Methodology for the development of analytical fragility curves for retrofitted bridges

Jamie E. Padgett\textsuperscript{1,*},† and Reginald DesRoches\textsuperscript{2}

\textsuperscript{1}Department of Civil and Environmental Engineering, Rice University, MS-318, 6100 Main St., Houston, TX 77005, U.S.A.

\textsuperscript{2}School of Civil and Environmental Engineering, Georgia Institute of Technology, Atlanta, GA, U.S.A.

SUMMARY

Fragility curves for retrofitted bridges indicate the influence of various retrofit measures on the probability of achieving specified levels of damage. This paper presents an analytical methodology for developing fragility curves for classes of retrofitted bridge systems. The approach captures the impact of retrofit on the vulnerability of multiple components, which to date has not been adequately addressed, and results in a comparison of the system fragility before and after the application of different retrofit measures. Details presented include analytical modeling, uncertainty treatment, impact of retrofit on demand models, capacity estimates, and component and system fragility curves. The findings indicate the importance of evaluating the impact of retrofit not only on the targeted response quantity and component vulnerability but also on the overall bridge fragility. As illustrated by the case study of a retrofitted multi-span continuous (MSC) concrete girder bridge class, a given retrofit measure may have a positive impact on some components, yet no impact or a negative impact on other critical components. Consideration of the fragility based only on individual retrofitted components, without regard for the system, may lead to over-estimation or under-estimation of the impact on the bridge fragility. The proposed methodology provides an opportunity to effectively compare the fragility of the MSC concrete bridge retrofit with a range of different retrofit measures. The most effective retrofit in reducing probable damage for a given intensity is a function of the damage state of interest. Copyright © 2008 John Wiley & Sons, Ltd.

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\*Correspondence to: Jamie E. Padgett, Department of Civil and Environmental Engineering, Rice University, MS-318, 6100 Main St., Houston, TX 77005, U.S.A.
\*E-mail: Jamie.Padgett@rice.edu

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Seismic fragility curves are essential tools for assessing the vulnerability of a particular bridge, or a class of bridges, and offer a means of communicating the probability of damage over a range of potential earthquake ground motion intensities. Fragility curves for bridges in their retrofitted condition provide a number of advantages and opportunities for bridge owners. This includes offering tools to evaluate alternative retrofit measures for bridges, assess the regional risk to an inventory comprised of as-built and retrofitted structures [1–3], or perform probabilistic return on investment examinations. Regardless of the ultimate application of such tools, fragility curves for retrofitted bridges are critical pieces of the risk and reliability assessment of bridges exposed to the seismic hazard. As such, an appropriate methodology for their development is necessary.

The relationship between bridge damage and ground motion intensity has been investigated since the 1980s and early 1990s, and an array of approaches and methodologies has been employed. However, only in more recent years have the first studies on probabilistic evaluation of a retrofit’s influence on damage potential been initiated. Empirical fragility curves based on bridge damage data from the Northridge [4, 5], Loma Prieta [4], and Kobe [5, 6] earthquakes have been developed for as-built bridges. However, as a result of the limited empirical data, developing fragility curves for retrofitted bridges using this approach is unrealistic.

In the absence of adequate empirical data and in efforts to address an array of different bridge types and regions of the country while maintaining homogeneity of the data, analytical methods have often been used to develop bridge fragility curves. In these analytical studies, the structural demands and/or capacities used to evaluate failure probability are estimated through such methods as elastic spectral [7, 8], non-linear static [9–11], and non-linear time history [12–16]. There has been relatively little work evaluating the effect of retrofit on bridge fragility or assessing and proposing viable methodologies for the development of retrofitted bridge fragility curves.

Analytically derived fragility curves for two sample multi-frame concrete bridges retrofit with steel jackets were developed by Kim and Shinozuka [17] and enhancement factors proposed for application to empirically based as-built fragilities. The enhancement factors and fragility curves were based solely on the column vulnerability and impact of retrofit on this component. Karim and Yamazaki [18] have assessed the impact of isolation on bridge fragility using a simplified methodology for bridges in Japan with different pier heights.

