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SEISMIC LIFE-CYCLE COST ANALYSIS OF AGED BRIDGES

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ABSTRACT

The deteriorating health of current bridge infrastructure directly indicates the potential risks of structural damage and subsequent economic consequences not only under operational but also extreme loading conditions, such as seismic events. In the past decade, there has been an increasing interest in the life cycle cost (LCC) estimation of infrastructure, for maintenance and rehabilitation planning and prioritization of investments. Incorporating the expected value of economic losses due to lifetime exposure to natural hazards can be a critical factor to consider in the life cycle analysis (termed "seismic life cycle cost" in this work). Although previous studies have evaluated the probability of bridge damage in their as-built condition, reliable estimates of seismic life cycle cost depend upon the time-dependent seismic vulnerability of aging bridges. The present study focuses on evaluation of the fragility of aging bridges due to the time-dependent deterioration of multiple bridge components from corrosion, for example reinforcement reduction of reinforced concrete (RC) columns and degradation of bridge bearings. Subsequently, the time-dependent system fragility curves are incorporated in life cycle cost models for aging bridges. Finally, a discussion on the relative differences between the life cycle cost estimate of the as-built and aging bridge is presented for different case study parameters, including remaining life, discount factor, and exposure conditions. The findings highlight the importance of considering the effects of time evolving deteriorated structural condition on seismic vulnerability when assessing the expected seismic life cycle costs.

Introduction

Highway bridges constitute critical links in the transportation network and thereby the proper functioning of these infrastructure elements is critical to support nationwide economic and social activities. However, the detrimental effects of aging and deterioration has prompted ASCE (ASCE 2009) to label more than 26 percent of U.S. bridges as structurally deficient and functionally obsolete with regard to normal functioning and performance under extreme events like earthquakes and hurricanes. With the amount of resources restricted, there is an increasing need to identify the most critical elements in the bridge infrastructure to prioritize restoration and retrofit activities. In this regard life-cycle cost analysis of bridges has proven to be an essential tool to support decision making since it takes into consideration costs incurred from initial construction, restoration and maintenance, demolition etc. along the service life of the bridge.

In the last decade considerable progress has been made in the life-cycle cost estimation of structures by taking into account the probability of damages resulting from extreme events.

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For instance Wen and Kang (2001) provided a framework for optimal design of structures with desired reliability indices under multiple threat scenarios based on the principle of minimum life-cycle cost. Ellingwood and Wen (2005) emphasized on the implementation of risk-benefit based design decisions for structures to identify the most economically favorable design decision, particularly in regions of moderate seismicity. A fundamental assumption in the life-cycle cost formulation in the above mentioned studies is that the deterioration of structural capacity with time is ignored, or may be assumed to remain constant. However, in reality this is seldom the case in analyzing bridge vulnerability with more than one in four bridges in the U.S identified as structural degraded due to aging (ASCE 2009).

Only recently have the effects of cumulative seismic vulnerability of columns subjected to corrosion deterioration been considered in the life-cycle cost analysis for bridges (Kumar et al 2009). Although based on single component deterioration mechanism (namely, RC columns), the methodology proposed by the authors was used to evaluate the optimal design parameters for the bridge. In the present paper, a framework for life-cycle cost analysis of bridges is put forth by taking into account the deterioration of multiple bridge components, namely, RC columns and steel bridge bearings. Additionally, a new methodology is proposed to assess the life-cycle cost of the aging bridge based on the distribution of different damage states as a non-homogeneous Poisson processes along the service life of the bridge. A case study application of seismic life cycle cost estimation is conducted on a typical non-seismically designed multiple-span continuous (MSC) steel girder bridge as commonly found in Central and Eastern United States (CEUS).

