Risk-based seismic life-cycle cost–benefit (LCC-B) analysis for bridge retrofit assessment

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Abstract

Bridges constitute key elements of the nation’s infrastructure and are subjected to considerable threats from natural hazards including seismic events. A range of potential bridge retrofit measures may be used to mitigate seismic damage in deficient bridges, and help to avoid associated social and economic losses. However, since resources are often limited for investment in seismic upgrade, particularly in regions of large but infrequent events, a risk-based approach for evaluating and comparing the cost effectiveness of different mitigation strategies is warranted. This paper illustrates a method for evaluating the best retrofit measures for non-seismically designed bridges based on life-cycle costs and cost–benefit analysis. The approach integrates probabilistic seismic hazard models, fragility of as-built and retrofitted bridges for a range of damage states, and associated costs of damage and retrofit. The emphasis on life-time performance and benefits, as opposed to initial retrofit cost alone, not only permits risk-wise investment, but also helps to align upgrade actions with highway agency missions for sustainable infrastructure. An application of the seismic life-cycle cost–benefit analysis is conducted for four representative bridges of different classes, and seven different retrofit options ranging from the use of seat extenders, to isolation bearings, to steel jackets. The same bridges are evaluated located at three sites of varying seismicity: Caruthersville, Missouri; Charleston, South Carolina; and Los Angeles, California. A summary of the proposed optimal retrofit measures for the case-study bridges and locations is presented. The results show that not only do the magnitude of the expected losses and resulting retrofit cost–benefit differ by location, but the most cost-effective retrofit for a particular bridge may vary as well due to local seismic hazard characteristics and the effect of retrofit at different damage levels.

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1. Introduction

Consideration of the socio-economic impacts of natural hazards such as earthquakes, hurricanes, windstorms, and floods has historically motivated improvement in new design or rehabilitation of existing structures. Bridges, as key elements of the transportation infrastructure, have been found to be a critical vulnerable component in past earthquakes such as Loma Prieta (1989), Northridge (1994), Hanshin-Awaji (Kobe) (1995), and Izmit (1999) [1–3]. In addition to direct losses due to bridge damage in such events, indirect social and economic impacts can be attributed to inhibited emergency response efforts, increased travel time in the transportation network, business disruption, among others. While there have been widespread bridge seismic retrofit programs adopted in some regions of the United States to mitigate such consequences [4], questions still remain as to the most cost-effective upgrades and an approach for their assessment and selection, particularly in regions of large but infrequent events.

Highway agencies are constricted by the amount of resources available for immediate investment in seismic upgrade, yet the life-time expected costs and benefits associated with the bridge performance are of increasing interest. This is due to the importance of weighing up-front expenses versus long term risks and benefits, as well as the interest in aligning upgrade actions with highway agency missions for sustainable infrastructure [5,6], which necessitates life-time expected costs to be considered in addition to increasing public safety. Consequently, life-cycle cost (LCC) modeling for bridges has gained widespread interest in recent years. A rational approach to life-cycle cost analysis considers all costs incurred over the structure’s life-time ranging from initial construction, to maintenance and repair, to deconstruction, among others, expressed in present day dollars [7]. Although most studies evaluating bridge life-cycle costs have emphasized assessment of maintenance or repair strategies [9–11], past work by other researchers has noted the important contribution of extreme events to a structure’s life-cycle costs [8]. For example, Wen [12]
emphasized use of the principle of minimum life-cycle cost criteria to achieve the desired reliability for performance-based design of steel buildings under multiple threats scenarios. The use of economic cost–benefit ratios to attain the optimal reliability levels in reinforced concrete buildings located in seismic zones has also been proposed [13]. Furthermore, Takahashi et al. [14] demonstrated the use of Monte Carlo simulation in life-cycle costing of a nine story building with damping devices while considering a non-stationary earthquake occurrence model.

Pertaining to highway bridges, Sato et al. [15] proposed a framework to prioritize bridges in need of seismic retrofit with different service lives and different levels of importance on the basis of life-cycle cost. Furuta et al. [16] used multi-objective optimization techniques to evaluate bridge maintenance actions that minimize the life-cycle cost associated with both maintenance and seismic risk to a Japanese bridge pier. Cho et al. [17] has compared the effectiveness of retrofit measures such as elastomeric bearings and restrainer cables on the basis of minimum life-cycle cost, primarily based on first order reliability methods for probabilistic seismic damage analysis and the use of 2-dimensional analytical models of steel bridges in Korea. While their study does not account for the relative differences in life-cycle cost estimation stemming from different bridge types or hazard locations, which may potentially affect the selection of optimal retrofit measure, it illustrates the opportunity to conduct bridge retrofit assessment using life-time financial objective measures. Analysis of a wide range of retrofit measures for different classes of multi-span highway bridges is yet to be fully explored through a rigorous risk-based approach to cost–benefit analysis, or using input fragility models rooted in 3-dimensional non-linear dynamic analysis. In the present study, the effect of various critical parameters such as change in hazard exposure, bridge type (e.g. multi-span continuous or multi-span simply supported steel girder), or retrofit measure on the seismic life-cycle cost and cost–benefit ratio is explored to provide new insight on viable retrofit measures.