Although the majority of studies on bridge fragility have considered only a single bridge component as representative of the overall bridge fragility (namely the columns), recent studies have recognized the contribution of multiple critical components to the vulnerability of the bridge system [19, 20]. This is particularly important for bridge types such as those found in the Central and Eastern U.S. (i.e. a multi-span simply supported girder bridge with multi-column bents), where the columns do not necessarily dominate the seismic response and vulnerability, as may be expected in other bridge types such as a multi-frame box girder bridge. Moreover, recent studies have highlighted the fact that although bridge retrofits tend to be targeted at a specific response quantity, other components of the bridge may be affected by the retrofit in either a positive or a negative way [21]. Only by capturing the retrofit’s influence on the fragility of multiple key components (columns, abutments, and bearings) can this dynamic behavior be translated into an impact on the reliability and performance of the bridge system.

To address this need, this paper presents a methodology for developing fragility curves for classes of retrofitted bridges. The approach captures the impact of retrofit on the vulnerability of multiple components, which to date has not been adequately addressed, and results in a comparison
of the fragility before and after the application of a number of different retrofit measures. The methodology presented follows a similar approach as that outlined by Nielson and DesRoches [20] for developing system level fragility curves and evaluates the application to assessing the impact of retrofit on component and system vulnerability. However, a different approach to uncertainty treatment within the fragility methodology is adopted in this work. Details are presented herein as to the level of uncertainty treatment proposed for developing fragilities for general classes of retrofitted bridges, the impact of retrofit on the probabilistic seismic demand models (PSDMs), and the associated capacity estimates for retrofitted components. Typical characteristics of the Central and Eastern U.S. bridges are considered and a case study bridge class (multi-span continuous (MSC) steel girder bridge) retrofit with a range of measures is used to illustrate the methodology.

2. OVERVIEW OF ANALYTICAL METHODOLOGY AND CASE STUDY BRIDGE CLASS

Fragility curves represent the ability of an engineered system or component, such as a bridge or column, to withstand a specified event [22]. Simply stated, the fragility defines the conditional probability of the seismic demand \( D \) placed upon the structure exceeding its capacity \( C \) for a given level of ground motion intensity (IM), as shown in the following equation:

\[
\text{Fragility} = P[D > C | IM]
\]  

(1)

Therefore, models or estimates of the seismic demand and capacity are required for fragility analysis and are accomplished in this approach by developing PSDMs and limit state capacities before and after retrofit. This methodology requires analytical modeling and simulation in order to establish the PSDMs from simulated bridge responses. Details on the analytical modeling, simulation, and uncertainty treatment used in this study will be provided in subsequent sections.

2.1. Probabilistic seismic demand models

Like Cornell et al. [23] have done for buildings, the seismic demand, \( S_d \), can be represented by a power model:

\[
S_d = a IM^b
\]  

(2)

where \( a \) and \( b \) are unknown regression coefficients. The conditional seismic demands, or PSDM, are often modeled using a lognormal distribution as shown in the following equation:

\[
P[D > d | IM] = 1 - \Phi \left( \frac{\ln(d) - \ln(S_d)}{\beta_{D|IM}} \right)
\]  

(3)

where \( \Phi(\cdot) \) is the standard normal cumulative distribution function, \( S_d \) is the median value of the seismic demand (from Equation (2)), and \( \beta_{D|IM} \) is the logarithmic standard deviation (dispersion) of the demand conditioned on the IM, which is estimated in the regression analysis. Here, the ultimate result of developing a PSDM is to provide a relationship between peak component responses and ground motion intensity through a probabilistic model.
2.2. Limit state capacities

Limit states capacities, or capacity estimates for short, are defined as a measure of the capacity of the bridge component to withstand the demands placed upon it during seismic excitation. These models are often defined based on expert judgment, experimental data, analytical methods, or combinations thereof. The as-built capacity estimates used as a part of this methodology are those presented by Nielson [24] and will be discussed in further detail in the following sections. As many other studies have assumed, the limit states capacities follow a lognormal distribution [19, 25, 26]. This provides ease of convolution with the lognormally distributed demands, where a closed-form solution for the fragility is also in lognormal form as shown in the following equation:

\[
\text{Fragility} = \Phi \left( \frac{\ln(S_d/S_c)}{\sqrt{\beta_{DIM}^2 + \beta_C^2}} \right)
\]

where \( S_c \) is the median value of the structural capacity (for the limit state) and \( \beta_C \) is its associated logarithmic standard deviation of structural capacity. The details associated with estimating these parameters will be presented throughout the paper.

