Consideration of Post-Repair Performance in Seismic Loss Assessment of Structures

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ABSTRACT: Performance-based earthquake engineering is a methodology that allows for seismic assessment of structures in terms of decision variables that are most important to stakeholders. However, currently, this approach does not facilitate consideration of changes in structural behavior if the structure is repaired, and, hence, cannot be used to compare service life performance differences between the different repair strategies. This study develops and illustrates a probabilistic seismic loss assessment framework that explicitly considers structural post-repair performance and examines the implications of the selected repair strategies over remaining service life of the structure after an earthquake. The proposed framework uses the Monte Carlo method to simulate many service life scenarios of a structure that are consistent with site-specific seismic hazard. For each of these scenarios, expected losses are calculated, considering changes in the performance of the structure if previous earthquake in the scenario warrants extensive repair that changes structural behavior. The framework is applied to two reinforced concrete bridges to illustrate the application.

1. INTRODUCTION

Performance based earthquake engineering (PBEE) is a probabilistic methodology intended to assess a structure's seismic performance and represent it with measures of interest to stakeholders (Deierlein at al. 2003), such as economic losses. However, in its current form, the framework does not allow for changes in performance over the service life of the structure. These changes could come from deterioration of the structural elements with time, or from retrofit or repair of certain elements.

This paper extends the current seismic loss assessment framework to account for earthquake damaged structures that are subsequently repaired, explicitly considering the post-repair performance of the structures. The study is motivated by potentially significant differences in performance of original versus repaired structures, and among structures repaired using different strategies. For example, repaired reinforced concrete (RC) bridge columns may have better or worse performance than the original columns, depending on the repair strategy used (e.g. He et al. 2015; Valigura at al. 2018).

The proposed framework uses a Monte Carlo (MC) method to simulate many service life scenarios of a structure that are consistent with site-specific seismic hazard. These service life scenarios may involve one or more earthquakes, each associated with a shaking intensity at the site of interest. For each of these scenarios, expected losses are calculated from nonlinear dynamic analysis and an assembly-based probabilistic loss assessment framework. If a previous earthquake has already damaged the structure to an extent that repairs are required, these assessments are based on performance of the repaired structure.

The framework is applied to two RC bridges. The results are then compared with results of the classic PBEE assessment framework to demonstrate the importance of accounting for post-repair behavior for informed decisions about repair actions.

2. BACKGROUND

2.1. PBEE

A generalized form of PBEE consists of four elements: a hazard model, a demand model, a damage model, and a decision model (Deierlein et al. 2003). In the PBEE framework, each of these models uses the output of the previous model to probabilistically evaluate the expected value of a decision variable based on site and structure information. The hazard model characterizes the frequency of exceedance of an intensity measure (IM) of interest; in the demand model, the structure is analyzed and the engineering demand parameters (EDPs) are quantified; the damage model characterizes damage states (DSs) that can occur in the physical structure given calculated EDPs; and, finally, the decision model evaluates the DSs in terms of decision variable(s) (DV). This framework permits propagation of key sources of uncertainty associated with each step of the assessment to the DV. PBEE method has been formalized by the FEMA P-58 (FEMA 2012) document.

2.2. Loss assessment for time-variant performance

There are two types of changes of structural behavior that may occur over the service life: (i) due to chronic stressors and (ii) due to acute stressors. Changes due to chronic stressors may manifest as degradation of structural behavior with time, i.e. aging of structures, corrosion of steel members and structural reinforcement, or stiffening of elastomeric bearings in bridges. The degradation of the behavior can be expressed as a function of time, and hence so can its effects on the seismic (and other hazard) vulnerability. For example, Ghosh and Padgett (2010) proposed a framework for time-dependent fragility analysis, and developed a polynomial to represent the effect of aging on the median value of IM for a given DS fragility curve as a function of time. Shekhar et al. (2018) incorporated this framework into life-cycle cost analysis of highway bridges. Bisadi and Padgett (2015) discretized a site seismic hazard curve into many service life scenarios (similarly to presented study). They used scenarios to optimize design variables to obtain life-cycle costs lower than a predetermined threshold, but considered only effects of aging, not multiple repair strategies.

