IMPACT OF LONG-TERM MATERIAL DEGRADATION ON SEISMIC PERFORMANCE OF A REINFORCED CONCRETE BRIDGE

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ABSTRACT

A Performance-Based Earthquake Engineering (PBEE) assessment methodology is being used to study the impact of long-term material degradation on seismic performance of reinforced concrete bridges. The methodology, currently under development by the Pacific Earthquake Engineering Research Center, is being applied to an overpass with typical California highway details, and preliminary results are presented. The structure was analyzed in its sound as well as a deteriorated condition considering spalling and reduced reinforcing steel areas resulting from corrosion. A simple modeling approach was employed to study analytically the impact of such deterioration on drift demands and potential damage to the structure. The ground motion occurrence, resulting structural response, and predicted damage were combined in PBEE assessments of the sound and deteriorated bridges. The deterioration resulted in a factor-of-three increase in the frequency of steel yielding caused by earthquake shaking. The PBEE framework provides a systematic method to compare quantitatively the effect of deterioration on seismic damage.

Introduction

Deterioration in reinforced concrete structures is a common problem resulting in corrosion of steel reinforcement and spalling of concrete. In seismic design, reinforced concrete structures are sized assuming that the structure will be sound and not have any damage from deterioration when an earthquake loading may strike. Ideally, regular maintenance would ensure that the design properties remain throughout the life of the structure. However in reality, deterioration is widespread, as indicated by the often cited ASCE report card for America's infrastructure (ASCE, 2005). It is unclear to what extent deterioration can occur before the seismic vulnerability of the structure is increased and by how much. Razak and Choi (2001) investigated experimentally the effect of corrosion on the natural frequency and modal damping of reinforced concrete beams. Lee *et al.* (2002) investigated experimentally the influence of rebar corrosion on the strength of a reinforced concrete column under cyclic loading and found that structural behavior changes were largest due to decreased confinement from spalling and a reduction of mechanical properties in corroded steel reinforcement.

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The research presented here focuses on investigating analytically the influence of deterioration in reinforced concrete columns on the dynamic response of highway bridges. To quantify the impact of deterioration on seismic response and performance, a systematic performance-based earthquake engineering assessment methodology, currently under development by the Pacific Earthquake Engineering Research (PEER) Center, was employed. The PBEE methodology has been applied to two case study highway bridges. Sample results from one case study, a typical highway overpass in California, USA, are presented here.

Performance-Based Earthquake Engineering

The PBEE methodology currently under development by the PEER Center involves four main steps: ground motion hazard analysis, structural analysis, damage analysis, and loss analysis (Moehle and Deierlein, 2004). In ground motion hazard analysis, the annual frequency with which a given seismic Intensity Measure (IM), e.g. spectral acceleration, will exceed certain levels is calculated and expressed with a hazard curve. In structural analysis, the relationship between IMs and engineering demand parameters (EDPs) such as drift ratios, are evaluated. EDPs are measurements that can be determined by analyses but are not easily related to amounts of damage or repair needs. In damage analysis, damage measures (DMs) are related to EDPs, where DMs are conditions such as spalling in reinforced concrete that can be related to a required repair method. In loss analysis, decision variables (DVs) such as cost or casualties are related to the DMs, to facilitate communication between engineers and owners in terms of the potential consequences of a particular design.

Each step in the PBEE assessment methodology is carried out individually in a probabilistic fashion. All of the steps (or any sub-set in series) can be combined to provide an assessment of overall system performance. For the current investigation of the influence of deterioration on reinforced concrete bridge performance, examples from the first three steps (ground motion hazard analysis, structural analysis and damage analysis) are given. The final result is a computed annual frequency of a given damage measure occurring, as explained below.