Aging Bridge Life-Cycle Cost Formulation

Under the assumption of negligible degradation of structural capacity, the probability of failure of the bridge system under extreme event loading like earthquakes does not vary with time. Consequently, given an annual earthquake occurrence rate particular to the bridge site, the annual rate of exceedance of a particular damage state of the bridge can be modeled by a homogeneous Poisson process. Numerically, this can be represented as follows:

Assuming that the site specific earthquake occurrence rate is ν per year, the mean annual rate of exceedance of a particular damage state i is given by (Nuti and Vanzi 2003)

$$\lambda_i = \nu P_i \tag{1}$$

where, P_i is the annual probability of meeting or exceeding of the damage state i under consideration. Following this Poisson process assumption, the time, $t = T$, between the beginning of exposure of the bridge to earthquakes ($t = 0$), and the occurrence of first failure ($t = T$), is an exponentially distributed random variable with a cumulative density function $(F_T(t))$ given by Eq. 2 as follows (Nuti and Vanzi 2003):

$$F_T(t) = 1 - e^{-\lambda_i t} = 1 - e^{-\nu P_i t} \tag{2}$$

The probability of failure, or exceeding the limit state, in the period $[0, T]$ is equivalent to the cumulative density function from Eq. 2 and can be represented as:

$$P_T = F_T(t) = 1 - e^{-\lambda_i t} \tag{3}$$

However, taking into consideration the effects of aging and degradation, the bridge components become increasingly more vulnerable to extreme events. With the probability of failure varying (increasing) with time, the annual rate of exceedance of a particular damage state is now modeled as a non-homogeneous Poisson process given by:

$$\lambda_i(t) = \nu P_i(t) \tag{4}$$

where, $P_i(t)$ is the annual probability of failure of the bridge at time $t = t$. Subsequently, the cumulative density function or the probability of failure/exceeding a particular limit state i in the period $[0, T]$ is now represented as

$$P_T = F_T(t) = 1 - e^{-\int_0^T \nu P_i(t)dt} \tag{5}$$

Furthermore, assuming that C_i is the cost associated with damage state i to restore the bridge to its original functionality level at any time t, and k is the inflation free logarithmic interest rate, the present value (PV) of the bridge along its service life ($t = 0$ to $t = T$), for a given damage state is given by:

$$PV = \int_0^T \lambda_i(t) e^{-\int_0^T \nu P_i(t)dt} \cdot \frac{C_i}{e^{kt}} \, dt \tag{6}$$

Over all damage states considered [$i = 1$ to n], and using simple conversions from the inflation free logarithmic interest rate (k) to discount ratio (d), the net present value (NPV) of the aging bridge structure in the discrete space further simplifies to:

$$NPV = \sum_{i=1}^{i=n} \sum_{t=0}^{t=T} \lambda_i(t) e^{-\sum_{t=0}^{t=T} \lambda_i(t)} \cdot \frac{C_i}{(1+d)^t} \tag{7}$$

The mean annual rate of failure $\lambda_i(t)$, due to occurrence of a particular damage state i can be approximated by the annual probability of damage due to damage state *i* only as:

$$\lambda_i(t) = P_{i_{only}}(t) = PA_i(t) - PA_{i+1}(t) \tag{8}$$

where, $PA_i(t)$ is the annual probability of exceedance of damage state i at time *t*. To estimate these probabilities of exceeding different damage states, the use of risk assessment techniques incorporating bridge fragility curves is employed. These seismic fragility curves are statements of the probability of bridge damage conditioned upon earthquake intensity for different damage states. While further details on the construction of time dependent seismic fragility curves particular to bridges can be found elsewhere (Ghosh and Padgett 2009) the general expression representing the time dependent fragility of bridge system is presented herein as:

$$P[DS_i(t)|PGA = a] = \Phi\left[\frac{\ln(PGA) - \ln(m_i(t))}{\zeta_i(t)}\right] \tag{9}$$

where, $DS_t(t)$ is the damage state at time t, *PGA* is the peak ground acceleration, a being the realization of the PGA, $\Phi(\cdot)$ is the standard normal cumulative distribution function and $m_t(t)$ and $\zeta_t(t)$ are the median values and logarithmic standard deviation at time t in the service life of the bridge.