Changes in performance due to acute stressors may come from rapid changes in structural properties or geometry. These changes could be associated with, for example, seismic repair or retrofit of structures. The effects of the repairs and retrofits on structure's performance have been examined in a number of studies (Harrington 2016; He et al. 2015), which showed that the difference in seismic behavior can be significant. This study presents a framework that can incorporate changes in performance due to both chronic and acute stressors (main focus) in the seismic loss assessment of a structure.

3. PROPOSED SEISMIC LOSS ASSESSMENT FRAMEWORK

The presented method uses MC simulation to capture the discontinuities in structural performance due to repair or retrofit and, hence, more realistically predicts repair costs over the service life of the structure. The method represents the seismic hazard at a site by multiple service life scenarios. Each earthquake in each scenario is evaluated in terms of seismic losses, and of how it might change future performance. The performance of the bridge during each scenario is quantified by annualized losses that represents the total repair costs or losses experienced by the structure, quantified by an annuity. The annualized losses can be compared amongst different design alternatives to establish the most economically sound design.

3.1. Service life scenarios that represent seismic hazard

In the U.S., seismic hazard curves are available from the online USGS (2018) hazard tool. These curves quantify the mean annual frequency of exceedance of a given IM level. MC simulation is then involved to discretize the hazard into a series of earthquake/shaking intensity events.

One realization of a hazard consistent scenario for a site at UC Berkeley is shown in Figure 1. This service life scenario consists of 7 earthquakes with Sa(T=1s) between 0.01 – 1 g. From the figure it is obvious that the low intensity shaking governs the seismic hazard on the site, while large intensity shaking occurs sporadically. This particular scenario would not replicate the seismic hazard curve for the site, but if one used several thousand or millions of randomly generated scenarios and built a time span from these scenarios, they would be consistent with the seismic hazard.

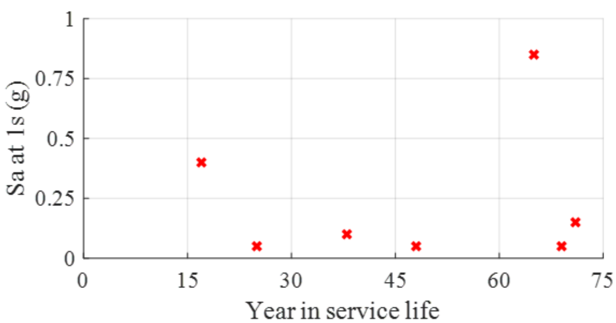

Figure 1. A service life scenario for a site at UC Berkeley

3.2. Annualized losses accounting for post-repair performance

The annualized lifetime losses can be calculated easily using the classic PBEE approach by convolving the seismic hazard curve and cost curve, which relates repair costs of the structure to IM. There are two major steps in obtaining the cost curve for a structure. First, the engineer needs to assess the vulnerability of the structure and its components. The vulnerability is often expressed in terms of fragility curves. The fragility curves can be assembled either using simplified analysis such as pushover analysis and empirical fragility curves, or through more sophisticated dynamic analysis, such as incremental dynamic analysis (IDA) (Vamvatsikos and Cornell 2002) or similar analysis, to obtain EDPs vs. IM. The EDPs are then compared to the onset of DSs to create fragility curves. In the second step, the fragility curves are converted into losses by evaluating the repairs associated with a given DS in terms of DV.

The *proposed method* considers a simulation model of the original (undamaged) structure and model(s) of the repaired structure. Each of these simulation models is analyzed through IDA to obtain structural responses, and through comparison with DS fragility curves, repair costs and losses. Depending on the structural behavior, and the nature of the repairs for different damage levels, the analysis may require more than just one repair model. In particular, a repair model should be developed for any repair strategy that significantly affects the structural behavior of the structure. As an example, a significantly damaged RC column repaired with steel jacket will behave much differently from the original column, and a repair model should be developed for this case. However, minor concrete spalling of the cover concrete repaired with patching will most likely cause no significant change in behavior.