Influence of Deterioration on Seismic Response of a Highway Bridge (PBEE Structural Analysis: IM-EDP Relations)

The influence of local deterioration in reinforced concrete columns (piers) on the seismic response of highway bridges was investigated. The influence of deterioration in bridge columns was investigated by comparing the analytical results of bridge response using sound reinforced concrete columns with results using deteriorated reinforced concrete columns. For the PBEE assessment, the Intensity Measure (IM) selected was spectral acceleration and the Engineering Demand Parameter (EDP) presented here was column drift.

Highway Bridge Model and Analysis

One of the highway bridges selected for investigation and presented here was based on typical designs from the California Department of Transportation (Caltrans) (Ketchum et al., 2004). The bridge is a 5-span post-tensioned box girder bridge (Fig. 1). The reinforced concrete section of the columns is shown in Fig. 2.

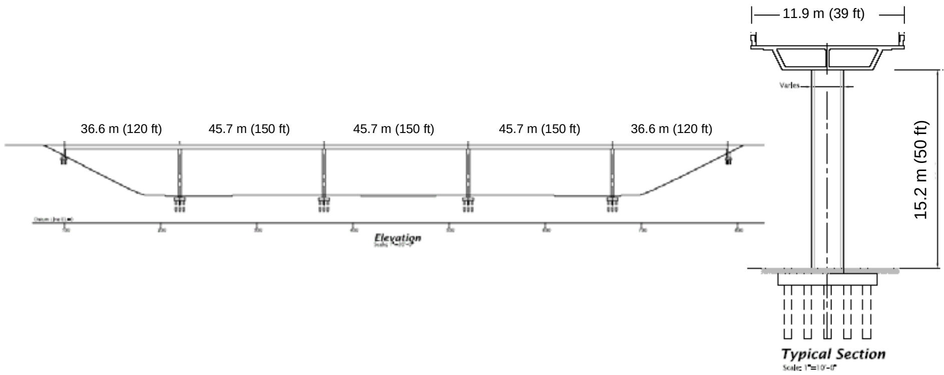

Figure 1. Elevation of Caltrans highway bridge (adapted from Ketchum et al., 2004)

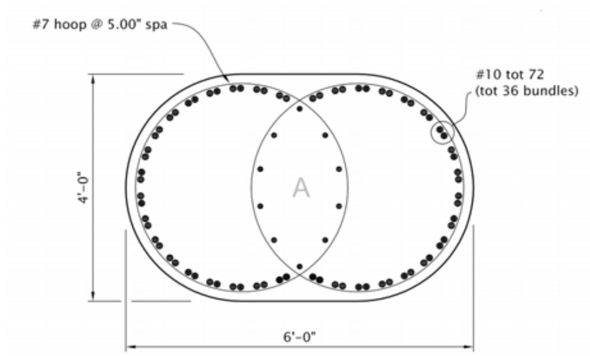

Figure 2. Cross-section of Caltrans highway bridge column (Mackie & Stojadinovic, 2005)

A 2D model of the bridge using fiber elements was analyzed using OpenSees (2005). The fiber elements called for the use of uniaxial nonlinear material properties for the concrete and reinforcing steel. The concrete was modeled as confined according to Mander et al. (1988). The steel reinforcement was assigned the Giuffré-Menegotto-Pinto model with isotropic strain hardening and including the Bauschinger effect under cyclic loading. The fiber elements used assume plane strain and thus perfect bond between the steel and concrete. To account for bond-slip, a beam-column joint element (Lowes et al. 2003) was used at the base of the column. Three of the four nodes on the joint element were fixed and stiff material properties were provided for the bond-slip springs that are not at the column-joint intersection. Details are given in Matsuki (2005) and a similar approach was taken in Douglas and Billington (2005).