This paper examines the cost effectiveness of range of different seismic retrofits for four typical classes of non-seismically designed bridges. A risk-based approach to assess the seismic life-cycle costs and benefits of retrofitting bridges is presented to evaluate the relative upfront costs and long term benefits of seismic retrofit with various measures. A case study application is conducted to evaluate retrofit options for representative non-seismically designed multiple-span steel and concrete girder overpass bridges, such as those common in the Central and Eastern United States. The illustration considers seven retrofit options examined under different hazard exposures, given the seismic hazard characteristics of Charleston, SC and Caruthersville, MO as compared to Los Angeles, CA. A better understanding of the cost effectiveness of mitigating seismic risk to bridges under uncertainty is provided, by explicitly incorporating probabilistic modeling of the hazard occurrence, fragility models of existing and retrofitted bridge performance, and seismic LCC modeling considering life-time exposure to seismic hazards.

2. Seismic life-cycle cost model

A risk-based seismic life-cycle cost–benefit (LCC-B) analysis as proposed herein requires integration of information regarding the seismic hazard, bridge performance, costs associated with bridge damage, and costs of retrofit. The life-cycle costing integrated in the seismic retrofit cost–benefit analysis emphasizes the potential costs due to seismic damage and does not address bridge maintenance costs within the scope of this study. This section outlines the seismic LCC-B modeling framework, which will subsequently be applied in forthcoming sections for bridge retrofit evaluation.

2.1. Seismic life-cycle costing

The uncertainty in seismic ground shaking is often described in terms of the probability distribution of an adequate earthquake intensity measure, such as peak ground acceleration (PGA), over a given period, typically one year as presented by the USGS [18]. The seismic hazard is frequently presented as a mean seismic hazard curve, \( H(a) \), which offers the annual probability of exceeding specified levels of PGA, \( a \), as shown below:

\[
H(a) = P[PGA > a]
\]  

(1)

The seismic hazard is convolved with bridge fragility models in order to evaluate the annual probability of exceeding different levels of damage. Bridge fragility curves provide a quantitative measure of the vulnerability of bridges in their existing or retrofitted condition, or susceptibility to different levels of damage, termed as damage states. The fragility curves used in this study for slight, moderate, extensive, and complete damage states will be discussed further in following sections. The seismic fragility curves are estimated as a lognormal distribution, indicating the probability of meeting or exceeding different levels of damage (DS) conditioned upon the peak ground acceleration:

\[
P[DS > a|PGA = a] = \Phi \left( \frac{\ln(PGA) - \ln(\text{med}_{sys})}{\sigma_{sys}} \right)
\]  

(2)

where \( med_{sys} \) is the median value of the fragility of the bridge system in units of PGA, \( \sigma_{sys} \) is the logarithmic standard deviation of the system fragility, and \( \Phi(\cdot) \) is the standard normal cumulative distribution function. The intensity measure of peak ground acceleration is used in the seismic life-cycle cost formulation presented due to the identification of PGA as an optimal intensity measure for fragility analysis of the types of bridge analyzed in the case study as further discussed in the next section. However, the same approach for LCC-B analysis can be conducted through appropriate substitution of other hazard intensity measures. As previously noted, this seismic fragility can be convolved with the seismic hazard in order to assess the annual probability of exceeding each damage state:

\[
P_{ij} = \int P[DS > a|PGA = a] \frac{|dH(a)|}{da} da
\]  

(3)

As illustrated in previous studies [19–21] earthquake occurrence is assumed to follow a Poisson process in the present analysis of the expected life-cycle cost. It is acknowledged that in certain regions characterized by large infrequent earthquakes non-Poissonian time dependent models may represent earthquake occurrences more accurately [22–24], and that the Poisson assumption herein is adopted for the sake of modeling simplicity. Under such assumptions, the expected value of the life-cycle costs due to seismic damage in present day dollars can be expressed as follows:

\[
E[LCC] = \frac{1}{2T} \left( 1 - e^{-\lambda T} \right) \sum_{j=1}^{4} \left[ C_j \left( \ln(1 - P_{Tj}) - \ln(1 - P_{Tj+1}) \right) \right]
\]  