To evaluate the annual probability of exceedance the time dependent bridge fragilities can be convolved with the bridge site specific hazard curve as:

$$PA_t(t) = \int \Phi\left[\frac{\ln(PGA) - \ln\ m_t(t)}{\zeta_t(t)}\right]\left|\frac{dH(a)}{da}\right| da \quad (10)$$

Where, $H(a)$ is the location specific annual probability of exceeding a specific level of $PGA = a$.

Representative Case Study: Aging LCC Estimation of MSC Steel Girder Bridge

The LCC estimation framework for aging bridges developed in the previous section is applied to an example bridge to assess the changes in the net present value of losses due to seismic exposure, as compared to the pristine bridge. The bridge considered is a multiple-span continuous steel girder bridge, with characteristics similar to typical bridges of the same class found in CEUS. The bridge is assumed to be located in Caruthersville, MO and hence the corresponding seismic hazard curve for this location is used in estimating the net present value for the deteriorating bridge. The bridge geometry, corrosion deterioration modeling of its components (RC columns, steel bridge bearings) and its effects on bridge response, estimation of time dependent fragility curves and life-cycle cost formulation are described in the following sections.

Bridge Geometry and Finite Element Modeling

The MSC steel bridge considered in this study shown in Fig. 1 is a typical zero-skew, non-seismically designed bridge with three spans and multiple column bents, with each bent consisting of three columns. The bridge substructure is connected with the superstructures using high type steel bridge bearings: steel expansion bearings at the abutments and steel fixed bearings over the multiple-column bridge piers. To aid the nonlinear finite element analysis of the deteriorating bridge, a finite element model of the bridge is developed in OpenSees (Mazzoni et al. 2009) using the suggestions by Nielson and DesRoches (2007). The reinforcing steel in the concrete columns and the steel bridge bearings are prone to corrosion deterioration which leads to degradation in structural capacity of these components and affects the bridge performance. The deterioration mechanisms are discussed in the following sections.

Corrosion Deterioration Modeling of Reinforced Concrete Columns and Steel Bridge Bearings

The corrosion deterioration of the bridge components can be primarily attributed to chloride ingress through the reinforced concrete components which eventually leads to reduction in effective bar diameter of reinforcing steel in concrete columns and steel anchor bolts in the bearings. Additionally, the buildup of corrosion debris may potentially lead to 'frozen' or locked bridge bearings and thereby affecting the translational and rotational movements (Silano and Brinckerhoff 1993).

The corrosion of reinforcing steel in the RC columns and steel anchor bolts does not initiate on immediate exposure of the bridge to a corrosive environment. The process initiates after a certain interval of time called the corrosion initiation time which can be modeled as (Enright and Frangopol 1998)

$$T_i = \frac{x^2}{4D_c}\left[erf^{-1}\left(\frac{C_0 - C_{cr}}{C_0}\right)\right]^{-2} \tag{11}$$

where, x is the concrete cover depth, D_c is the diffusion coefficient, C_0 is the equilibrium chloride concentration at concrete surface and C_{cr} is the critical chloride concentration that causes the dissolution of the protective passive layer around the reinforcement or anchor bolt and initiates corrosion.

