

**EFFECT OF CUMULATIVE SEISMIC DAMAGE AND  
CORROSION ON LIFE- CYCLE COST OF REINFORCED  
CONCRETE BRIDGES**

A Thesis

by

RAMESH KUMAR

Submitted to the Office of Graduate Studies of  
Texas A&M University  
in partial fulfillment of the requirements for the degree of

MASTER OF SCIENCE

December 2007

Major Subject: Civil Engineering

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Chair of Committee,	Paolo Gardoni
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## ABSTRACT

Effect of Cumulative Seismic Damage and Corrosion on Life-Cycle Cost of Reinforced Concrete Bridges. ( December 2007)

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Chair of Advisory Committee: Dr. Paolo Gardoni

Bridge design should take into account not only safety and functionality, but also the cost effectiveness of investments throughout a bridge life-cycle. This work presents a probabilistic approach to compute the life-cycle cost (*LCC*) of corroding reinforced concrete (RC) bridges in earthquake prone regions. The approach is developed by combining cumulative seismic damage and damage associated to corrosion due to environmental conditions. Cumulative seismic damage is obtained from a low-cycle fatigue analysis. Chloride-induced corrosion of steel reinforcement is computed based on Fick's second law of diffusion.

The proposed methodology accounts for the uncertainties in the ground motion parameters, the distance from source, the seismic demand on the bridge, and the corrosion initiation time. The statistics of the accumulated damage and the cost of repairs throughout the bridge life-cycle are obtained by Monte-Carlo simulation. As an illustration of the proposed approach, the effect of design parameters on the life-cycle cost of an example RC bridge is studied. The results are shown to be valuable in better estimating the condition of existing bridges (i.e., total accumulated damage at any given time) and, therefore, can help schedule inspection and maintenance programs. In

addition, by taking into consideration the deterioration process over a bridge life-cycle, it is possible to make an estimate of the optimum design parameters by minimizing, for example, the expected cost throughout the life of the structure.

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## 1. INTRODUCTION

Bridges experience various damages and deteriorations during their service life. Therefore, they need regular inspections, maintenance and repairs to ensure that they perform above a minimum performance level at all times. A large amount of funds are required today for the repairs and upgrade of deficient bridges. For example, according to USA TODAY (2006), the Federal Highway Administration puts the current cost of upgrading bridges at \$63 billion. This situation makes optimum fund allocation and life-cycle cost analysis a priority for bridge management systems and resource allocation.

The problem of corrosion of steel reinforcement has been discussed widely in life-cycle cost analysis (e.g. Stewart and Val 2003, Li 2003, Kong and Frangopol 2004, Val 2005) but the change in failure probability over a period of time due to cumulative seismic damage has not been addressed in as much detail. The objective of this work is to present a methodology to include the effect of cumulative seismic damage and corrosion of RC bridges in the life-cycle cost analysis. A variation of the low-cycle fatigue theory (Kunnath et al. 1997) that takes into consideration the deterioration in concrete and steel is used in this work. To account for corrosion, the proposed model uses a probabilistic seismic demand model for corroded bridges developed by Choe et al. (2007).

The methodology is developed for a single-column bridge idealized as a single degree of freedom (SDOF) system. The formulation of cumulative seismic damage for bridges with multiple columns can be built on the proposed approach but is beyond the

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This thesis follows the style of *Journal of Bridge Engineering, ASCE*.

scope of this work. As a practical illustration, the proposed formulation is used to assess the *LCC* of an example bridge.

This thesis is divided into six sections. The second section presents how the proposed approach accounts for the rate of occurrence of earthquakes and their magnitudes, and the structural demand parameters. The third section discusses how the failure probability of RC bridges is computed accounting for the cumulative seismic damage. In the fourth section, the proposed approach is extended to take into account corrosion. The fifth section of the thesis presents the methodology to compute the life-cycle cost (*LCC*) of a bridge. Finally, the sixth section presents the conclusions.

## 2. SEISMICITY AND STRUCTURAL DEMAND

Structural life-cycle cost (*LCC*) analysis requires first to estimate the seismic characteristics of a region (e.g., earthquake rate of occurrence, and earthquake sources). This section presents the probabilistic model used in the proposed methodology to simulate the occurrence and the magnitude of earthquakes. In addition, this section describes the computation of structural demand parameters like drift, seismic energy, and number of inelastic cycles of the response of an equivalent SDOF system.

### 2.1 Seismicity modeling and prediction of ground motion parameters

In this study, the moment magnitude  $M_w$  is used to express the intensity at the source of an earthquake. Magnitudes are sampled independently of the time of occurrence of each earthquake using a cumulative distribution function derived from frequency-magnitude relationship given by Gutenberg and Richter (1944) as

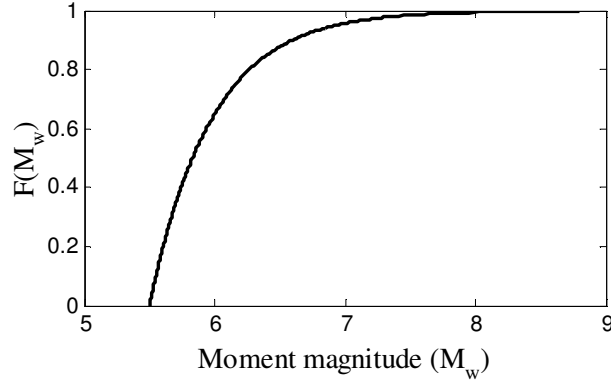
$$N_{eq}(M_w) = 10^{a-bM_w} \quad (1)$$

where,  $N_{eq}(M_w)$  is the cumulative annual rate of earthquakes having magnitudes greater than  $M_w$ , and  $a$  and  $b$  are dimensionless parameters that depend on the regional seismicity. The derived sampling distribution is

$$F(M_w) = 1 - \frac{10^{a-bM_w}}{10^{a-bM_{w,\min}}} \quad (2)$$

where,  $M_{w,\min}$  is the smallest possible magnitude of earthquakes for the given region.