(4)

where \( j \) is the damage state, \( T \) is the remaining service life of the bridge, \( C_j \) is the cost associated with damage state \( j \), and \( P_{Tj} \) is the \( T \)-year probability of exceeding damage state \( j \), estimated as:

\[
P_{Tj} = 1 - (1 - P_{Rj})^T
\]  

(5)

An inflation adjusted discount ratio, \( \alpha \), is used for converting future costs into present values, which is based on the idea that costs and benefits incurred now are worth more to the decision makers. The life-cycle cost in Eq. (4) does not include the initial cost of construction or maintenance costs of the bridge, but rather gives an estimate of the cost incurred due to damage from life-time seismic exposure. Inherent in the equation is the assumption that regardless of the
damage level, the bridge is restored to its original state after each seismic hazard occurrence. Additionally, the capacity of the structural components against seismic loading is assumed to remain constant over the bridge's remaining life (i.e. its seismic fragility unchanged in time). Hence the T-year probability of exceeding each damage state can be found as given in Eq. (5) with the damage state exceedance probabilities at each point in time assumed to be identical and independent.

The LCC model presented provides an expression of the expected value of losses due to life-time exposure to seismic hazards, whereby the use of alternative fragility curves for different retrofitted conditions enables a comparison of the economic impacts of retrofit.

2.2. Risk-based life-cycle cost–benefit analysis

A cost–benefit analysis is a well known tool for comparing alternative investments, though is frequently used in scenario driven applications. The model presented above, however, presents an opportunity to assess the cost effectiveness of bridge retrofit through a risk-based framework. The fragility curves coupled with probabilistic seismic hazard curves and costs estimated from bridge damage allow for assessment of economic losses with and without the retrofit in place. The benefit of a particular retrofit, \( r \), is evaluated as the difference between the expected present value of the losses without retrofit, \( LCC_{\text{as-built}} \), and the present value of the losses with retrofit, \( LCC_{\text{r}} \), as shown in:

\[
Benefit = E[LCC_{\text{as-built}}] - E[LCC_{\text{r}}]
\]

(6)

It is noted that the present study implicitly assumes the same maintenance cost for the as-built or retrofitted bridge. The cost–benefit ratio (CBR) for a particular retrofit is then assessed as the ratio between the net present value of the investment in retrofit \( Benefit \), and the initial cost of the retrofit \( Cost\):

\[
CBR = \frac{Benefit}{Cost}
\]

(7)

The CBR is a measure of the financial return for each dollar invested in the seismic retrofit under consideration. A CBR greater than one indicates a positive return on investment, and the retrofit with the largest CBR has a larger expected savings in losses over the remaining life, per dollar invested in mitigation. It is noted that a CBR less than one may still be favorable in certain cases due to non-monetary benefits of retrofit and social responsibility, such as loss of life avoided.

3. Application for representative bridges

The framework for risk-based seismic LCC-B analysis presented above is applied in a case study assessment of retrofit evaluation for representative non-seismically designed highway bridges. The following sections describe the input models and assumptions for the comparative assessment. While recent studies have been conducted to support risk-based decision making for other structural systems, such as steel moment frame buildings \cite{19,25}, reinforced concrete buildings \cite{26}, or wood-frame residential construction \cite{27}, such an assessment has not been conducted to evaluate a range of retrofit options for bridges. The bridge types and retrofits considered below are primarily targeted at non-seismically designed highway overpass bridges, such as those typical of Central and Eastern US (CEUS) bridge inventories. Bridge owners in this region are particularly challenged to make risk-informed decisions due to the nature of low probability high consequence events, and the need to manage uncertainties associated with seismic performance given the limited budgets available for seismic upgrade.

3.1. Bridge types and retrofits considered

Four different bridge types are considered in the analysis, including multiple span continuous steel girder (MSC steel), multiple span simply supported steel girder (MSSS steel), multiple span continuous concrete girder (MSC concrete), and multiple span simply supported concrete girder (MSSS concrete) bridges. All four classes are typically zero skew three span highway overpass bridges supported on multi-column bents. The non-seismic detailing considered is common of CEUS bridge inventories, such as having approximately 1% longitudinal reinforcement in poorly confined reinforced concrete columns, short seat widths, and vulnerable high-type steel fixed and rocker bearings in the steel bridges \cite{28}. The representative bridges selected from each bridge class, including dimensions and dynamic periods, used in the present analysis are given in Table 1. Past studies have illustrated that these bridges types are susceptible to damage to fixed and expansion bearings leading to potential span unseating, excessive ductility demands on non-seismically detailed columns, pounding between adjacent spans and between the deck and abutment, among other issues that may lead to potential inhibition of post-event bridge functionality and the need for repair or replacement \cite{28,29}.