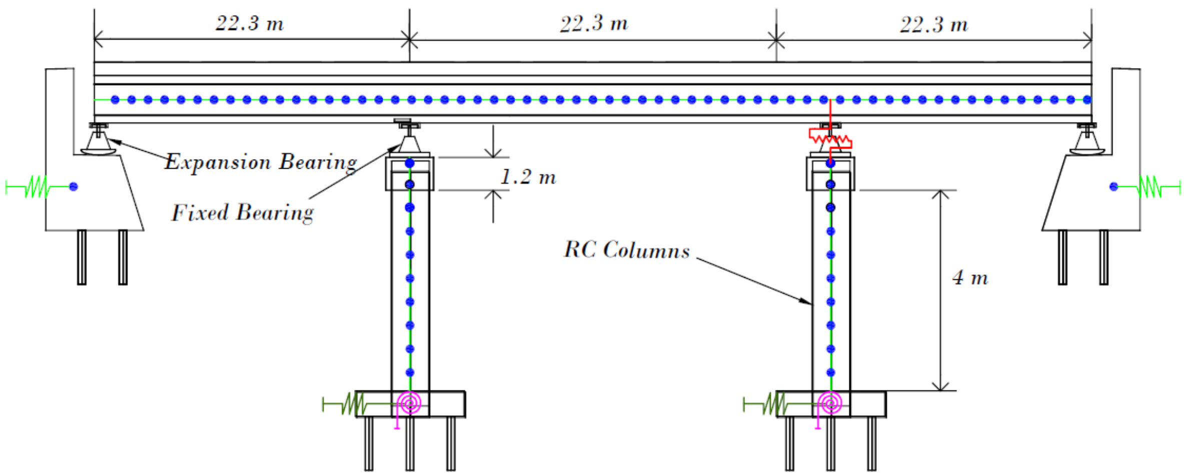


Figure 1 Case study MSC steel bridge showing components under deterioration consideration

Once corrosion mechanism is initiated, the subsequent area of loss of steel occurs at a uniform rate depending on the rate of corrosion given by,

$$A(t) = \begin{cases} mD_i^2\frac{\pi}{4} & for\ t \leq T_i \\ m(D(t))^2\frac{\pi}{4} & for\ T_i < t < T_i + D_i/r_{corr} \\ 0 & for\ t \geq T_i + D_i/r_{corr} \end{cases} \tag{12}$$

where, $A(t)$ is the area of reinforcing steel at time t, m is the number of rebars in the case of columns or number of anchor bolts in the bearing assembly, r_{corr} is the rate of corrosion, D_i is the initial diameter of reinforcing steel or anchor bolt and $D(t)$ is the diameter at time t.

The parameters assumed for this corrosion study and their assumed distributions are presented in Table 1 based on corrosion related studies on existing bridge components in the United States (Whiting et al. 1990; Weyers et al. 1994; Enright and Frangopol 1998). The base environmental exposure condition leading to corrosion considered in this study is the case of deicing salt exposure.

Under these assumptions, the normalized (with respect to initial) area loss of reinforcing steel in the concrete columns is shown in Fig 2a. Also shown in the figure in the degradation of the moment resisting capacity of the deteriorated 50 year old column and reduced ultimate lateral strength of the steel fixed bearings in the transverse direction.

Table 1. Descriptors of random variables affecting the corrosion deterioration of RC columns

Descriptor	**Unit**	**Distribution**	**Mean**	**COV**
Cover Depth (x)	cm	Log-normal	3.81	0.20
Diffusion Coefficient (D_c)	cm^2/year	Log-normal	1.29	0.10
Surface Chloride Concentration (C_0)	wt % concrete	Log-normal	0.10	0.10
Critical Chloride Concentration (C_{cr})	wt % concrete	Log-normal	0.040	0.10
Rate of Corrosion (r_{corr})	mm/year	Log-normal	0.127	0.3

* COV = Coefficient of variation

In the steel expansion bearings, corrosion deterioration also causes reduction in thickness of bridge keeper plates besides causing shear strength loss of anchor bolts. Additionally following the suggestions of Mander et al. (1996) it is assumed that due to corrosion debris accumulation the coefficient of friction of expansion bearings increases from 0.04 to 0.12 in the longitudinal direction. Further details on steel bearings and RC column deterioration can be found in Ghosh and Padgett (2009).