Figure 1 shows the plot of  $F(M_w)$  for  $a=4.56$  and  $b=0.91$ . These values are reported by US Geological Survey (1999) for the San Francisco Bay Area.



**Figure 11** Cumulative density function for  $a=4.56$  and  $b=0.91$

The occurrence of earthquakes is modeled as a Poisson's process with a mean rate appropriate for the region. The Poisson distribution is written as

$$f(x) = \frac{(\nu)^x}{x!} \exp(-\nu) \quad (3)$$

where,  $x$  is the number of occurrences in the time window  $T_H$  which is the time span over which  $LCC$  is computed,  $\nu$  is the mean number of earthquake occurrences in  $T_H$  and  $f(x)$  is the probability density function (PDF) of  $x$ . In a Poisson's process the time intervals between two occurrences follow an exponential distribution. Therefore, the time of occurrences of the  $(M + 1)^{th}$  earthquake is simulated as follows:

$$t_{M+1} = t_M + \Delta t \quad M = 1, 2, 3... \quad (4)$$

where,  $t_M$  is the time of occurrence of the  $M^{th}$  earthquake and  $\Delta t$  is the time interval between two earthquakes simulated using the following PDF

$$f(\Delta t) = \left( \frac{v}{T_H} \right) e^{-\left( \frac{v\Delta t}{T_H} \right)} \quad (5)$$

The peak ground acceleration  $A_H$  and peak ground velocity  $V_H$  at the bridge site are computed using the ground motion attenuation relationships given by Campbell (1997).

The attenuation relationship for  $A_H$  is written as

$$\begin{aligned} \ln(A_H) = & -3.152 + 0.904M_w - 1.328 \ln \sqrt{R_{SEIS}^2 + [0.149 \exp(0.647)]^2} \\ & + [1.125 - 0.112 \ln(R_{SEIS}) - 0.957M_w] F \\ & + [0.440 - 0.171 \ln(R_{SEIS})] S_{SR} \\ & + [0.405 - 0.222 \ln(R_{SEIS})] S_{HR} + \varepsilon_A \end{aligned} \quad (6)$$

where,  $R_{SEIS}$  is the distance of the source from the site of the bridge,  $F$  is the index variable for the style of faulting,  $S_{SR}$  and  $S_{HR}$  are the index variables for local site conditions. The term  $\varepsilon_A$  is the model error that is modeled as a random variable with the Normal distribution with mean of zero and standard deviation given by Eq. (7)

$$\sigma_A = \begin{cases} 0.889 - 0.0691M_w & M_w < 7.4 \\ 0.38 & M_w \geq 7.4 \end{cases} \quad (7)$$

The attenuation relationship for  $V_H$  is given by

$$\begin{aligned} \ln(V_H) = & \ln(A_H) + 0.26 + 0.29M_w - 1.44 \ln [R_{SEIS} + 0.0203 \exp(0.958M_w)] \\ & + 1.89 \ln [R_{SEIS} + 0.361 \exp(0.576M_w)] \\ & + (0.0001 - 0.000565M_w) R_{SEIS} - 0.12F - 0.15S_{SR} \\ & - 0.30S_{SR} + 0.75 \tanh(0.51D)(1 - S_{HR}) + f_v(D) + \varepsilon_V \end{aligned} \quad (8)$$

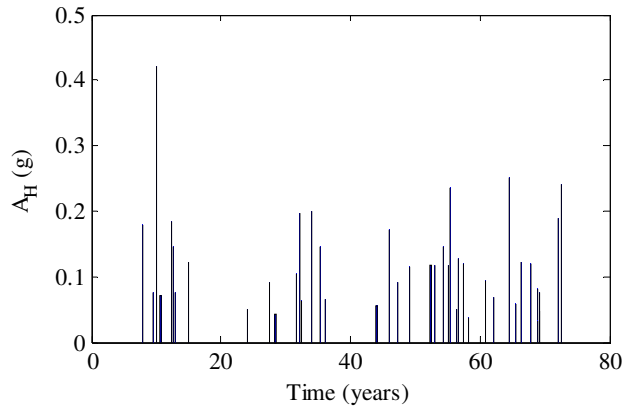
where  $\varepsilon_V$  is the model error again modeled as a random variable with mean of zero and standard deviation given by Eq. (9)

$$\sigma_v = \sqrt{\sigma_A^2 + 0.06^2} \quad (9)$$

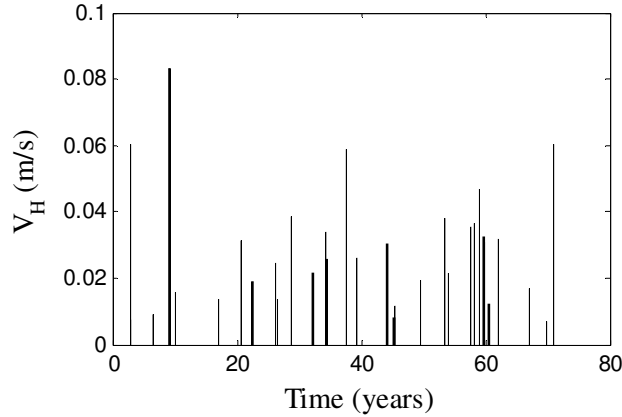
The function  $f_v(D)$  in Eq. (8) is given by Eq. (10)

$$f_v(D) = \begin{cases} 0 & D \geq 1 \text{ km} \\ -0.30(1 - S_{HR})(1 - D) - 0.15(1 - D)S_{SR} & D < 1 \text{ km} \end{cases} \quad (10)$$

where,  $D$  is depth to the base rock from ground surface at the bridge site. Figures 2 and 3 show one possible realization for ground motion parameters for a life time of the bridge for the random variables shown in Table 2.