The deterioration considered was cover concrete spalling and corrosion of longitudinal reinforcement, which was represented in the analyses by reducing the cross-sectional areas of the cover concrete and longitudinal reinforcement, by 50%. This general modeling approach was based on a calibration and validation study presented in Matsuki (2005). Two-dimensional nonlinear dynamic analyses were performed in order to evaluate the seismic response of the bridge under strong earthquake motions. Specifically, incremental dynamic analyses (IDAs) were employed to evaluate the response of the bridge (Vamvatsikos and Cornell, 2002). For the incremental dynamic analyses, the first-mode spectral acceleration (S_a) of a suite of 20 ground motions (Table 1) was scaled to 0.2, 0.4, 0.6, 0.8, 1.0 and 1.5g and dynamic analyses (using the Newmark method) were performed on the bridges. The ground motions were imposed on the nodes at the fixed base of columns. These analyses were performed on a bridge with sound columns and one with columns that included the representation of cover concrete spalling and severe longitudinal reinforcement corrosion as described above.

Table 1. Earthquake Ground Motions*

Case	Earthquake Motion	PGA (g)	PGV (cm/sec)	PGD (cm)	Sa(T_1=1.5sec,5%) [Caltrans Br] (g)
1	Japan Highway Design EQ I-I-2	0.326	N/A	N/A	0.630
2	Japan Highway Design EQ II-I-2	0.781	N/A	N/A	0.581
3	Kobe, Japan / KJM000	0.821	81.3	17.7	0.801
4	Kobe, Japan / KJM090	0.599	74.3	20.0	0.299
5	Imperial Valley / ELC180	0.313	29.8	13.3	0.168
6	Imperial Valley / ELC270	0.215	30.2	23.9	0.187
7	Loma Prieta / CLS000	0.644	55.2	10.9	0.185
8	Loma Prieta / CLS090	0.479	45.2	11.4	0.338
9	Northridge / STM090	0.883	41.7	15.1	0.340
10	Northridge / STM360	0.370	25.1	7.2	0.250
11	Cape Mendocino / RIO270	0.385	43.9	22.0	0.295
12	Cape Mendocino / RIO360	0.549	42.1	18.6	0.230
13	N. Palm Springs / NPS210	0.594	73.3	11.5	0.439
14	N. Palm Springs / NPS300	0.694	33.8	3.9	0.122
15	Chichi, Taiwan / CHY006-E	0.364	55.4	25.6	0.405
16	Chichi, Taiwan / CHY006-N	0.345	42.8	15.2	0.273
17	Kocaeli, Turkey / DZC180	0.312	58.8	44.1	0.248
18	Kocaeli, Turkey / DZC270	0.358	46.4	17.6	0.593
19	Victoria, Mexico / CPE045	0.621	31.6	13.2	0.270
20	Victoria, Mexico / CPE315	0.587	19.9	9.4	0.095

* *From the PEER strong motion database (2005)*

IM-EDP Relationships

Fig. 3 illustrates the spectral acceleration versus maximum drift ratio (peak horizontal displacement divided by the length from the base of the column to the center of gravity of the girder). Interpolation curves of median values are plotted along with curves bounding the data at 15% and 85%. Comparison of the median between the sound case and the deteriorated column case shows only minor differences. When differences are observed, they are more pronounced in higher levels of spectral acceleration. A more significant difference in the IM-EDP relationship is noted between the sound case and deteriorated case in the 85% curve again particularly at higher levels of spectral acceleration. The differences are most pronounced above S_a = 0.6, and indicate a larger dispersion in the results.

