costs and:

$$C_{ins} = \sum_{i=1}^{nins} \frac{C_{ins}(i)}{(1+r)^{i\Delta t}} \quad (22)$$

$$E(C_{rep}) = E(\sum_{j=1}^{nrep} \frac{C_{rep}(j)}{(1+r)^{t_j}}) \quad (23)$$

where $C_{ins}(i)$=1000 € ; Δt= inspection intervals; r=discount rate which in this paper conisdered 4% and t_j is the age of bridge deck in the time of repair.

Table 2. Inspection and repair strategies for limit crack width 0.3 mm.

Strategies (Cases)	Mean X_{cr}	Mean No. of Repair	E(LCC)
	(%)	#	Euro
Case 1: X=5%, InsIt=2 years	7.6	23.9	80,800
Case 2: X=10%, InsIt =2 years	12.6	14.8	70,000
Case 3: X=20%, InsIt =2years	22.4	8.5	58,500
Case 4: X=2%, InsIt =1years	6.3	28.7	97,500
Case 5: X=2%, InsIt =3years	8.9	20.8	74,700

Note: InsIt: Inspection intervals

To present the applicability of the method, according to Table 2 five inspection and repair strategies is considered. The results of the analysis for case1, case 2 and case 3 is presented in Figure 7. Case 3 with 20% acceptable extent of cracked area has lower expected life cycle cost. The acceptable threshold of repair and critical crack width have a paramount importance in decision analysis.

Again according Figure 7 case 5 in contrast to case 4 and case 1, with the same acceptable threshold of repair, yields to lower expected life cycle cost which is due to fewer inspections and later repairs which means lower present value of repair costs.

It should be noted that the repair threshold X may reach a time between two succeeding inspection periods. Apparently it will not be detected until it is inspected and consequently the deference between repaired area after inspection and repair threshold, e.g. $X_{cr} - X$, will increase. One shall consider this fact that delaying inspection despite of lowering life cycle costs increases the risk of failure. For example in Table 2 the case 5 with 3 years inspection interval in contrast to case 1 with biannual inspections has a lower expected life cycle cost but the mean repaired area is 8.9 % while case 2 with 7.6 % mean repaired area has a lesser deviation from acceptable repair threshold equal to 5%.

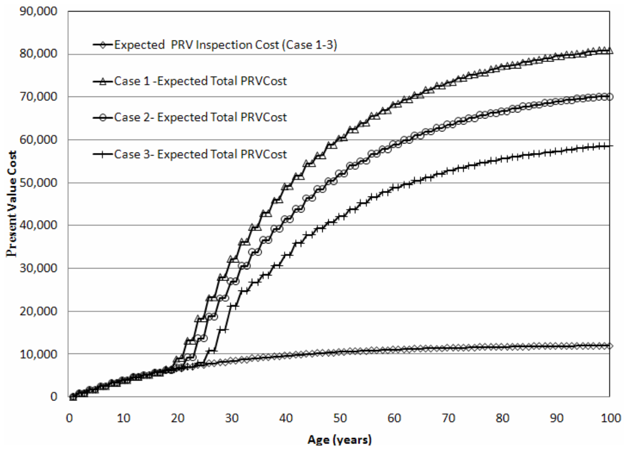

Figure 7. Effect of acceptable extent of cracked area on Life cycle cost.

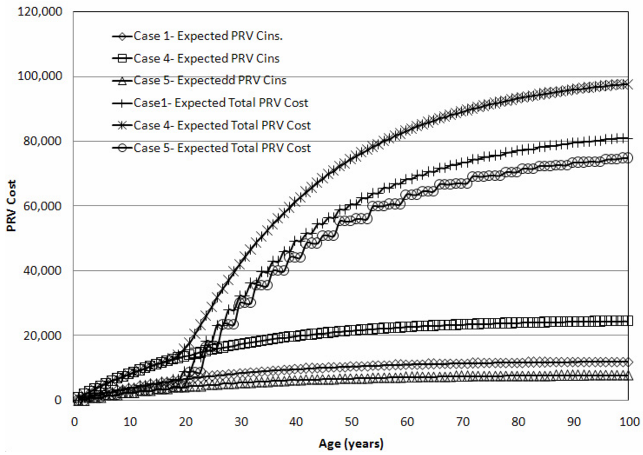

Figure 8. Effect of inspection interval period on life cycle cost.

5 CONCLUSION

The importance of considering spatial variability of material properties and environmental factors emphasized. In this regard the closed form analytical model for crack width increase with time, based on concrete fracture mechanics, was incorporated into spatial-temporal Monte Carlo analysis.

The results of such an analysis can be beneficial in determining the extent and likelihood of damage and plan for inspection and repair more realistically. It is showed that when this reliability analysis be coupled with life cycle cost analysis a more solid and informed decision analysis can be accomplished.

The shortcoming of time-consuming event-based Monte Carlo analysis for maintenance planning can be overcome with the aid of artificial intelligence, e.g. ANN and GA. This is a demanding filed of research.

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