**Figure 2** A realization of  $A_H$  values for  $T_H=75$  years



**Figure 3** A realization of  $V_H$  values for  $T_H=75$  years

**Table 1.** Statistical parameters for seismicity modeling

Variable	Distribution	Parameters	Range
$M_w$	See Eq.(3)	$a=4.56$ $b=0.91$	$M_w \geq 5.5$
$R_{SEIS}$	Uniform		$20km \leq R_{SEIS} \leq 100$
$D$	Uniform		$3km \leq D \leq 6km$
$F$	Bernoulli	$p=0.5$	$F=0,1$
$x$	Poisson	$\nu = 150$ $T_H = 75 yrs$	$x \geq 0$

## 2.2 Spectral acceleration and seismic energy demand

Two quantities needed in the proposed formulation are the spectral acceleration,  $S_a$ , and the seismic energy demand,  $E_l$ , for each simulated earthquake. The spectral acceleration  $S_a$  is computed by scaling  $A_H$  as explained by Kunnath and Chai (2004). This formulation to compute elastic response spectrum, originally proposed by Vidic et al. (1994) and later modified by Chai et al. (2000), can be written as

$$S_a = \Omega_a A_H \quad (11)$$

where,  $\Omega_a$  is a scaling factor defined as

$$\Omega_a = \begin{cases} 1 + 2.5(C_a - 1) \frac{T}{T_c} & 0 < T \leq 0.4T_c \\ C_a & 0.4T_c < T \leq T_c \\ 2\pi C_v \frac{V_H}{A_H T} & T < 0.4T_c \end{cases} \quad (12)$$

where,  $T$  is the fundamental period of the bridge (or the equivalent SDOF system),  $T_c$  is the characteristic period of ground motion,  $C_a$  is the ratio of elastic spectral acceleration to peak ground acceleration in the short period range, and  $C_v$  is the ratio of spectral velocity to peak ground velocity in the velocity controlled region of the response spectrum. The values suggested by Chai et al. (2000) for  $C_a$  and  $C_v$  are 2.5 and 2.0, respectively. The value of  $T_c$  is given by Eq. (13)

$$T_c = 2\pi \frac{C_v V_H}{C_a A_H} \quad (13)$$

The seismic energy demand  $E_I$  is defined by Kunnath and Chai (2004) as

$$E_I = \frac{1}{2} m v_e^2 \quad (14)$$

where,  $v_e$  is the equivalent input energy velocity given by Eq. (15)

$$v_e = \Omega_v V_H \quad (15)$$

where,  $\Omega_v$  is a velocity amplification factor defined as



$$\Omega_v = \begin{cases} \Omega_v^* \left[ \frac{2T}{T_c} - \left( \frac{T}{T_c} \right)^2 \right] & T \leq T_c \\ \Omega_v^* \left[ \frac{T}{T_c} \right]^{-\lambda} & T > T_c \end{cases} \quad (16)$$

where,  $\Omega_v^*$  is an energy amplification factor given as

$$\Omega_v^* = \frac{0.25A_H}{V_H} \sqrt{t_d T_c} \sqrt{\frac{\lambda + 0.5}{2\lambda + 2}} \quad (17)$$

where,  $\lambda$  is an input energy spectrum parameter and is equal to 0.5 as suggested by Kunnath and Chai (2004) and  $t_d$  is the strong ground motion duration. The value of  $t_d$  is given by Trifunac and Brady (1975) as follows:

$$t_d = -4.88s + 2.33M_w + 0.149R_{SEIS} \quad (18)$$

where  $s$  is a geologic site parameter and is equal to 0.0, 1.0 and 2.0 for alluvium, intermediate and rock, respectively. The type of soil used for the numerical example shown later in this thesis is alluvium.

### 2.3 Peak displacement demand

The quantities  $S_a$  and  $T$  are used to compute the peak displacement demand  $U_{max}$  based on the probabilistic demand model developed by Gardoni et al. (2003) as

$$\ln\left(\frac{U_{max}}{H}\right) = 0.61 + 3.90\theta_{\delta_2} + (1 + \theta_{\delta_2})\hat{d}_{\delta} + \sigma_{\delta}\epsilon_{\delta} \quad (19)$$

where,  $H$  is the clear height of the column,  $\theta_{\delta_2}$  is a model parameter equal to  $-0.153$  and  $\sigma_{\delta} = 0.216$ . The variable  $\hat{d}_{\delta}$  is the natural logarithm of the deterministic drift demand computed using a deterministic procedure originally proposed by Chopra and

Goel (1999) for the case of buildings and later modified by Gardoni et al. (2003) for the case of bridges, and  $\varepsilon_\delta$  is a random variable that has the standard normal distribution.

## 2.4 Number of inelastic cycles

For a given earthquake response, the equivalent number of constant amplitude inelastic cycles  $N$  corresponding to a certain amplitude is needed to compute the seismic damage. The value of  $N$  corresponding to  $U_{\max}$  is obtained from cyclic demand spectrum (Kunnath and Chai, 2004) as

$$N = \frac{\alpha E_l}{4\alpha_h V_y U_{\max}} \quad (20)$$

where  $V_y$  is the lateral force at yield and the parameter  $\alpha$  is the ratio of hysteretic energy to seismic energy demand,  $E_l$  that can be written as

$$\alpha = 1.13 \frac{(\mu - 1)^{0.82}}{\mu} \quad (21)$$

where,  $\mu$  is the ductility demand given by

$$\mu = \frac{U_{\max}}{U_y} \quad (22)$$

where  $U_y$  is the displacement at yield of the column top. The parameter  $\alpha_h$  is a coefficient suggested by Kunnath and Chai (2004) to account for the deterioration of stiffness due to cyclic loading and is equal to 0.5.