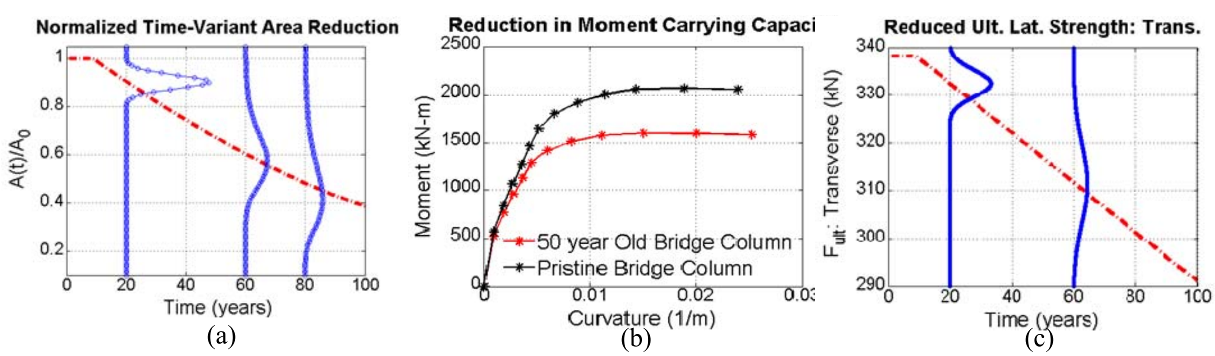
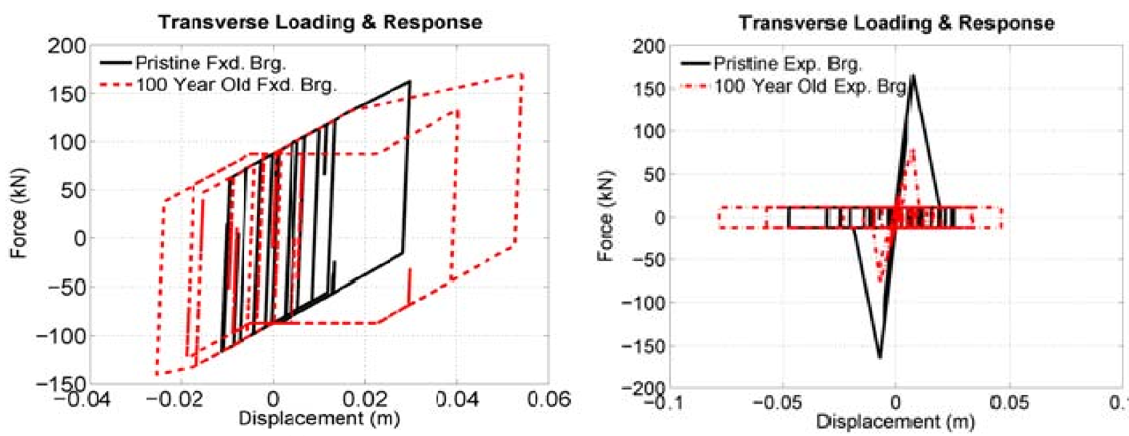


Figure 2 a) Area reduction of reinforcing steel in concrete columns b) reduced moment resisting capacity of corroded column and c) reduction in the ultimate lateral strength of fixed bearings in the transverse direction.

Effect of Corrosion of Bridge Components on Fragility

A deterministic analysis of the corroded case study bridge subjected to an earthquake of intensity 0.5g PGA reveals several interesting trends in the bridge response. It is observed that in the longitudinal direction, a higher damage localization in the concrete columns results in significantly high column curvature ductility demands and reduced fixed bearing deformations. Additionally, expansion bearing deformations are also reduced in this direction due to increased coefficient of friction as a result of corrosion debris accumulation. In the transverse direction however, the demand placed on the columns is negligible. Additionally, the reduced ultimate strength and post-yield stiffness of the corroded fixed and expansion bearings results in considerably high deformation demand on these structural components. This is shown in Fig. 3 for both type of bearings. It is observed that there is approximately 67% and 65% increase in the peak displacement of the corroded 100 year old fixed and expansion bearings respectively as compared to the pristine bearings. These findings from deterministic analysis highlight the detrimental effects of corrosion on critical components like concrete columns and bearings.



(a) (b)

Figure 3 Transverse loading and response of a) corroded fixed bearings and b) corroded expansion bearings along the service life of the bridge.