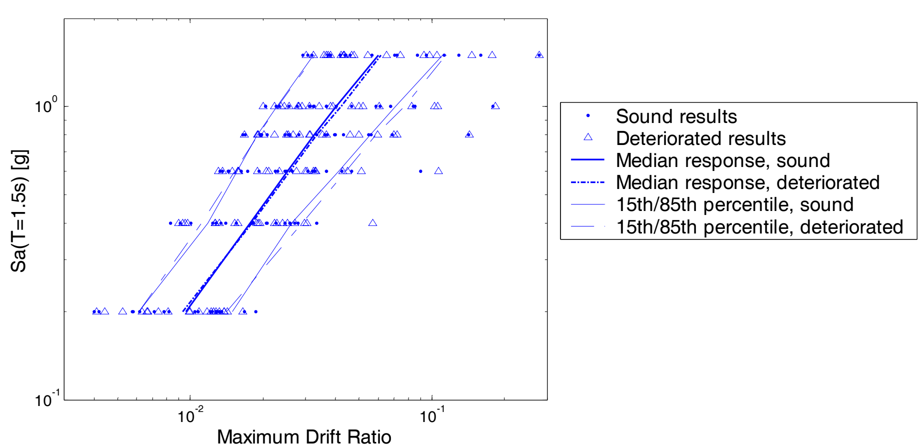

Figure 3. Spectral acceleration vs. maximum drift ratio

Influence of Deterioration on Seismic Damage of a Highway Bridge
(PBEE Assessment: EDP-DM Relations)

The influence of local deterioration in reinforced concrete columns on seismic damage to the highway bridge was investigated by developing EDP-DM relationships in the form of fragility curves. A fragility curve indicates a probability of damage in each level of seismic intensity or in this case, seismic demand (of drift). Fragility curves for highway bridges (and components of highway bridges) can be developed from empirical data (e.g. Basöz and Kiremidjian, 1997, Karim and Yamazaki, 2001) or analytically (e.g. Shinozuka et al., 2000, Choi et al., 2004). The authors are not aware of fragility curves developed for highway bridges with sound and deteriorated reinforced concrete shown in a comparative way as will be done here.

A comparison between fragility curves for a sound reinforced concrete bridge and a representative deteriorated reinforced concrete bridge was made. The same state of deterioration was assumed here as for the IM-EDP analysis presented above. Deterioration was considered to be localized at the bottom of the column. The development of the analytical fragility curves involves 9 steps described below. The first three were conducted in the development of the IM-EDP relationships described above.

1. <u>Construct analysis model</u>: The model of the typical Caltrans highway bridge that was used for the IM-EDP relationships was adopted for the fragility curve development.
2. <u>Select a suite of strong ground motions</u>: The 20 strong motions used for the IM-EDP relationships were also adopted here (Table 1).
3. <u>Perform Incremental Dynamic Analysis</u>: The results from the IDAs previously described were adopted here.
4. <u>Select Engineering Demand Parameters (EDPs)</u>: The single EDP of drift ratio was selected for consideration here.
5. <u>Select Damage Measures (DMs)</u>: Damage measures are descriptions of damage to structural or non-structural elements that can be easily associated with a post-earthquake decision such as repair method, repair cost, or structure closure. Damage measures considered here include crushing of cover concrete, crushing of the concrete core, and yielding of longitudinal reinforcing steel.
6. <u>Obtain EDP and DM Data from the IDAs</u>: From the nonlinear dynamic analyses the maximum compressive strain in the cover and core concrete, the maximum strain in the longitudinal reinforcement and the maximum horizontal displacement at the top of the pier were recorded.
7. <u>Perform regression analysis on DMs for a given EDP</u>: Linear regression analyses were conducted using the maximum drift ratio (EDP) as a predictor in Equation 1 to estimate the maximum compressive strain of cover concrete, maximum compressive strain of core concrete and maximum strain of longitudinal reinforcing steel. Note that the recorded strains used in the regressions are values indicating that the damage measures (DMs) of crushing and yielding have been reached.

$$y_i = a + bx_i + \delta_i \tag{1}$$

where *a, b*: coefficients obtained from linear regression analysis
x_i: The EDP (drift ratio in this case) caused by ground motion *i*

y_i: strains caused by ground motion i, used to indicate the onset of a DM
δ_i: the "residual," or prediction error, associated with ground motion i

8. Calculate conditional probability P(DM|EDP): Fragility is defined as the conditional probability of exceeding a prescribed limit state (in this case, a Damage Measure). To calculate this probability, the following approach was taken. Using the regression model from Equation 1, the relevant strains associated with a given EDP level can be predicted using Equation 2:

$$Y = a + bX + \Delta \tag{2}$$

where X is the EDP level of interest and Y is the distribution of resulting strain. The mean value of Y is equal to $a + bX$, and its standard deviation is equal to the standard deviation of the zero-mean random variable Δ, which accounts for the uncertainty in Equation 1. The standard deviation of Δ was estimated using the sample standard deviation of the δ_i's from Equation 1 and Δ was assumed to be normally distributed. This model can then be used to predict the probability of a Damage Measure occurring at a given EDP level.