### 3. CUMULATIVE SEISMIC DAMAGE

Under earthquake loading, bridge columns undergo several cycles of inelastic deflections. Therefore, low-cycle fatigue analysis is used in this work to evaluate the seismic damage. In addition, an approximate strength degradation equation suggested by Das et al. (2006) is used to compute the structural properties of the damaged structure. This section first presents the background and the method adopted to model the low-cycle fatigue. Then, the computation of damage index  $DI$  is discussed. Lastly, the methodology to compute structural properties of a damaged structure is presented.

#### 3.1 Low-cycle fatigue

Based on Coffin (1954) and Manson (1953), the Coffin–Manson theory of fatigue formulates the behavior of longitudinal bars under reversed cyclic loading as

$$\varepsilon_p = \varepsilon'_f (2N_f)^c \quad (23)$$

where,  $\varepsilon_p$  is the plastic strain amplitude,  $\varepsilon'_f$  and  $c$  are material constants determined experimentally,  $2N_f$  is the number of half cycles for the first fatigue crack on the longitudinal reinforcement bar. Mander et al. (1994) obtained the following expression for  $\varepsilon_p$  based on experiments on reinforcement bars:

$$\varepsilon_p = 0.08(2N_f)^{-0.5} \quad (24)$$

Similarly, Kunnath et al. (1997) obtained the following expression:

$$\varepsilon_p = 0.065(2N_f)^{-0.436} \quad (25)$$

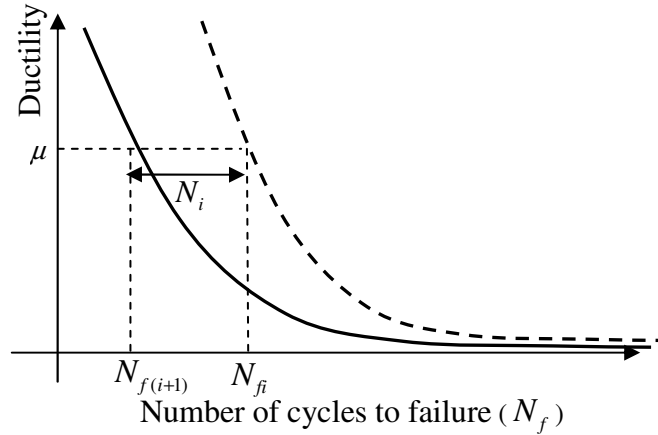
Tsuno and Park (2004) carried out an experimental work and reviewed the damage models developed by Mander et al. (1994) and Kunnath et al. (1997). Tsuno and Park tested five RC columns with different loading patterns and compared the observed damage with the predicted damage. It was observed that Kunnath's model predicts failure well for RC columns that are seismically designed according to CALTRANS or AASHTO to have a dominant flexural failure mode. Kunnath's model was based on experiments on RC columns and thus accounts for damage in columns as a composite of steel and concrete. Mander's model was based on experiments on steel reinforcement and accounted for fatigue in the steel only. However, one disadvantage of Kunnath's model was that it underestimated the damage in extreme loading cases having large displacements in the first cycle. Mander's model was found to be more accurate in such extreme loadings than Kunnath's model. In this study, instead of the relation between  $\varepsilon_p$  and  $N_f$  (i.e., Eqs. (23) through (25)) a modification of the relation suggested by Kunnath and Chai (2004) is used to model the low-cycle fatigue behavior for circular ductile RC columns as follows:

$$N_f = \left( \frac{8.25}{\mu} \right)^4 \quad (26)$$

where,  $N_f$  is number of cycles to failure corresponding to the ductility demand  $\mu$ . The above expression can be used only for the first earthquake and has to be modified for the future earthquakes because the column deteriorates with every passing earthquake. A variation of Eq. (26) is proposed to make it suitable for damaged columns as follows

$$N_{fM} = \left( \frac{8.25}{\mu} \right)^4 - \sum_{i=1}^{M-1} N_i \quad M = 2, 3, 4 \dots \quad (27)$$

where,  $N_{fM}$  is the number of cycles to failure for the  $M^{th}$  earthquake and  $N_i$  is the number of cycles in the  $i^{th}$  earthquake that preceded. The expression in Eq. (27) can be explained using Figure 4. If  $N_i$  is the number of cycles used up in the  $i^{th}$  event, then  $N_i$  has to be subtracted from the column capacity for the  $i^{th}$  event to obtain the deteriorated capacity for the  $(i+1)^{th}$  event. Thus, the fatigue curve after the  $i^{th}$  event is translated by the amount  $N_i$  towards the left.



**Figure 4.** Updating fatigue curve

### 3.2 Cumulative Damage Index, $DI$

Using the well known Miner's rule explained in Miner (1945), the cumulative damage index can be written as follows:

$$DI = \sum_{j=1}^m \frac{1}{2N_{fj}} \quad (28)$$

where,  $DI$  is cumulative damage index after  $m$  half-cycles,  $N_{fj}$  is the number of cycles to failure corresponding to the displacement in the  $j^{\text{th}}$  half-cycle of loading. Eq. (28) can be modified to compute the cumulative seismic damage index after the  $M^{\text{th}}$  earthquake as follows:

$$DI_M = \frac{N_M}{N_{fM}} + DI_{M-1} \quad M = 2, 3, 4, \dots \quad (29)$$

where,  $N_M$  is the equivalent number of constant amplitude inelastic cycles in the  $M^{\text{th}}$  earthquake computed using Eq. (20) and  $N_{fM}$  is the number of cycles to failure for the peak displacement of  $M^{\text{th}}$  earthquake computed using Eq. (26). Theoretically a column should collapse when  $DI$  is equal to 1.0 but the experimental results in Kunnath et al. (1997) shows that ductile columns, typically designed to fail in flexure, collapse when the value of  $DI$  that exceed 0.6.