Although a single deterministic analysis helps to identify and analyze the critical bridge components, development of bridge seismic fragility curves requires a full probabilistic analysis. This is made possible with the help of 96 two component ground motions from the Wen and Wu (2001) and Rix and Fernandez (2004) ground motion suites, particular to the ground motion characteristics of CEUS. The probabilistic analysis is further aided by sampling upon uncertainties in concrete compressive strength, steel strength, foundation pile stiffness, damping ratios, gap between the deck and abutment, corrosion parameters affecting the cross sectional area of longitudinal reinforcement and as well as the strength of bridge bearing components.

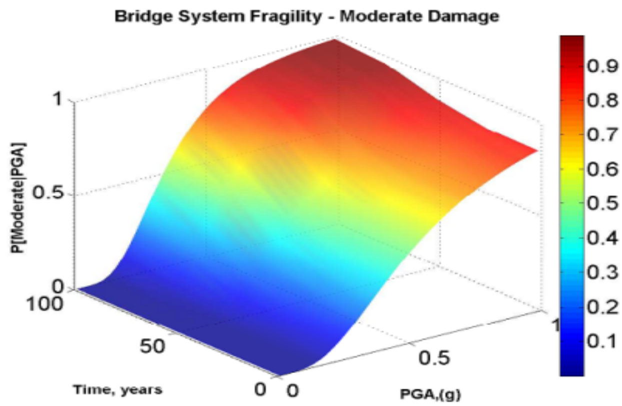


Figure 4 Deteriorating bridge fragility surface for moderate damage state.

With a full probabilistic analysis carried out at different points in time, 3-dimensional fragility surfaces are constructed which indicate the probabilities of exceedance of a particular damage state given a ground motion intensity and a point in time along the service life of the deteriorating bridge. This is shown in Fig. 4 for the moderate damage state of the bridge, though all four damage states are analyzed (slight, moderate, extensive, and complete). Subsequently, a regression analysis is conducted to establish time dependent polynomial functions to estimate the median and dispersion values for different damage states at different points in time in the life of the aging bridge. With the time dependent fragility parameters established, the net present value of the losses due to seismic exposure of the deteriorating bridge can be estimated using the formulations in previous sections.

Seismic LCC of Aging Bridge

In addition to being a function of the annual probability of occurrence of different damage states, the net present value presented in Eq. 7 is also a function of the bridge lifetime (T), discount ratio (d) and the cost associated with the repair of each damage state (C_i). For the present study, an inflation adjusted discount ratio of 2%, a service life of 100 years and a mean corrosion rate of 0.127 mm/year (Table 1) is taken as the base case. With the assumption of repair cost ratios presented by Basoz and Mander (1999) (Table 2), the costs associated with restorations corresponding to each damage state is calculated as a fraction of the bridge replacement cost. For the present case study, the replacement cost is assumed to follow a normal distribution with a mean of $94.37 per sq.ft. of deck area and a standard deviation of $18.36 as per the suggestions by Nilsson (2008) for typical MSC steel bridges in the CSUS. Additionally, to account for indirect losses due to increased travel time pertaining to bridge damage, total cost of losses associated with each damage state is assumed to be 13 times larger than the estimated repair costs.

Table 2 Repair Cost Ratios suggested by Basoz and Mander (1999)

Damage State	**Repair Cost Ratio**
Slight	0.03
Moderate	0.08
Extensive	0.25

Complete	1.0 (for n < 3)
	2.0/n (for n ≥ 3)

With the above assumed parameters, the estimated probability density function of the seismic LCC of a bridge with 100 years service life reveals that the mean LCC increases from \$122280 for the pristine bridge to \$136890 for the deteriorated bridge. Other than the 12% increase in the mean seismic LCC, a 20 % increase in the standard deviation is also observed. A preliminary sensitivity study also reveals that the LCC of the deteriorating as well as the pristine bridge is significantly affected by changes in the discount ratio and assumed service life of the bridge. Figs. 5a and 5b shows the effects of discount ratio and service life on the seismic LCC of the pristine and aging bridge. It is observed that lower values of discount ratio and longer service lives would individually result in increase in the mean and standard deviation of the net present value for the as-built and degraded bridge. It is also noted that estimation of seismic LCC is more sensitive to changes in discount ratio as compared to changes in length of service life.