Using this predictive model, fragility curves are then computed by finding the probability that the relevant strain associated with a given EDP level (as predicted by Equation 2) is large enough to cause the onset of a given DM. The relevant strains and threshold values are summarized below. Note that because Δ is normally distributed and Equation 2 is linear, then Y is also normally distributed and the fragility curves take the form of normal Cumulative Distribution Functions.

Table 2. Damage measures and occurrence criteria

Damage Measure	**Strain Value Used to Indicate DM**	**Threshold Value for Strain**
Cover concrete crushing	Maximum compressive strain in cover concrete	Strain associated with concrete crushing (0.003)
Core concrete crushing	Maximum compressive strain in core concrete	Strain associated with confined core concrete crushing (0.01)
Reinforcing steel yielding	Maximum strain in reinforcing steel	Yield strain of steel (0.0021)

9. Plot fragility curves: Fragility curves are plotted using the EDP and P (DM|EDP) calculated above. Results are presented in Figures 4-6.

Results

For the damage measure of cover concrete crushing, a 30% difference in the median capacity was observed between the sound and deteriorated bridge with the deteriorated bridge having less capacity against cover crushing. For the damage measure of yielding of longitudinal reinforcement, the deteriorated bridge had a median capacity 30% lower than that of the sound case. This general trend was expected given that deterioration was modeled with a reduced area of longitudinal steel. For the damage measure of core concrete crushing, a difference of less than 1% in the median capacity between the sound and deteriorated bridge was observed. It is noted that while the general trends in results seem reasonable, the exact causes are not further investigated here. Rather, the results are next combined with a hazard analysis and the previously presented structural analysis to perform a more complete PBEE assessment. Of interest is how any significant or insignificant differences in a single step may or may not be significant in the larger picture of the performance-based assessment. Improvements in modeling deterioration are under investigation to assess and improve the accuracy of predictions in the case study presented here.