2.3. Component and system fragility

The formulation presented above illustrates the fragility for a single representative component and demand parameter. In the proposed methodology, it has been deemed that multiple critical component vulnerabilities can be affected by a given retrofit measure. Therefore, a number of different component fragilities are captured and can be evaluated as shown in Equation (4). The components’ responses considered range from those associated with the columns to abutments to bearings, as listed in Table I. As discussed in Section 3, the fragility methodology considers three-dimensional analytical models and seismic response; hence, the response parameters evaluated include both longitudinal and transverse actions.

The component fragilities offer valuable insight as to the relative vulnerability of different bridge components and the impact of retrofit on their susceptibility to damage. However, the ultimate goal of the fragility analysis is to evaluate the impact of retrofit on the bridge system fragility. As a result, a systems level approach to evaluating the retrofitted bridge vulnerability is also conducted by considering the fragility for a particular damage state as the union of probabilities of each of the components being in that same state. This system failure definition is associated with the assumption that damage to any component may inhibit the functionality of the bridge system—an

<table>
<thead>
<tr>
<th>Component demand parameter</th>
<th>Abbreviation</th>
</tr>
</thead>
<tbody>
<tr>
<td>Column curvature ductility demand</td>
<td>( \mu_u )</td>
</tr>
<tr>
<td>Longitudinal expansion bearing deformation</td>
<td>Exp_Long</td>
</tr>
<tr>
<td>Transverse expansion bearing deformation</td>
<td>Exp_Tran</td>
</tr>
<tr>
<td>Longitudinal fixed bearing deformation</td>
<td>Fxd_Long</td>
</tr>
<tr>
<td>Transverse fixed bearing deformation</td>
<td>Fxd_Tran</td>
</tr>
<tr>
<td>Active abutment deformation</td>
<td>Ab_Act</td>
</tr>
<tr>
<td>Passive abutment deformation</td>
<td>Ab_Pass</td>
</tr>
<tr>
<td>Transverse abutment deformation</td>
<td>Ab_Tran</td>
</tr>
</tbody>
</table>
idealization that is strengthened by the fact that associated limit states for each component imply
a loss of anticipated traffic carrying capacity to the bridge. To accomplish this estimate of bridge
system level fragility, Nielson and DesRoches [20] proposed the use of joint probabilistic seismic
demand model (JPSDM). A similar approach is used in this work for estimating the demand on
various components and impact of retrofit on the JPSDM.

The JPSDM is developed by assessing the demands placed on each component (marginal distribu-
tion) through a regression analysis as previously discussed. This is performed in the logarithmically
transformed state, noting that Equation (2) can be rewritten as

\[ \ln(S_d) = \ln(a) + b \ln(IM) \]

The covariance matrix is developed through estimation of the correlation coefficients between the
transformed demands placed on the various retrofitted components in order to fully characterize
the conditional joint normal distribution. A Monte Carlo simulation is used to compare realizations
of the demand (using the JPSDM defined by a conditional joint normal distribution) and capacity
to calculate the probability of system failure across a range of IMs and for each damage state.
Specifically, system failure analysis by integration of the JPSDM over all failure domains [27] is
accomplished by sampling upon the demand model and the capacity model for a particular IM
(peak ground acceleration (PGA) in this work). Exceedance of the capacity sample by the demand
is tracked through an indicator function, and the probability of exceedance for the IM of interest is
estimated directly. This procedure is repeated for each level of IM for slight, moderate, extensive,
and complete damages. A regression analysis is used to estimate the lognormal parameters, which
characterize the retrofitted bridge system fragility. Further details can be found elsewhere [20, 28].
The advantage of this approach is the ability to capture the impact of retrofit on multiple vulnerable
components that are critical to the bridge’s seismic performance when estimating the bridge system
vulnerability.