### 3.3 Structural properties of damaged structure

The structural properties  $\mathbf{x}_p$  of the pristine bridge are defined as follows:

$$\mathbf{x}_p = (k, T, U_y, V_y) \quad (30)$$

where,  $k$  is the lateral column stiffness. The vector  $\mathbf{x}_p$  in Eq. (30) is represented by  $\mathbf{x}_{pM}^-$  right before the  $M^{th}$  earthquake and by  $\mathbf{x}_{pM}^+$  right after the  $M^{th}$  earthquake.

$$\mathbf{x}_{pM}^- = (k_M^-, T_M^-, U_{yM}^-, V_{yM}^-) \quad M = 1, 2, 3, \dots \quad (31)$$

$$\mathbf{x}_{pM}^+ = (k_M^+, T_M^+, U_{yM}^+, V_{yM}^+) \quad M = 1, 2, 3, \dots \quad (32)$$

Das et al. (2006) suggested Eqs. (33) and (34) to account for any change in the fundamental period and displacement at yield due to an earthquake. These equations suggest that the earthquake loading decreases the column stiffness and increases the displacement at yield as follows:

$$k_M^+ = k_M^- \left[ 1 - \frac{U_{\max M} - U_{yM}^+}{U_u - U_{yM}^-} \right]^{0.1} \quad M = 1, 2, 3, \dots \quad (33)$$

where,  $k_M^-$  and  $k_M^+$  are column stiffness right before and after the  $M^{th}$  earthquake. The quantity  $U_u$  is the maximum displacement under monotonic loading of the pristine column. The yield displacement  $U_{yM}^+$  after the  $M^{th}$  earthquake is given by Eq. (34)

$$U_{yM}^+ = \left[ \frac{k + k_M^-}{k + k_M^+} \right] U_{yM}^- \quad M = 1, 2, 3, \dots \quad (34)$$

where,  $k$  is the pristine column stiffness. The values of  $T_M^+$  and  $V_{yM}^+$  can be found from following equations:

$$T_M^+ = 2\pi \sqrt{\frac{m}{k_M^+}} \quad M = 1, 2, 3, \dots \quad (35)$$

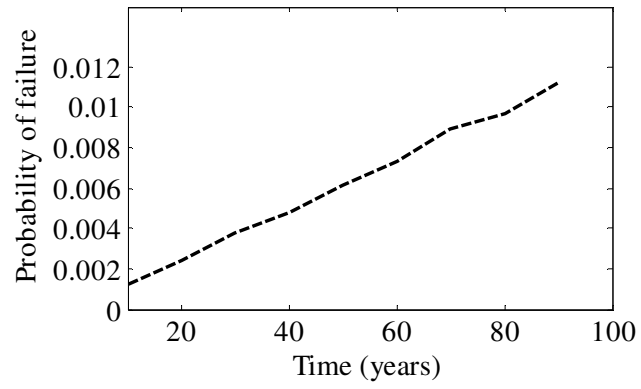
$$V_{yM}^+ = k_M^+ U_{yM}^+ \quad M = 1, 2, 3, \dots \quad (36)$$

A basic Monte-Carlo simulation using the random variables listed in Table 1 and the structural parameters listed in Table 2 is performed to compute the failure probabilities at various time instances during the service life. Based on the experimental observations of Kunnath et al. (1997), failure was assumed to occur when  $DI \geq 0.6$ . As expected (see Figure 5) it is found that the failure probability increases with the age of the bridge due to the damage accumulated during past earthquakes.

**Table 2.** Structural properties of the example bridge

Quantity	Symbol	Value
Axial load on column	$P (N)$	$0.1A_g f_c$
Height of column	$H (mm)$	4000
Diameter of column	$D (mm)$	1500
Area of longitudinal bars	(%)	2.0
Diameter of transverse reinf.	$d_s (mm)$	9.5
Spacing of transverse reinf.	$s_v (mm)$	60
Nominal strength of concrete	$f'_c (MPa)$	36
Nominal yield strength of steel	$f_y (MPa)$	475





**Figure 5.** Effect of Cumulative Seismic Damage on Failure Probability.  
Numerical error of simulation =2 %

## 4. EFFECT OF CORROSION

Corrosion reduces the steel reinforcement area which in turn increases the vulnerability of a bridge. This section presents the methodology to predict the corrosion initiation following Choe et al (2007) and the computation of structural properties of the corroded structure.

### 4.1 Corrosion initiation

Corrosion is initiated in the steel reinforcement when chloride concentration exceeds a critical value  $C_{cr}$ . The corrosion initiation time  $T_{corr}$  is given by Dura-Crete (2000) as follows:

$$T_{corr} = X_I \cdot \left[ \frac{d_c^2}{4k_e k_t k_c D_0 (t_0)^n} \left[ erf^{-1} \left( 1 - \frac{C_{cr}}{C_s} \right) \right]^{-2} \right]^{\frac{1}{1-n}} \quad (37)$$

where,  $X_I$  is a model uncertainty coefficient to account for the idealization implied by Fick's second law,  $d_c$  is the clear cover,  $k_e$  is an environmental factor,  $k_t$  is an influence factor for test methods to determine the empirical diffusion coefficient  $D_0$ ,  $k_c$  is an influence factor for curing,  $t_0$  is the reference period for  $D_0$ ,  $n$  is the age factor,  $C_{cr}$  is the critical chloride concentration,  $C_s$  is the chloride concentration on the surface, and  $erf(\cdot)$  is the error function. Dura-Crete (2000) provides the statistics (distribution, mean, and standard deviation) of the random variables in Eq. (37) accounting for different material and environmental factors. For completeness, these statistics are also provided in Appendix 1.