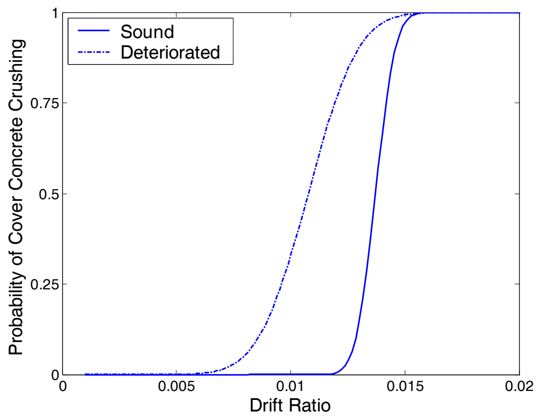

Figure 4. Cover concrete crushing

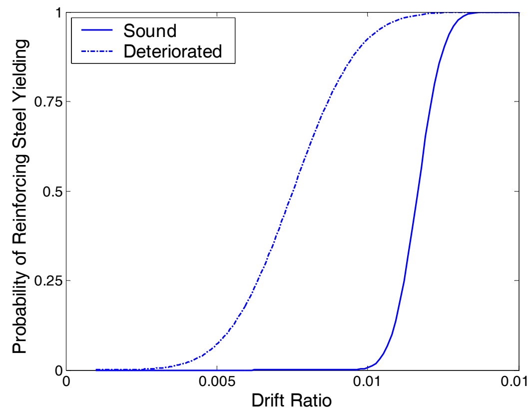

Figure 5. Longitudinal steel yielding

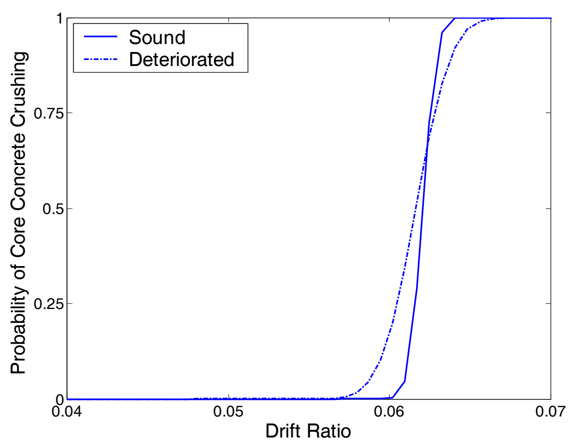

Figure 6. Core concrete crushing

Combined results using the PEER PBEE assessment methodology

In order to assess the performance of the original and deteriorated bridges, the above results were combined using the following PBEE assessment equation:

$$\lambda_{DM} = \int_{IM} \int_{EDP} G(DM \mid EDP) \left| dG(EDP \mid IM) \right| \left| d\lambda(IM) \right| \tag{3}$$

where $\lambda(IM)$ is the annual rate of exceeding a given ground motion intensity level IM at the specified site and $G(DM|EDP)$ and $G(EDP|IM)$ are conditional Complimentary Cumulative Distribution Functions obtained from the results above. By combining these conditional predictions and integrating over all possible IM and EDP values, one can compute the annual rate of damage measure DM occurring, denoted $\lambda(DM)$.

An intermediate result that is also informative is the relationship between IM and DM. This can be obtained by combining the structural response results and fragility curves using a reduced version of Equation 3:

$$G(DM \mid IM) = \int_{EDP} G(DM \mid EDP) \left| dG(EDP \mid IM) \right| \quad (4)$$

For example, the probability of steel yielding as a function of IM is displayed in Fig. 7 for two different levels of damage on the Caltrans bridge. It can be observed that the deteriorated bridge is more likely to have steel yielding than the undamaged bridge at a given IM level. This result reflects differences in the EDP-IM relationships for the two cases, as well as differences in their EDP-DM fragility curves.

Next, all of the data are combined using Equation 4 to compute the annual rate of a given DM occurring. In order to complete this calculation, it is necessary to obtain the ground motion hazard $\lambda(IM)$. For this application, the bridge is assumed to be located in Los Angeles, and the ground motion hazard was computed for the site using Probabilistic Seismic Hazard Analysis. The result is shown in Fig. 8. For the two damage levels shown in Fig. 7, the following results were obtained: at the Los Angeles site considered the original bridge will have an annual rate of steel yielding of 0.021, while the bridge with corroded longitudinal steel has an annual rate of 0.060 (i.e., one would expect steel yielding about once every 50 years for the original bridge versus once every 17 years for the deteriorated bridge).