In order to address the potential vulnerabilities of the multi-span bridges, five different retrofit measures are considered in this study: steel column jackets, elastomeric isolation bearings, steel restrainer cables, seat extenders, and transverse shear keys. The combined use of seat extenders and shear keys, or restrainer cables and shear keys is also considered. Fig. 1 illustrates the seismic retrofit measures considered and their location in a typical three span overcrossing bridge. Full height steel jackets are considered, in which 10 mm thick A36 steel casings provide increased compressive strength and ultimate strain of confined concrete and enhance the ductility capacity of the columns. The elastomeric bearings replace existing fixed and expansion bearings to provide a form of isolation, shifting the natural period of the bridge out of the region of dominant earthquake energy and decoupling the superstructure from the substructure. Longitudinal steel restrainer cables are provided at the deck–abutment interface, as well as between the deck and bent beam of simply supported bridges. They limit the relative hinge displacement of the system and aim to prevent undesirable girder unseating which would lead to the collapse of a bridge span. Furthermore, since many bridges have relatively short seat widths

<table>
<thead>
<tr>
<th>Bridge type</th>
<th>End span length (m)</th>
<th>Mid span length (m)</th>
<th>Deck width (m)</th>
<th>Column height (m)</th>
<th>No. of girders</th>
<th>( T_1 ) (s)</th>
<th>( T_2 ) (s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>MSC concrete</td>
<td>12.2</td>
<td>22.6</td>
<td>12.8</td>
<td>3.93</td>
<td>8</td>
<td>0.59</td>
<td>0.49</td>
</tr>
<tr>
<td>MSSS concrete</td>
<td>12.2</td>
<td>13.4</td>
<td>10.4</td>
<td>4.23</td>
<td>5</td>
<td>0.46</td>
<td>0.37</td>
</tr>
<tr>
<td>MSC steel</td>
<td>22.3</td>
<td>22.3</td>
<td>10.3</td>
<td>4.08</td>
<td>5</td>
<td>0.33</td>
<td>0.23</td>
</tr>
<tr>
<td>MSSS steel</td>
<td>12.2</td>
<td>13.7</td>
<td>10.5</td>
<td>4.02</td>
<td>5</td>
<td>0.25</td>
<td>0.21</td>
</tr>
</tbody>
</table>

\( T_1 \) and \( T_2 \) = first and second period (longitudinal and transverse, respectively, for all bridge types).
and unseating is of great concern during a seismic event, the use of seat extenders serve as a failsafe to deck collapse by providing an extended support length. Being the simplest and one of the least expensive means of preventing unseating, they allow the superstructure to float over the substructure and increase the capacity of the bridge to sustain longitudinal displacement. The final form of retrofit considered in this analysis are shear keys which often take the form of reinforced concrete blocks doweled into bent beams or at the abutment. They serve to restrain the deck motion when a bridge is excited in the transverse direction and facilitate shear force transfer to the substructure. A detailed discussion of each retrofit measure, typical design assumptions and further details for the bridge types considered may be found elsewhere [29]. For the bridge retrofit assessments presented, the remaining life of the bridge, $T$, is assumed to be 50 years for all of the bridges as a base case for comparison. The sensitivity of the results to this parameter is later discussed.

3.2. Seismic LCC-B input models and case study assumptions

3.2.1. Seismic hazard

In order to evaluate the seismic LCC and cost effectiveness of various retrofit measures for the four bridges considered, the risk-based model presented in Section 2 is applied. Given that the bridge classes considered are most typical to the CEUS, two locations are selected out of this region and a third for comparison on the West coast. The LCC-B is conducted considering the same four bridges and retrofits, but considering the following sites: Charleston, South Carolina; Caruthersville, Missouri; and Los Angeles, California. These locations represent unique seismic hazard characteristics in order to evaluate their impacts on anticipated life-time seismic costs and retrofit impacts. The location specific seismic hazard curves from the USGS [18] are presented in Fig. 2 for the three sites. The relative flatness of the Caruthersville and Charleston hazard curves compared to the Los Angeles hazard curve is noted, and characteristic of the difference in West Coast versus Central and Eastern US hazards. The implications that the higher potential for large infrequent events in the CEUS may have on cost-effective retrofit selection will be investigated.