For each scenario, the MC simulation procedure is as follows, and shown in Figure 2:

1) For the first earthquake event in the scenario, a set of correlated EDPs is generated from the model of original structure based on the IM of the ground motion experienced.
2) The EDPs are compared with the onset of the damage states, and damage states of all elements are determined. The fragilities of the damage states are based on experimental data or observations, and treated probabilistically.
3) The repair methods for each element are designed based on current design and repair provisions to obtain the material/labor/process quantities. These are then evaluated using unit "repair costs" to obtain DV. (More thorough description of Steps 1 - 3 can be found in Valigura et al. (2018).)
4) Based on the extent of the repairs, a repair

model that best characterizes the behavior of the repaired structure is selected for the next earthquake in the scenario service life. If the damage and repair are not significant, the current model remains. Once the model is chosen, the next earthquake in the service life scenario is analyzed with Steps 1 - 4.

5) When all earthquake events in the scenario are analyzed, the values of DV from each earthquake are summed and divided by the length of the service life to obtain annualized losses (AL).
6) Steps 1-5 are performed for all scenarios. The mean value of AL from all scenarios then represent the EAL.

If the step of updating current model (Step 4) is skipped, DVs in all earthquakes are based on performance of original structure, and the calculated annualized losses from the *proposed method* are equal to those predicted using *FEMA P-58*.

4. CASE STUDY STRUCTURES

4.1. Overview of prototype bridges

The method is applied to two prototype bridges; Prototype bridge 1 (PB1) is model bridge No. 3 from Ketchum et al. (2004). This bridge is a 5-span post-tensioned concrete box girder superstructure bridge with monolithic piers. Prototype bridge 2 (PB2) is the La Veta Avenue Crossing. It is a 2-span concrete box girder superstructure bridge with the pier consisting of two columns. The design of column transverse reinforcement of PB2, and of abutments of both PB1 and PB2 were altered from their original designs to better characterize behavior of the newly built bridges in high seismic areas (Valigura et al. 2018).

4.2. Column repair strategies

The strategies for column repairs are based on the DS. The DS for columns assumed in this study are adopted from Valigura et al. (2018) and presented in Table 1.

For DS1 and 2, the repairs consist of epoxy injections and concrete patching and do not significantly affect the capacity of the column. The damage during DS3 through 5 compromises the shear strength and confinement of the column (Vosooghi and Saiidi 2013), and, hence, external jackets are needed to restore the shear, moment and deformation capacity of the column. This study considers carbon fiber reinforced polymer