## 4.2 Structural properties of corroded structure

The reduction in steel reinforcement area due to corrosion is given by Choe et al (2007) based on the work Vu and Stewart (2000) as follows:

$$d_b(t|T_{corr}) = \begin{cases} d_{b0} & t \leq T_{corr} \\ d_{b0} - \frac{1.0508(1-w/c)^{-1.64}}{d} (t - T_{corr})^{0.71} & T_{corr} < t \leq T_f \\ 0 & t > T_f \end{cases} \quad (38)$$

$$T_f = T_{corr} + d_{b0} \left\{ \frac{d_c}{1.0508(1-w/c)^{-1.64}} \right\}^{\frac{1}{0.71}} \quad (39)$$

Here the reinforcement bar diameter  $d_b$  is expressed as a function of time  $t$ , the corrosion initiation time  $T_{corr}$ , the initial diameter  $d_{b0}$ , the water to binder ratio  $w/c$ , and the time  $T_f$  when, in theory,  $d_b$  reaches zero. The value of  $d_b$  at time  $t$  is computed using the value of  $T_{corr}$  which in turn is simulated using Eq. (37).

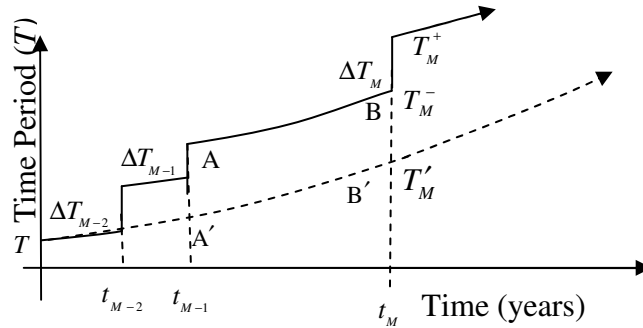
For each realization of life span in Monte-Carlo simulation one value of  $T_{corr}$  (Eq. (37)) is simulated along with a set of values of ground motion parameters and time of occurrences  $t_M$  to completely represent a possible scenario of the seismic and corrosive environment. While the ground motion parameters are used to compute seismic damage,  $T_{corr}$  is used for evaluating  $d_b$  at time  $t_M$  to compute the structural properties of corroded structure.

The vectors  $\mathbf{x}_{pM}^-$  and  $\mathbf{x}_{pM-1}^+$  in Eqs. (31) and (32) are equal when corrosion is not initiated (i.e.,  $t < T_{corr}$ ). After the initiation of corrosion,  $\mathbf{x}_{pM}^-$  is computed as follows:

$$T_M^- = T_M' + \sum_{i=1}^{M-1} \Delta T_i \quad M = 2, 3, 4, \dots \quad (40)$$

$$\Delta T_i = T_i^+ - T_i^- \quad (41)$$

where,  $T_M'$  is the fundamental period at time  $t_M$  (the time of occurrence of the  $M^{\text{th}}$  earthquake) of the structure deteriorated due to the corrosion only. As illustrated in Figure 6,  $T_M'$  does not include the effect of the seismic damage. The value of  $T_M'$  can be found by computing column stiffness from a moment curvature analysis using the reduced reinforcement area at time  $t_M$  obtained from Eq. (38).



**Figure 6.** Deterioration in fundamental period due to earthquakes and corrosion

In Figure 6, the curve  $A'B'$  represents the deterioration in the fundamental period due to corrosion only. The curve  $AB$  takes into account the deterioration due to both the seismic damage and the corrosion. It is assumed that  $AB$  is parallel to  $A'B'$  at a distance equal to the sum of the  $\Delta T_i$  due to each past earthquake. This assumption implies that the corrosion rate is independent of the seismic damage, which is reasonable because,

though localized corrosion might be accelerated near the cracks caused by earthquakes, the rate of the uniform corrosion in the column is unaffected. Similarly, the displacement at yield  $U_{yM}^-$  can be written as:

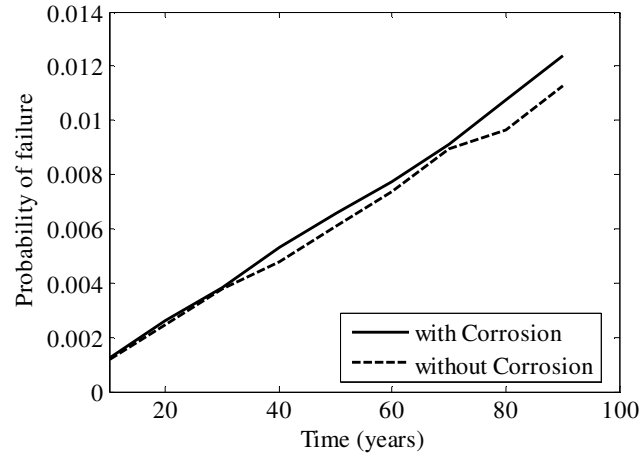
$$U_{yM}^+ = U'_{yM} + \sum_{N=1}^{M-1} \Delta U_{yN} \quad M = 2, 3, 4 \dots \quad (42)$$

$$\Delta U_{yN} = U_{yN}^+ - U_{yN}^- \quad (43)$$

where,  $U'_{yM}$  is the yield displacement at time  $t_M$  due to the corrosion only. Figure 7 compares the contributions of cumulative seismic damage and corrosion in the deterioration of the bridge. The environmental conditions used in calculations are given in Table 3. It can be noticed that the contribution to the failure probability of the corrosion is small compared to that of the cumulative seismic damage.