3.2.2. Fragility curves

The relative vulnerability of bridges in their as-built and retrofitted state are represented though their fragility models, as discussed above. Each fragility curve is characterized by a median value of ground motion PGA and an associated dispersion factor, or lognormal standard deviation, for integration in the seismic LCC-B model. Peak ground acceleration is used as the intensity measure for conditioning the fragility curves in this work due to the findings of previous studies that have identified PGA as the optimal IM for the types of non-seismically designed CEUS bridges considered [30]. Details of the methodology used for developing the fragility curves used in this study can be found elsewhere [31]. In the approach presented, fragility curves are developed using 3-dimensional non-linear time history analysis for probabilistic seismic demand modeling. The bridge system fragility curves are developed by comparing the demand models to capacity estimates for multiple vulnerable components, including the columns, fixed and expansion bearings, active, passive and transverse...
response of the abutments. Each damage state (slight, moderate, extensive, and complete) is related to an anticipated level of post-event functionality, ranging from temporary restrictions for one day to complete closure beyond 30 days. Uncertainty in the ground motion, material properties, component modeling, demand, and capacity are all considered in the analysis. A series system assumption is applied in the adopted methodology, implying that failure (or damage) of any one component is indicative of overall damage to the system. Furthermore, the bridge system fragility analysis considers the correlation between component damage. However, rather than being developed for general portfolios of bridges, the fragility curves used in this study are developed for the particular bridge geometry presented for the case study, in order to enable the bridge retrofit LCC-B analysis using geometry-specific cost estimates.

Table 2 presents the fragility models for the four bridge types in their as-built state. The fragility curves are scaled to account for the various retrofit measures using the median value modification factors previously developed for each bridge type [31]. The modification factors are presented in Appendix A for ease of reference. This scaling of the fragility curve reflects the impact of retrofit on the bridge system fragility, noting that the impact of retrofit differs by bridge type and damage state. Applying the modification factors presented in [31] introduces the same assumptions regarding the retrofit measures and their design used in deriving the scaling factors into the current study of LCC-B. Thus the finding of this study are relevant for retrofit assumptions that were adopted based typical CEUS practice. For example the steel jackets were considered to be full column height jackets; restrainers designed to carry half the superstructure weight; elastomeric bearings designed to provide approximately a 2.5-fold increase in fundamental period in the longitudinal direction; shear keys designed to limit column force transfer to half of their shear strength; and seat extenders provide an additional 152 mm of support length [31]. Sample fragility curves for the MSC concrete bridge corresponding to the moderate and complete damage states are presented in Fig. 3 for the as-built bridge compared to various retrofit measures. As illustrated by the plot, and inferred from the median value scaling factors in the tables presented in the Appendix, the elastomeric bearings reduce the vulnerability of the bridge system at the moderate damage state while the seat extenders are more effective at the complete damage state.

3.2.3. Cost estimates

In addition to the seismic hazard curves and fragility models, the present value of total losses presented in Eq. (4) is also a function of the remaining life, discount factor, and cost associate with repair of each damage state. For the base case retrofit assessment, a remaining life, \( T \), of 50 years is assumed for all of the bridges. The inflation adjusted discount ratio, \( \alpha \), is taken as 3%. Costs associated with repair from each damage state, \( C_i \), are estimated as a fraction of the replacement cost using the repair cost ratios estimated by Basoz and Mander [32]. Table 3 presents the best mean repair cost ratios for each damage state assessed in the fragility analysis. The replacement costs for each bridge are estimated based on regional bridge construction costs and presented in Table 4. These replacement costs are estimated using replacement costs per deck area determined by historic construction data from local departments of transportation [33], and the bridge specific deck area readily assessed from Table 1. The losses estimated as the fraction of replacement cost include only direct losses due to structural damage. However, the indirect losses due to increased travel time from bridge damage have been found in past studies to be on the order of 5–20 times larger than the direct losses [34]. As a simple ap-

Table 2

<table>
<thead>
<tr>
<th>Bridge type</th>
<th>Slight</th>
<th>Moderate</th>
<th>Extensive</th>
<th>Complete</th>
</tr>
</thead>
<tbody>
<tr>
<td>MSC concrete</td>
<td>0.15</td>
<td>0.81</td>
<td>0.58</td>
<td>0.66</td>
</tr>
<tr>
<td>MSSS concrete</td>
<td>0.21</td>
<td>0.69</td>
<td>0.61</td>
<td>0.60</td>
</tr>
<tr>
<td>MSC steel</td>
<td>0.22</td>
<td>0.53</td>
<td>0.43</td>
<td>0.51</td>
</tr>
<tr>
<td>MSSS steel</td>
<td>0.27</td>
<td>0.45</td>
<td>0.52</td>
<td>0.41</td>
</tr>
</tbody>
</table>