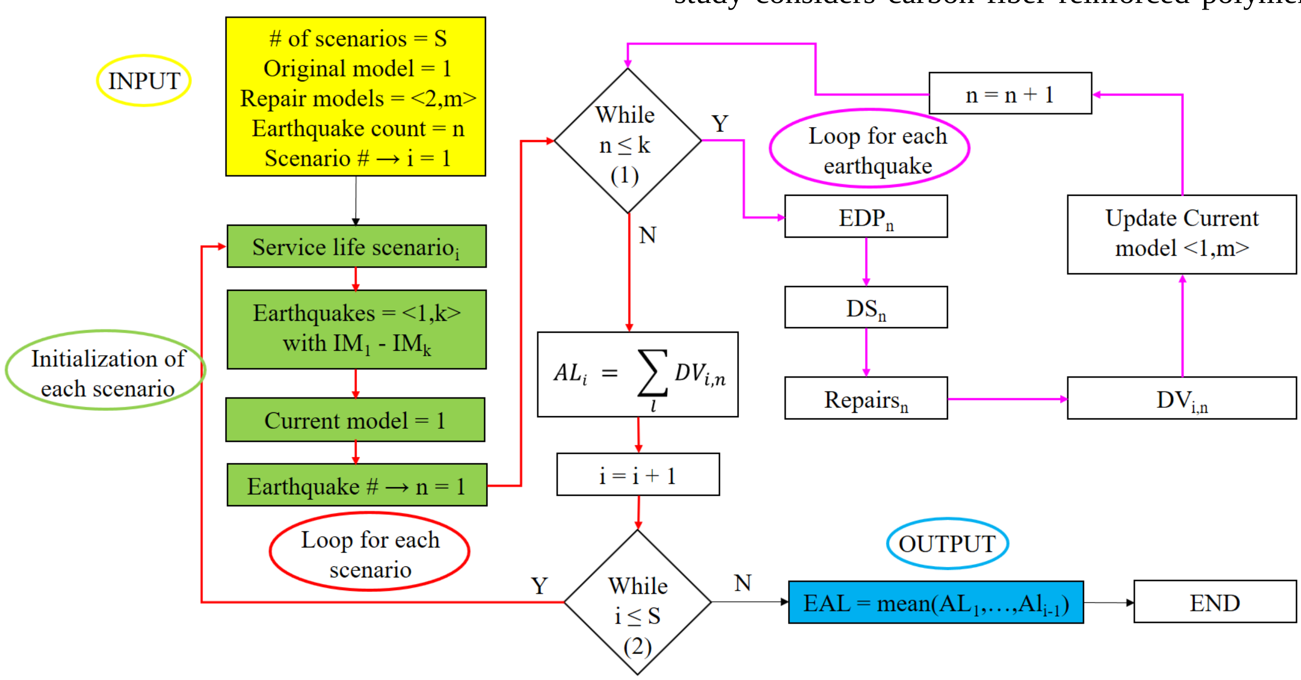

Figure 2. Proposed method's algorithm to calculate EAL

(CFRP), steel, RC, and prestressed jackets. To facilitate comparison between jackets in terms of post-repair performance, each of the jackets is designed to the minimum requirements in the standards or in the literature (Buckle et al. 2006; Caltrans 2008, 2011, 2013; Vosooghi and Saiidi 2013). For DS6, where the longitudinal steel is damaged, replacement of the column is assumed.

Table 1. Column repair methods

DS	***DS description***	***Repair***
DS1	*Flexural cracking*	*Epoxy injections*
DS2	*First spalling*	*Concrete patching*
DS3	*Spalling up to height of 1/10 of column diameter*	*Jacket type: RC, Steel, CFRP, Prestressed*
DS4	*Spalling up to height of 1/2 of column diameter*	
DS5	*Visible transverse or longitudinal rebar*	
DS6	*Buckling or fracture of longitudinal rebar*	*Column replacement*

4.3. Performance of original and repaired bridges

Dynamic analysis (in the form of an IDA) and loss assessment of the original bridge models, as well as of the repaired bridge models, was performed by Valigura et al. (2018) using the structural analysis platform *OpenSEES*. The assumption in that study was that the external jackets were applied to all column plastic hinges. The question that arises from this assumption is how to determine when to use the repaired model and when to use the model of the original bridge in estimating DV.

The ideal solution is to develop repair models for all possible combinations of external jackets applied on the column plastic hinges. However, that would be computationally expensive. Instead, the authors used the model of the original bridge for all cases where fewer than half of the plastic hinges required external repair jackets, and the repair model for all other cases. The authors deemed the approach applicable because of the high correlation between the DS between plastic hinges at different columns. This high correlation means that in the majority of cases, almost all plastic hinges either require the repair jackets or no jackets are required. The performance of the original is characterized using the cost curves in Figure 3.