2.4. Description of case study retrofitted bridge class

The methodology presented above for fragility development for retrofitted bridges will be illus-
trated along with further details on key aspects of the approach. These include uncertainty treat-
ment, analytical modeling, influence of retrofit on the demand model and capacity estimates, and
retrofitted component fragilities, among others. A particular class of retrofitted bridges will serve
as a case study—the MSC concrete girder bridge class. This is a common bridge type found in the
Central and Southeastern U.S. (CSUS). Details considered for the bridges are based on previous
studies making inferences from the national bridge inventory database [29] and a thorough review
of bridge plans for the region [30, 31].

These are typically zero-skew, non-seismically designed bridges having three spans and multiple-
column bents. The reinforced concrete columns have limited reinforcement (i.e. roughly 1% longi-
dudinal steel and widely spaced transverse ties). Alternating fixed and expansion elastomeric pads
with steel dowels serve as the bridge bearings, and continuity is provided between the deck and
girders of the three spans. Eight representative configurations of the MSC concrete girder bridge
class were identified by Nielson [24] and are listed in Table II. These will be used to represent
the range of geometries of bridges in the class, as they were developed based on the population
distributions of the CSUS bridge inventory and generated using the Latin hypercube sampling
technique. An example of MSC concrete girder bridge configuration is shown in Figure 1.
Table II. Sample representative bridge geometries for the MSC concrete girder bridge class [24].

<table>
<thead>
<tr>
<th>Bridge no.</th>
<th>Mid-span length (m)</th>
<th>Deck width (m)</th>
<th>Column height (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>39.6</td>
<td>21.2</td>
<td>4.00</td>
</tr>
<tr>
<td>2</td>
<td>22.6</td>
<td>12.8</td>
<td>3.93</td>
</tr>
<tr>
<td>3</td>
<td>18.9</td>
<td>10.8</td>
<td>6.29</td>
</tr>
<tr>
<td>4</td>
<td>21.0</td>
<td>8.00</td>
<td>3.19</td>
</tr>
<tr>
<td>5</td>
<td>26.2</td>
<td>13.1</td>
<td>4.20</td>
</tr>
<tr>
<td>6</td>
<td>10.4</td>
<td>14.1</td>
<td>3.64</td>
</tr>
<tr>
<td>7</td>
<td>14.5</td>
<td>8.70</td>
<td>4.46</td>
</tr>
<tr>
<td>8</td>
<td>15.2</td>
<td>9.80</td>
<td>5.93</td>
</tr>
</tbody>
</table>

Figure 1. Generic geometry of the multi-span continuous concrete girder bridge.

The retrofit measures addressed as a part of this study include steel restrainer cables (RCs), steel column jackets (SJ), elastomeric isolation bearings (EB), seat extenders (SEs), transverse shear keys (SKs), and combinations of the above. The measures considered are common in the CSUS bridge retrofits or those which have been identified as potentially viable to address the deficiencies of the bridges in the region [28].

3. ANALYTICAL MODELING AND SIMULATION

3.1. Ground motion suite

In the absence of recorded strong motions for the CSUS, two suites of synthetic ground motions are used as a part of this study. Wen and Wu [32] simulated ground motions for three cities in Mid-America (Memphis, TN; Carbondale, IL; and St. Louis, MO) for seismic performance assessment.
of structures, such that the median of the response spectra for the suite matches the uniform hazard response spectra for 10 and 2% probability of exceedance in 50 years. More recently, Rix and Fernandez [33] developed a suite of scenario-based synthetic ground motions for Memphis, TN, that will also be used in this work. These scenario ground motions were developed based on stochastic methods, considering non-linear site response, and the influence of the deep soil column of the upper Mississippi embayment. Twenty ground motions were simulated for each of the 11 different magnitude–distance pairs.