*Fig. 3. Example as-built and retrofitted fragility curves for the case study MSC concrete bridge at the moderate and complete damage states.*
proach to acknowledge and account for these indirect losses, the total cost of losses associated with each damage state is assumed to be 13 times larger than the estimated repair costs. Finally, in order to assess the cost–benefit of each retrofit as shown in Eqs. (6), retrofit cost estimates are required in addition to the benefits of avoided losses due to retrofit. Cost estimates are presented in Table 5 for each retrofit measure based on a review of CEUS retrofit practice and regional retrofit cost estimates [29,35]. For example, the cost of isolation with elastomeric bearings is estimated to be 5% of the bridge replacement cost, while the restrainer cables are estimated to cost $705/restrainer cable. It is noted that these input models and estimates form the basis of the illustrative seismic LCC-B case study presented herein, but that the results are highly sensitive to the cost estimates, which should be refined for alternate bridge specific applications.

4. Results and discussion of LCC-B analysis of retrofit of representative bridges

A seismic life-cycle cost analysis and subsequent retrofit cost–benefit analysis is conducted for the aforementioned case study based on the LCC-B model presented, location specific hazard curves, fragility curves for the as-built and retrofitted bridges, cost estimates, and assumptions presented in Section 3. To demonstrate the analysis method and results, a detailed discussion of the retrofit evaluation of one bridge type (multi-span continuous concrete) for a particular location (Caruthersville, Missouri) is presented in the subsequent sections. This is followed by the presentation of the overall analysis results for all three locations, four bridge types, and seven retrofit options. For simplification and easy reference, a list of abbreviations used in the graphs in this section is given in Table 6.

4.1. LCC and cost–benefit analysis: MSC concrete bridge in Caruthersville, Missouri

The expected life-cycle costs are evaluated for the MSC concrete bridge in its as-built and retrofitted condition, considering that it is located in Caruthersville, MO near the New Madrid seismic zone. Estimating the life-cycle cost for a particular bridge requires the knowledge of the probability of exceeding each damage state (refer to Eqs. (1)–(5)) and the cost associated with the different damage states as previously described. The 50-year probability of exceedance for each of the four damage states is estimated following a convolution of the hazard curve for Caruthersville and fragility curves for the MSC concrete bridge in its as-built and retrofitted conditions. Table 7 presents the 50-year exceedance probabilities for the as-built bridge. The repair cost for each damage state are presented in the Table as well, estimated from the repair cost ratios (Table 3) and replacement cost for the bridge of interest, which was found to be $438,237. The contribution of each damage state to the total expected life-cycle cost in present day dollars is shown in Table 7, prior to the summation of total expected LCC indicated in Eq. (4). While the complete damage state has the lowest 50-year exceedance probability, it contributes a majority of the expected LCC. It is also noted, however, that the contrast in damage state exceedance probability is more dramatic between the slight and complete damage states for the bridge sited in Los Angeles than for the present example in Caruthersville due to the slope of the hazard curve.

A similar approach as taken to find the expected life-cycle cost for the MSC concrete bridge with each retrofit measure: steel jackets, elastomeric isolation bearings, restrainer cables, seat extenders, seat extenders, and the combined use of seat extenders and shear keys, or restrainer cables and shear keys. Table 8 presents the expected LCC for each retrofit due to 50-year seismic exposure converted to present day dollars via Eq. (4). The change in LCC from the as-built case provides new insight as to the life-time losses avoided through retrofit. While traditional practice may emphasize comparison of the upfront investment alone, this approach provides an opportunity to evaluate the life-time benefits from risk mitigation. For example, for the MSC concrete bridge located in Caruthersville, the expected value of the seismic life-cycle costs decreases by 28% using the elastomeric bearing retrofit. This reduction is attributed to the reduction in the fragility at each damage state due to isolation. The seat extenders, which improve only the complete damage state, reduce the expected LCC by 17% due to the relative contribution of the complete damage state to the total LCC. Additionally, the standard deviations of the estimated life-cycle costs for the as-built bridge and for the bridge with different retrofit measures employed are also presented. It can be seen that besides being most successful in reducing the life-cycle cost, the elastomeric bearings also result in least deviations from the expected value. However, the final retrofit selection depends on both the expected life-cycle cost and the cost of retrofit in the form of a cost–benefit ratio analysis.