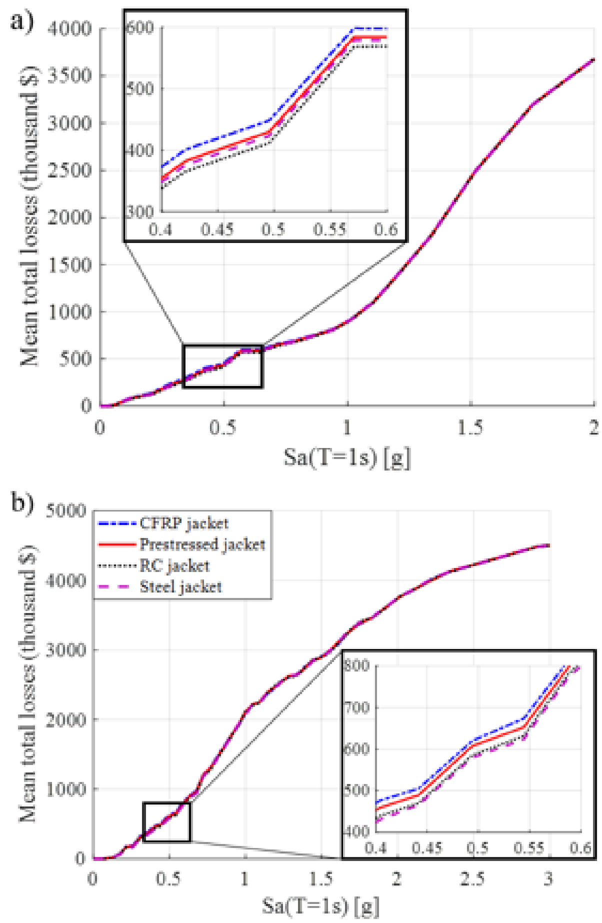

Figure 3. Cost curves for original bridges: a) PB1, and b) PB2

For Sa(T=1s) in the range of approximately 0.2g to 1.0g for both bridges, columns may be repaired by external jackets, and there is some variation based on the repair strategy selected. The zoom in Figure 3. shows that the columns repaired with CFRP jackets have the highest repair costs, while RC and steel jackets have the lowest repair costs (for PB1 and PB2, respectively).

The same approach is taken for the post-repair performance, as shown in Figure 4. For PB1, at small IM levels, the bridge repaired with steel jackets performs the best, while the original PB1 and the CFRP repaired bridge have the worst performance. In the case of PB2, the original bridge has the lowest losses for low intensities, while CFRP performs the worst.

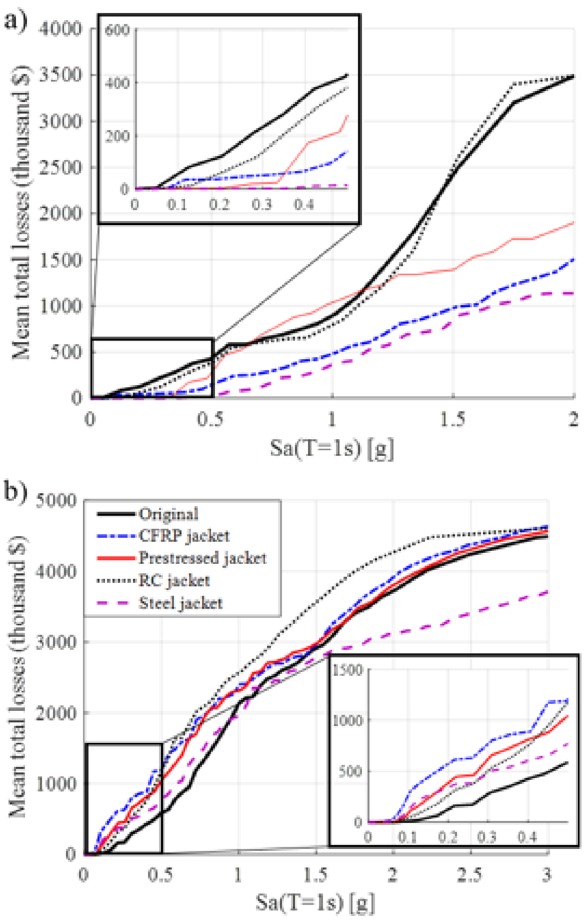

Figure 4. Post-repair cost curves of: a) PB1, and b) PB2

The different trends in the post-repair performance is due to initial stiffness of the two bridges repaired with different strategies. In the case of PB1, the stiffness of repaired bridges is enhanced for all but CFRP, while in the case of PB2, neither of the repair strategies restores the stiffness (Valigura et al. 2018). The performance at the lower intensities is important, because those events are more frequent and may often govern the annualized losses.