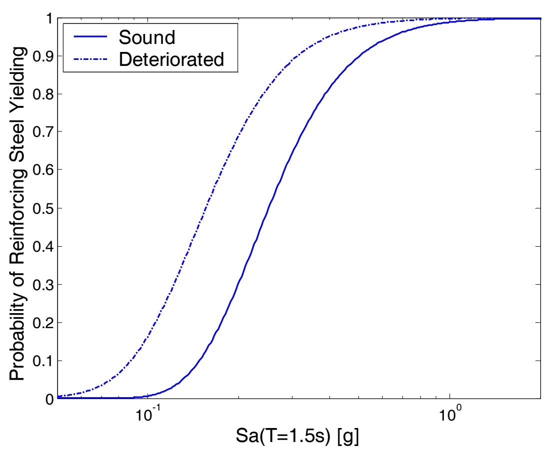

Figure 7. Probability of steel yielding as a function of ground motion intensity

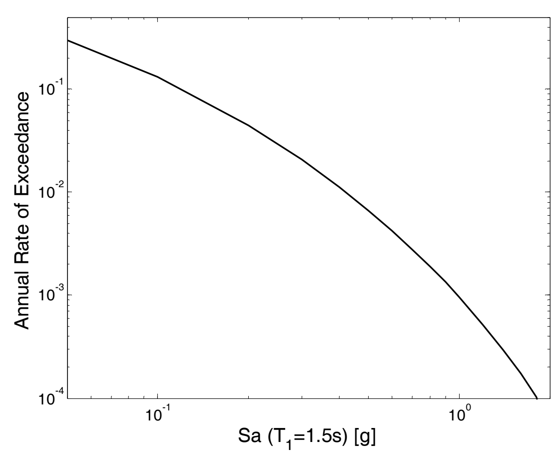

Figure 8. Ground motion hazard for the site in Los Angeles, California

The large difference in rates of yielding (a factor of three) between the original and deteriorated structure may be surprising at first, given that IM-EDP response relationships did not change dramatically and that changes in median values of the fragility curves were on the order of 15-35%. However, these changes are amplified when the ground motion hazard is considered, because low levels of IM are generally much more common than high levels (in Figure 8, doubling the considered spectral acceleration intensity results in roughly a factor of 5-10 decrease in rate of exceedance). This means that when the probability of yielding is increased at low IM levels, as is observed in Fig. 7, the annual rate of yielding increases significantly. This potential for greatly increased rates of damage will be helpful for policy makers to recognize when choosing what level of resources to devote to repair and maintenance of infrastructure.

Future Work

The pilot research presented here points to several areas of research needed to assess more fully the influence of deterioration on the seismic vulnerability of highway bridges. Only one deterioration pattern was assumed for these analyses and only for columns. Many more levels of deterioration and in different parts of bridge structures should be studied. Furthermore, all bridge columns were assumed to deteriorate uniformly. More detailed probabilistic models of the bridge condition should be considered in future research. This research was limited to the investigation of the influence of deterioration in reinforced concrete. It would be useful to evaluate analytically the influence of seismic retrofits (i.e. repair of deterioration) on the seismic response and damage of bridges. Finally, a study of the repair cost of highway bridges with deteriorated reinforced concrete should be conducted by expanding the PBEE assessment conducted here to include the fourth step of Loss Analysis.

Conclusions

In this paper, the influence of deteriorated reinforced concrete columns on the seismic response of a highway bridge with typical Caltrans details was investigated using the PEER PBEE assessment methodology. Details of two of the four PBEE steps were given for a comparison between a sound bridge and a deteriorated bridge wherein a reduction in cover concrete and longitudinal steel area were assumed to represent spalling and corrosion, respectively. It was found that for the structural analysis step (IM-EDP), differences between the sound case and the deteriorated case are generally minor in terms of maximum drift ratio under the suite of ground motions considered. Where differences were observed, they were more pronounced in higher levels of spectral acceleration. For the damage analysis step (EDP-DM), a 30% reduction in the median capacity against cover concrete crushing and steel yielding was observed in the deteriorated bridge relative to the sound bridge. For core concrete crushing there was a less then 1% difference in median capacity between the two bridge conditions. The assessments of bridge performance were then combined with a ground motion hazard curve to compute the annual frequency of exceeding the given damage levels. For the example calculation shown here, corrosion of longitudinal reinforcing steel resulted in a factor of three increase in the annual rate of reinforcing steel yielding due to seismic loading.

In summary, the influence of local deterioration in reinforced concrete columns on seismic response and damage of highway bridges was shown quantitatively based on a performance-based earthquake engineering methodology. It was found that using the PEER Performance Based Earthquake Engineering assessment methodology provides a systematic approach to comparing sound and deteriorated structures and generates observations not possible when only isolated analyses are conducted (e.g. a structural or damage analysis only). In particular, the influence of deterioration was demonstrated to a large enough degree to warrant a more thorough study of the effect of deterioration on seismic vulnerability.

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