**Table 3.** Statistical parameters for corrosion modeling

<b>Variable</b>	<b>Environment/Condition</b>
w/c	0.5
$t_0$	28 days
$k_e$	Atmospheric
$C_{cs}$	Atmospheric
$C_{cr}$	Constantly humid



**Figure 7.** Effect cumulative seismic damage and corrosion on failure probability.  
Numerical error of simulation =2 %

## 5. LIFE-CYCLE COST ANALYSIS

The life-cycle cost ( $LCC$ ) of a bridge can be expressed mathematically as follows:

$$LCC = C_C + NPV(C_{IN}) + NPV(C_M) + NPV(C_F) \quad (44)$$

where,  $C_C$  = initial construction cost,  $C_{IN}$  = cost of inspections,  $C_M$  = routine maintenance costs,  $C_F$  = failure costs. The inspections, failures and maintenances occur at different instances in time, thus it is necessary to transform all the costs to the net present value (NPV). This thesis focuses only on the computation of  $C_F$ , accounting for the effects of earthquakes and corrosion. The bridge is allowed to deteriorate until collapse (i.e.,  $DI \geq 0.6$ ) without receiving any maintenance. The entire bridge deck and column is replaced at collapse and the bridge is assumed to regain full strength after replacement. Thus the values of  $C_{IN}$  and  $C_M$  are taken equal to zero. The cost of failure is the sum of the financial losses ( $C_L$ ) due to the failure and the cost of repair ( $C_R$ ) to regain the lost performance level (i.e.,  $C_F = C_L + C_C$ ). In the case of collapse, it is assumed that the cost of repair is equal to the cost of reconstruction (i.e.,  $C_R = C_C$ ). Stewart and Val (2003) assumed the cost of failure due to collapse to be ten times the construction cost (i.e.  $C_L + C_C = 10 \times C_C$ ). Following the works of Kong and Frangopol (2003) and Stewart and Val (2003) the NPV of  $C_F$  is given by

$$NPV(C_F) = \frac{C_F}{(1+r)^{t_F}} \quad (45)$$

where,  $r$  is the discount rate and  $t_F$  is the time of failure. Substituting the values of  $NPV(C_F)$  in Eq. (44) the following is obtained

$$LCC = C_C + \sum_{i=1}^{n_F} \frac{10C_C}{(1+r)^{t_F}} \quad (46)$$

where,  $n_F$  is the number of failures in the time window  $T_H$ .

The total cost of the bridge construction  $C_C$  consists of the construction cost of the piers  $C_{\text{pier}}$ , the deck slab  $C_{\text{deck}}$ , and the piles  $C_{\text{pile}}$ . Therefore, the  $C_C$  can be written as

$$C_C = C_{\text{pier}} + C_{\text{deck}} + C_{\text{pile}} \quad (47)$$

The value of  $C_{\text{pier}}$  is computed as

$$C_{\text{pier}} = \frac{\pi d^2 H C_{uc}}{4} + \frac{\pi \left[ \rho_l d^2 + \rho_s (D - 2d_c)^2 \right] H \gamma_s C_{us}}{4} \quad (48)$$

where,  $C_{uc}$  is the cost per unit volume of the concrete work in column,  $C_{us}$  is the cost per unit weight of steel reinforcement work,  $\gamma_s$  is weight density of steel,  $\rho_l$  is area ratio of longitudinal steel to gross column area, and  $\rho_s$  is volumetric ratio of shear reinforcement to the column core. For illustration of the relation between  $LCC$  and design parameters, the  $LCC$  analysis is carried out by varying the amount of the  $\rho_l$  and  $D$ . The column strength parameters can also be varied by changing the grades of steel and concrete. But for the convenience and economy in construction usually these choices are limited.

The values of  $C_{\text{deck}}$  and  $C_{\text{pile}}$  are independent of  $\rho_l$  and  $D$  and are thus assumed to be constant in the  $LCC$  analysis. They are computed using the unit construction costs provided by CALTRANS Contract Cost Data (2006). The total deck area is assumed to

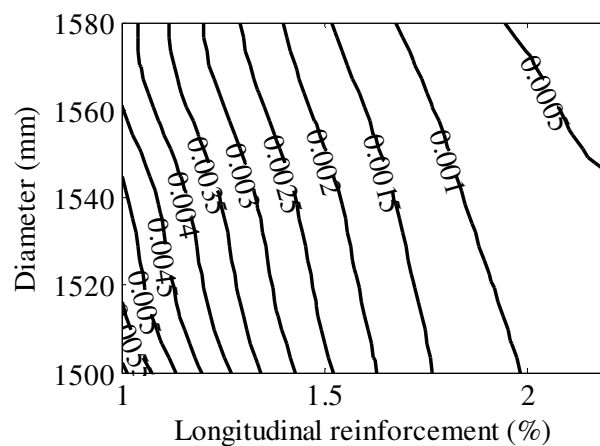


be 40 m by 10 m and pile depth is assumed to be 15 m. Table 4 provides the values of the parameters used in the computation of  $C_c$ .

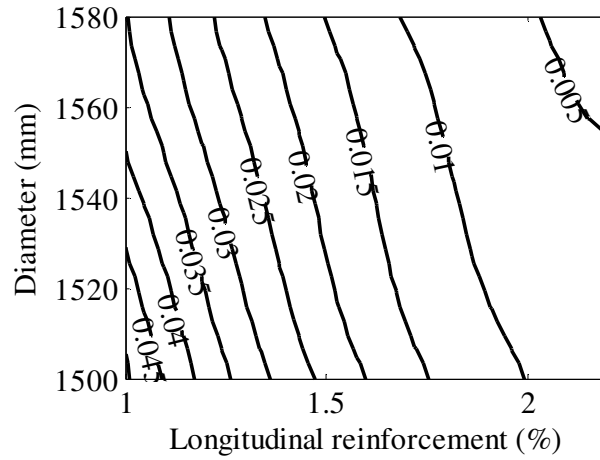
Monte-Carlo simulations are used to compute the expected  $LCC$ . Figures 8 and 9 show the contour plots for the failure probabilities for different values of chosen design parameters at  $t=0$  and  $t=75$  years, respectively.