4.4. Site selection

The design PGA values for PB1 and PB2 are 0.49 g and 0.4 g respectively. These values have probability of occurrence of 5% in 50 years as per bridge design code in California (Caltrans 2013). As shown in Figure 5, the authors selected sites with design PGA values of 0.3, 0.4, 0.5, and 0.6 g in several urban areas (and four additional sites at universities in the selected areas) to observe if there are trends between the level of overstrength (ratio between site design and bridge design PGA value) and differences between annualized loss with and without accounting for post-repair performance. All sites were assessed for site class D (V_{S30} = 259 m/s).

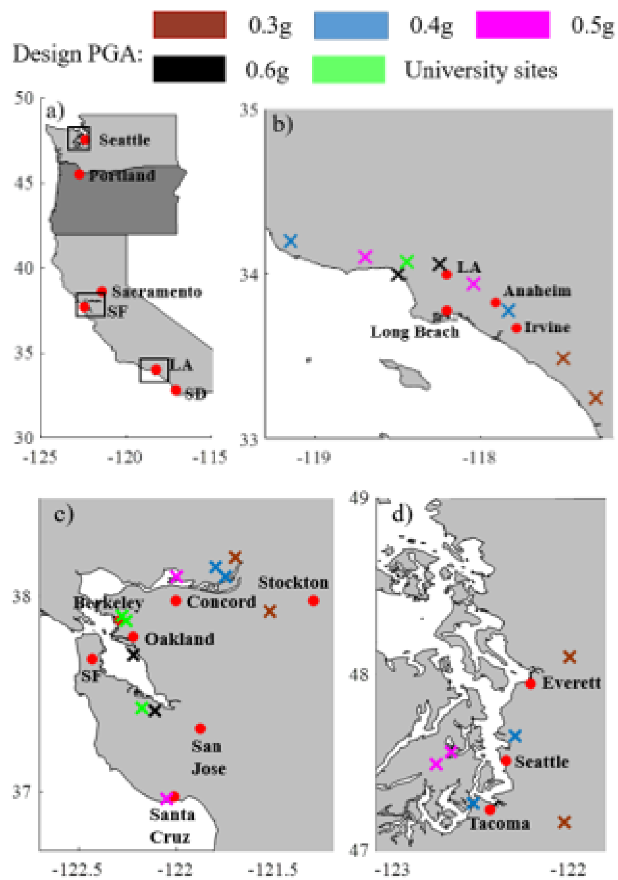

Figure 5. Sites considered showing: a) U.S. west coast, b) LA area, c) SF area, and d) Seattle area

5. RESULTS

5.1. Effects of post-repair performance

Both bridges are subjected to loss assessment using the *FEMA P-58* and the *proposed method*.

Figure 6 shows the error between the EAL value estimated using the *proposed method* and the value estimated using *FEMA P-58* for both bridges. Here, a positive value of error means that the *FEMA P-58* overestimates the EAL.

In the case of PB1, the EAL prediction of *proposed method* are lower than the prediction using *FEMA P-58*. This difference comes from the better performance of the repaired bridges than of the original bridge (Figure 4a). The magnitude of the error is striking. For example, for sites with a design PGA value of about 0.5 g, the estimates of the EAL using *FEMA P-58* can be almost double of what is predicted with the *proposed method*. The error between the predictions follows the ranking at low IM levels of the post-repair performance. The largest error is for the bridge repaired with steel jackets, because its post-repair performance at lower IMs is superior to the other repair strategies and to the original bridge. On the other hand, the lowest discrepancy is with bridges repaired with CFRP and RC jackets, where the post-repair performance is relatively close to original bridge.