Table 8 shows the costs and benefits of each retrofit measure, in terms of avoided losses, used to find the cost–benefit ratio (CBR) from Eqs. (6) and (7). The cost–benefit ratio provides a quick screening of which retrofit is the most economically beneficial. A comparison of the CBR reveals that the use of seat extenders is the most cost-effective retrofit measure for the MSC concrete bridge in Caruthersville, with a CBR of 1.7. This indicates that for every dollar invested the seismic retrofit, the realized expected return is an estimated $1.7. While the study presented herein targets maximum CBR emphasizing use of expected LCC, the potential importance of considering the variance in life-cycle cost and anticipated benefit of retrofit is also noted. Additional studies revealed that comparison of the different retrofit measures at the respective upper and lower limits of the CBR values, estimated as the expected life-cycle cost ±1 standard deviation for the as-built and retrofitted bridge, leads to the same retrofit selection for this study. Care should be taken, however, in selecting the most economic retrofit measure when the expected life-cycle costs are associated with high variances and differ significantly among the as-built bridge and various retrofit options.
4.2. Summary of LCC and cost–benefit analysis: all bridge types in all three locations considered

Following the same steps as explained in the previous section, life-cycle cost and cost–benefit ratio analysis is carried out for the four different bridge types with each retrofit measure at three different locations across the US. Fig. 4 shows the life-cycle cost estimates for all bridge types at the three locations, while the cost–benefit ratios are presented in Fig. 5. The magnitude of the expected losses, which is naturally a function of the local seismic hazard potential, varies across the three US sites. A comparison of the LCC for the as-built and retrofitted bridges reveals that the measure yielding the greatest reduction in expected losses differs by bridge type. For example, in Charleston, the LCC is reduced most by using the steel jackets for the MSSS concrete bridge; the elastomeric bearings followed by the seat extenders or steel jackets for both the MSSS steel and MSC concrete bridges. Additionally, the retrofit that yields the greatest reduction in life-cycle cost may also differ with location. For example, for the MSSS concrete bridge the steel jackets reduce the LCC the most for the Caruthersville and Charleston locations, while the use of the elastomeric bearings reduces the LCC the most for Los Angeles. This can be attributed to the higher potential for frequent modest earthquakes in Los Angeles exhibited by the hazard curve (Fig. 2), potentially resulting in slight or moderate bridge damage, which is most effectively countered by the elastomeric bearings for this bridge.

Moreover, the bar charts in Fig. 5 reveal which retrofit measure is the most cost-effective, having the highest cost–benefit ratio ($\textit{CBR}$). It is noted that a modestly effective retrofit measure in reducing damage potential and life-cycle costs, such as seat extenders, may have the highest cost–benefit ratio due to the relatively low initial investment in retrofit. However, in some cases more costly initial investments are followed by the seat extenders or steel jackets for both the MSSS steel and MSC concrete bridges. Additionally, the retrofit that yields the greatest reduction in life-cycle cost may also differ with location. For example, for the MSSS concrete bridge the steel jackets reduce the LCC the most for the Caruthersville and Charleston locations, while the use of the elastomeric bearings reduces the LCC the most for Los Angeles. This can be attributed to the higher potential for frequent modest earthquakes in Los Angeles exhibited by the hazard curve (Fig. 2), potentially resulting in slight or moderate bridge damage, which is most effectively countered by the elastomeric bearings for this bridge.

4.2. Summary of LCC and cost–benefit analysis: all bridge types in all three locations considered

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Moreover, the bar charts in Fig. 5 reveal which retrofit measure is the most cost-effective, having the highest CBR. It is noted that a modestly effective retrofit measure in reducing damage potential and life-cycle costs, such as seat extenders, may have the highest cost–benefit ratio due to the relatively low initial investment in retrofit. However, in some cases more costly initial investments
in retrofit, such as the use of elastomeric bearings for MSSS steel bridge in all locations, have the largest ultimate return on investment due to the life-time benefits in expected loss reduction. For some bridge types, the same retrofit measure is the most cost-effective regardless of location of the bridge. For example, the re-strainer cables for the MSC Steel bridge has the highest cost–benefit ratio for every site, though the increase in the seismic hazard results in an increase in CBR<sub>RC</sub> of 1.8, 3.4, and 9.8 for Charleston, Caruthersville, and Los Angeles, respectively.

It is interesting to note, however, that while seat extenders result in the highest cost–benefit ratio for the MSSS concrete bridge in both Caruthersville and Charleston, the use of elastomeric bearings for the same bridge in Los Angeles, California proves to be a more beneficial investment. This can be attributed to the effectiveness of the seat extenders at the complete damage state, which is a larger relative contributor to the total expected LCC in the CEUS hazard zones than the West Coast example, which has larger short return period events that may cause intermediate levels of damage mitigated by the elastomeric bearing retrofit but not affected by the seat extenders. This clearly proves the importance of considering the combined effects of a particular location’s seismic hazard characteristics, the retrofitted bridge fragility, as well as cost of retrofit. These findings are further summarized in Table 9, which lists the retrofit measure found to be most cost effective for the case-study bridges. It is noted that these results are specific to the bridge types and geometry, as well as life-cycle cost–benefit model inputs and assumptions of the case study presented herein.