**Table 4.** Parameters used in computing life-cycle cost

Item	Cost
Steel work	\$.2.25/kg
Concrete work	\$800/m <sup>3</sup>
$C_{deck}$	\$450/m <sup>2</sup>
$C_{pile}$	\$250/m
Length of bridge	#2, 20m spans
Width of bridge	10m
Pile depth	15m
$C_L$	$10 \times C_c$
$r$	7%

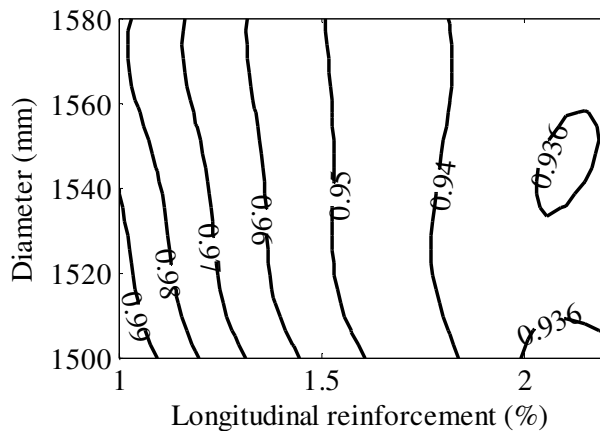


**Figure 8.** Contour plots of failure probability at  $T=0$  years



**Figure 9.** Contour plots of failure probability at T=75 years

The failure probability as expected is found to increase with time due to accumulated seismic damage and corrosion. Figure 10 shows the contour plot of the normalized expected *LCC* for different values of the chosen design parameters.



**Figure 10.** Contour plots of normalized *LCC*

The normalized costs are obtained by dividing the expected *LCC* of the bridge by a baseline expected *LCC* that correspond to and  $\rho_l = 1.0\%$  and  $D = 1,500$  mm. The minimum expected *LCC* is about 6% lower than the baseline *LCC*. The minimum is obtained by increasing  $\rho_l$  from 1.0% to 2.0% of gross column area and  $D$  by 3% (from 1,500 mm to 1,545 mm).

## 6. CONCLUSIONS

A methodology is presented to include the effect of cumulative seismic damage in life-cycle cost analysis of bridges. The methodology is then extended to account also for the structural deterioration due to the corrosion of steel reinforcement. The uncertainties in the ground motion parameters, seismic demand on the bridge and the corrosion process are accounted for in the methodology. The uncertainties in distance of source and the style of faulting can also be accounted in this methodology.

It is shown that the failure probabilities increase significantly over a bridge service-life. This is because of the damage accumulated during the repeated occurrence of small earthquakes that did not lead to failure. It is also found that, in seismic regions, the contribution of cumulative seismic damage is significantly higher than the one from corrosion. The developed methodology can be used in a life-cycle cost analysis to assess the optimal design parameters for a bridge. As a practical illustration, the proposed formulation is used to assess the *LCC* of an example bridge and find the optimal column diameter and reinforcement ratio.

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## APPENDIX

$D_o$ : Reference diffusion coefficient at  $t_0 = 28_{\text{day}}$

Condition	Distribution	Mean [ $\text{mm}^2/\text{yr}$ ]	St. dev. [ $10^{-12} \text{ m}^2/\text{s}$ ]
w/c=0.4	Normal	220.9	25.4
w/c=0.45	Normal	315.6	32.5
w/c=0.5	Normal	473	43.2

$n$ : Aging factor

Condition	Distribution	Mean	St. dev.	A	B
All	Beta	0.362	0.245	0	0.98

$k_e$ : Environmental correction factor

Condition	Distribution	Mean	St. dev.
Submerged	Gamma	0.325	0.223
Tidal	Gamma	0.924	0.155
Splash	Gamma	0.265	0.045
Atmospheric	Gamma	0.676	0.114

$k_c$ : Curing time correction factor

Condition	Distribution	Mean	St. dev.	A	B
curing 1day	Beta	2.4	0.7	1.0	4.0
curing 3day	Beta	1.5	0.3	1.0	4.0
curing 7day	Deterministic	1.0			
curing 28day	Beta	0.8	0.1	0.4	1.0

$k_t$ : correction factor for tests

Condition	Distribution	Mean	St. dev.
All	Normal	0.832	0.024

$X_f$ : modeling uncertainty

Condition	Distribution	Mean	St. dev.
All	Lognormal	1	0.05

$C_{cs}$ : chloride surface concentration (linear function of  $A_{cs}$  and  $\varepsilon_{cs}$ , % by weight of binder)

Condition	Distribution	$A_{cs}$		$\varepsilon_{cs}$	
		Mean	St. dev.	Mean	St. dev.
Submerged	Normal	10.348	0.714	0	0.58
Tidal	Normal	7.758	1.36	0	1.105
Splash	Normal	7.758	1.36	0	1.105
Atmospheric	Normal	2.565	0.356	0	0.405



$C_{cr}$ : critical chloride content (mass-% of binder)

	w/c ratio	Distribution	Mean	St. dev.
Constantly saturated	0.30	Normal	2.30	0.20
	0.40	Normal	2.10	0.20
	0.50	Normal	1.60	0.20
Constantly humid or many humid-dry cycles	0.30	Normal	0.50	0.10
	0.40	Normal	0.80	0.10
	0.50	Normal	0.90	0.15

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