The errors for PB2 are presented in Figure 6b. The trend is opposite than in the case of PB1. For PB2, *FEMA P-58* underestimates the EALs. This underestimation is associated with the lower post-repair performance of the repaired bridges than of the original bridge. Again, the error is consistent with ranking of the post-repair performance at low IMs. The largest effects are for the bridge repaired with CFRP jackets, which has the worst post-repair performance. The smallest differences between the *FEMA P-58* and the *proposed method* are when the bridge is repaired with RC jackets, which has the closest post-repair performance at low IMs to the original bridge. The error of *FEMA P-58* prediction of EAL for PB2 goes up to about 45% of the EAL predicted by *proposed method* for design value of PGA (0.4 g).

Figure 6 also illustrates the effect of location on the error. The largest errors between the two methods are for LA and SF area locations. This trend is due to high annual frequency of small IM shaking at those locations. On the other hand, Seattle area has a lower annual frequency of small IM shakings and, hence, the error is much lower there. This confirms that the EAL are mainly driven by the costs at low IM levels.

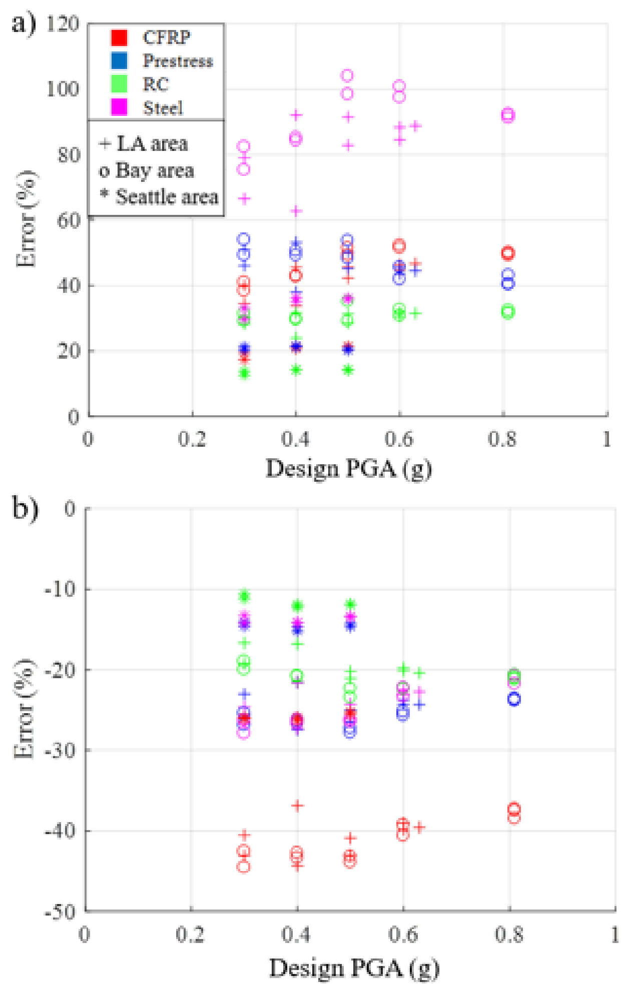

Figure 6. Error in EAL estimates between proposed method and FEMA P-58 for: a) PB1, and b) PB2

6. CONCLUSIONS

This study presents a Monte Carlo simulation based framework for calculating service life decision variables, such as expected annualized losses, considering the effect of repairs or other changes to the structure and their impact on seismic losses over the service life. This method requires simulation of many service life scenarios for the site of interest with intensity measure levels that are consistent with the seismic hazard. Each of the scenarios is separately evaluated by simulating damages to the structure (original and later repaired) during each of the earthquakes in the service life scenario. The damages are then evaluated in terms of repair costs to obtain annualized losses.