For a particular bridge type and location, while some economic benefits do not outweigh the costs of retrofit for any measure evaluated (i.e., CBR < 1 as shown in Fig. 5), this does not imply that retrofit is not warranted. An example of this is the MSSS steel girder bridge in Charleston, for which all cost–benefit ratios are less than one. As previously noted, indirect social impacts such as public safety threats and economic impacts such as business interruption are not addressed in this study, and warrant further investigation. There may be investments with a CBR less than one that are favorable because of non-monetary benefits and social responsibility, such as loss of life avoided, particularly given the lack of warning prior to an earthquake event. An assessment of the fragility curves themselves reveals the likelihood of achieving complete damage of the bridges in potentially large but infrequent events, particularly for the more vulnerable MSC steel followed by MSSS steel girder bridges. In addition, the results of the case study presented consider a 50 year remaining life of the structure. While the full results are not presented herein for brevity, alternate remaining life-times were also examined, including 25 and 75 years. While the relative cost effectiveness of the different retrofit measures does not tend to change considerably with remaining life, the magnitude of the cost–benefit of each measure is affected as anticipated. For example, the CBR<sub>RC</sub> of the elastomeric bearings is 0.44 for the MSSS steel girder bridge in Charleston with a remaining life of 50 years, and increased to 0.51 if the bridge is anticipated to remain in service for another 75 years. This may be of particular importance given the extended service life expected of many of our nation’s bridges.

### Table A1
Modification factors for multi-span simply supported steel bridge adapted from Padgett and DesRoches [31].

<table>
<thead>
<tr>
<th>Retrofit measure</th>
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<th>Moderate</th>
<th>Extensive</th>
<th>Complete</th>
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### Table A2
Modification factors for multi-span continuous steel bridge adapted from Padgett and DesRoches [31].

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### Table A3
Modification factors for multi-span simply supported concrete bridge adapted from Padgett and DesRoches [31].

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### Table A4
Modification factors for multi-span continuous concrete bridge adapted from Padgett and DesRoches [31].

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### 5. Conclusions

The risk-based life-cycle cost–benefit model presented can be used to support decision making on effective upgrade of bridge infrastructure in seismic zones, particularly given limited funds for investment in seismic retrofit. This approach integrates probabilistic hazard models, fragility of as-built and retrofitted bridges across a range of damage states, and associated costs of damage and retrofit. The method provides an opportunity to move beyond current emphasis on upfront investment cost or scenario driven cost–benefit analysis, to evaluate life-time expected losses avoided through retrofit.

The case study presented evaluates four different sample bridges representative of typical non-seismically designed highway overpass bridges, including multi-span continuous and...
multi-span simply supported steel and concrete girder bridges. The effect of siting the bridges in different hazard conditions was evaluated by considering the Caruthersville, MO and Charleston, SC hazard curves in the central and eastern US, as well as a West Coast hazard curve in Los Angeles, CA. A total of seven different retrofit options were evaluated. The results indicate a natural shift in magnitude of losses and retrofit cost–benefit ratio with increasing seismic hazard. However, they also indicate that due to the relative effect of different retrofit measures at different damage states (exhibited in the fragility model), as well as the nature of the local seismic hazard, the most cost-effective bridge retrofit may differ by location. For example, a relatively cheap retrofit measure with seat extenders, which is particularly effective in mitigating complete damage, tends to be more cost-effective in CEUS locations than in the West Coast example. Additionally, the findings underscore the fact that more costly initial investments in retrofit, such as the use of isolation, may be warranted for some bridge types, such as the MSSS steel bridge. This is due to the superior effectiveness of the elastomeric bearings in reducing the fragility at all damage states and its translated impact on reducing the LCC. Selection of the most cost-effective retrofit in certain cases may also depend on the level of uncertainty in the life-cycle cost of as-built and retrofitted bridge. While the results presented herein are specific to the case study example, the risk-based LCC-B model can be extended to other bridge types, retrofit measures, or locations for screening cost-effective investments in seismic upgrade.

Acknowledgement

This study has been supported in part by the Federal Highway Administration’s Pooled Funds Study #TPF-5 (155).

Appendix A

The following tables present the fragility modification factors derived in [31] for the retrofits and bridge types considered in this study. The modification factors are used as multipliers on the median value of the lognormal distribution to account for the effects of seismic retrofit on the bridge fragility curves for each damage state. Note that the previous study found relatively little impact of retrofit on the dispersion term.

Tables A1–A4.

References