Fig 5c on the other hand shows the influence of corrosion rate on the LCC of the aging bridge corresponding to an upper value (pertaining to severe deicing salt chloride ingress) and lower value (pertaining to mild atmospheric corrosion) of the corrosion rate as compared to the base case. Naturally as one would expect, a higher corrosion rate indicates a higher chance of structural deterioration and subsequently a higher estimate of seismic LCC. From the present study it is clear that a combination of low discount ratio, a long service life and high reinforcement corrosion rate will result in the highest estimate of seismic LCC.

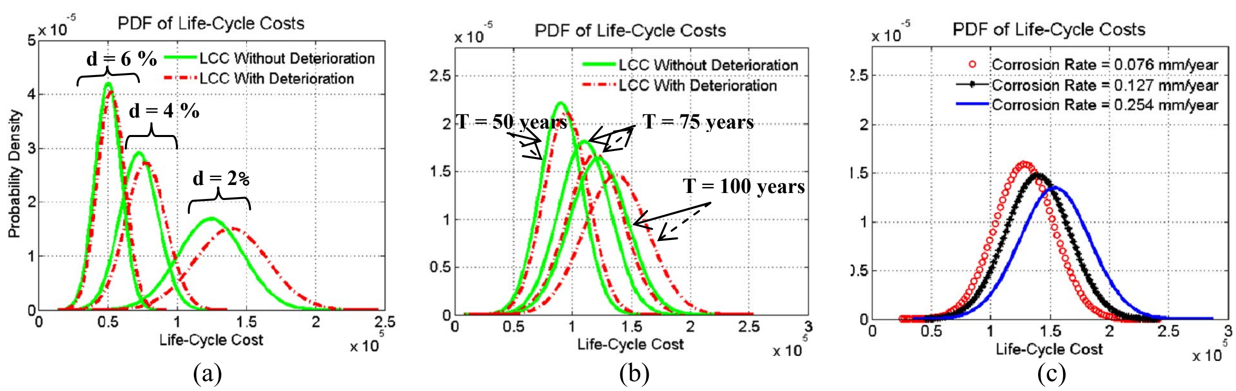


Figure 5 Sensitivity of seismic LCC estimation with a) different discount ratios (T=100 years), b) different service lives of the deteriorating bridge (d =2%) and c) different corrosion rates (d=2%, T=100 years)

Conclusions

This study focuses on the life-cycle cost estimation of aging bridges by taking into account the deterioration in structural performance due to corrosion of reinforced concrete columns and steel bridge bearings. Traditionally, seismic life cycle cost models are primarily based on vulnerability assessment of pristine bridges without taking into account the effects of deterioration mechanisms. An approach to estimate the seismic life cycle cost of deteriorating bridges is introduced considering a non-homogeneous Poisson process to account for the increased probability of failure of the bridge along its service life. A representative case study of a typical multi-span continuous steel girder bridge is also presented which reveals an approximate 12% increase in the mean net present value of the deteriorated bridge as compared to the pristine bridge.

The seismic life-cycle cost of the aging bridge is found to be sensitive to changes in parameters such as the discount ratio, length of service life and the rate of corrosion of the environment. While the discount ratio is found to be most significant parameter affecting the net present value, it is only natural to expect that a longer service life and a higher rate of corrosion will contribute to an increase in the seismic life-cycle cost of the aging bridge. Additionally, site of the bridge location and seismic characteristic of the region is also assumed to affect such cost estimates. Future research on this topic will emphasize aggregate estimates of the life-cycle cost derived from bridge component level fragility and repair, instead of traditionally relying on repair cost ratios at the system level. Additionally, the effectiveness of retrofit strategies, targeting improvement in both corrosion deterioration and seismic vulnerability, to reduce the life cycle cost will also be assessed.

Acknowledgements

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