The proposed approach uses millions of service life scenarios and also needs a sophisticated dynamic analysis as an input, both of which demand significant computational time. However, this analysis can significantly change decisions about repair methods and, hence, the authors believe in its usefulness, even with the necessity of increased computational cost. Given the computational cost of IDAs, the engineer should try to limit the number of repair models to as few as possible.

The method was applied to two prototype bridges. The results showed that failing to consider changes in performance over the lifetime of the structure can significantly affect the outcome of the decision variables. The error can be in either direction, meaning that the classical PBEE methods (FEMA P-58) can be either conservative or non-conservative.

7. REFERENCES

Bisadi, V., and Padgett, J. E. (2015). "Explicit Time-Dependent Multi-Hazard Cost Analysis Based on Parameterized Demand Models for the Optimum Design of Bridge Structures." *Computer-Aided Civil and Infrastructure*, *30*(7), 541–554.

Buckle, I. G., Friedland, I., Mander, J., Martin, G., Nutt, R., and Power, M. (2006). "Seismic Retrofitting Manual for Highway Structures: Part 1 - Bridges." *Report No. FHWA-HRT-06-032*. Federal Highway Administration.

Caltrans. (2008). *Bridge Design Aids 14-3: Fiber Reinforced Polymer (FRP) Composites Column Casing Systems,* California Dept. of Transportation, Sacramento, CA.

Caltrans. (2011). *Bridge Design Aids 14-2: Steel Column Casing Design and Details,* California Dept. of Transportation, Sacramento, CA.

Caltrans. (2013). *Caltrans Seismic Design Criteria Version 1.7,* California Dept. of Transportation, Sacramento, CA.

Deierlein, G. G., Krawinkler, H., and Cornell, C. A. (2003). "A Framework for Performance-Based Earthquake Engineering." *Proc., Pacific Conference on Earthquake Engineering,* New Zealand Society for Earthquake Engineering, Christchurch, New Zealand.

FEMA. (2012). "Seismic Performance Assessment of Buildings Volume 1 - Methodology." *FEMA P-58-1,* Federal Emergency Management Agency, Washington, DC.

Ghosh, J., and Padgett, J. E. (2010). "Aging Considerations in the Development of Time-Dependent Seismic Fragility Curves." *Journal of Structural Engineering*, *136*(12), 1497–1511.

Harrington, C. C. (2016). "Advancement of Performance-Based Earthquake Engineering for RC Frame Buildings: Application to Retrofit Design and Consideration of Vertical Ground Motions." *PhD Thesis*. University of Colorado, Boulder.

He, R., Yang, Y., and Sneed, L. H. (2015). "Seismic Repair of Reinforced Concrete Bridge Columns: Review of Research Findings." *Journal of Bridge Engineering*, *20*(12).

Ketchum, M., Chang, V., and Shantz, T. (2004). "Influence of Design Ground Motion Level on Highway Bridge Costs." *Report No. 6D01*, Pacific Earthquake Eng. Research Center.

Shekhar, S., Ghosh, J., and Padgett, J. E. (2018). "Seismic Life-Cycle Cost Analysis of Ageing Highway Bridges under Chloride Exposure Conditions: Modelling and Recommendations." *Structure and Infrastructure Engineering*, *14*(7), 941–966.

Valigura, J., Liel, A. B., and Sideris, P. (2018). "Risk-Based Assessment of Seismic Repair Costs for Reinforced Concrete Bridges, Considering Competing Repair Strategies." *Journal of Bridge Engineering*, *Under Review*.

Vamvatsikos, D., and Cornell, C. A. (2002). "Incremental Dynamic Analysis." *Earthquake Engineering and Structural Dynamics*, *31*, 491–514.

Vosooghi, A., and Saiidi, M. S. (2013). "Design Guidelines for Rapid Repair of Earthquake-Damaged Circular RC Bridge Columns Using CFRP." *Journal of Bridge Engineering*, *18*(9), 827–836.