Forty-eight ground motions from Wen and Wu [32] and 48 ground motions from Rix and Fernandez [33] are used in this study for the development of the PSDMs and fragility estimation. The ability of these suites to capture such inherent uncertainties as the earthquake source, wave propagation, and soil conditions dictates the ability of the fragility curve development procedure to propagate these aleatoric uncertainties. The developers of each suite of motions have given due attention to considering such uncertainties, yet with varying levels of fidelity or models implemented. For example, Wen and Wu [32] have considered uncertainty in such parameters as magnitude, epicentral location, focal depth, fault size, path attenuation, and soil amplification, whereas soil profile has been generically defined for three different sites in the region. For the Rix and Fernandez motions [33], which are scenario events with deterministic magnitude–distance pairs, care was taken to capture uncertainties in the source, path, and site in the development of the Fourier amplitude spectrum, whereas the phase spectrum contains Gaussian white noise. Further details on the uncertainty modeling approaches adopted by each can be found in their respective works. The development of the fragility curves produced in this work has also attempted to acknowledge and capture the epistemic uncertainties associated with different ground motion modeling approaches by utilizing both suites of synthetic motions. The reduced ground motion suites used in this work were identified by Nielson and DesRoches [30] for fragility analysis, consisting of a range of PGA and spectral acceleration values and representative of CSUS ground motion characteristics.

3.2. Analytical bridge models

Three-dimensional non-linear analytical models are developed in OpenSees [34] for use in the fragility analysis. A separate set of 96 models (note, the 96 models corresponds to the 96 ground motions) is generated for the as-built and each class of retrofitted bridges. Although each of the 96 different three-dimensional models has slight variations in its properties and base geometry (see discussion below regarding uncertainty treatment), the general modeling approach is consistent. The composite slab and girders are modeled with linear elastic beam–column elements, since the superstructure is expected to remain elastic under seismic excitation. Pounding between the deck and abutments is captured with a bi-linear contact element following the recommendations of Muthukumar [35]. The bearing models are dictated by the stiffness of the elastomer, sliding of the bearing, and stiffness and yield of the steel dowels, following the analytical modeling of Choi [31] and Nielson [24]. Discretized fiber sections applied to beam–column elements are used for the both the circular columns and concrete bent beams, and the pile foundations are modeled with simplified linear translational and rotational springs. The active, passive, and transverse responses of the abutments are represented by non-linear inelastic springs. Further details on the assumptions and analytical models themselves can be found elsewhere [24, 28].

Like the as-built bridge itself, properties of the retrofit change for each bridge sample and generated analytical model. This is due to the variation in bridge geometry, which affects the
anticipated retrofit design, as well as uncertainties in the realization of the retrofit properties (i.e. realized strength of steel in jacketed column). However, each retrofit measure has a typical modeling scheme, which is shown in Figure 2. The restrainer cables are modeled using non-linear tension-only elements with a gap representing the initial slack in the cable. Bi-linear springs in the longitudinal and transverse directions simulate the force–displacement of typical laminated elastomeric bearings. Keeper plates are common details for these bearings and are captured in the model by stiff springs in the transverse direction, engaging after the gap is closed. Similarly, the reinforced concrete shear keys engage only in the transverse direction after overcoming an initial gap, with the Coulomb friction models defining their behavior. The steel jackets are reflected in the analytical model by altering the column section model. Material models for the concrete fibers are affected by having an increased compressive strength and ultimate strain due to the jacket, and a slight increase in elastic modulus to reflect the increase in stiffness due to jacketing. In general, although the analytical model is somewhat affected by the use of steel jacketing, the primary impact of the retrofit is to increase the column ductility capacity. The seat extenders do not require unique analytical modeling; however, their effect on the fragility of the bridge will be to increase the limit state capacity for collapse due to excess longitudinal displacement. The impact of the various retrofits on the limit state capacities will be discussed in a later section.

3.3. Uncertainty treatment

One of the primary intentions of a fragility analysis is to capture the uncertainties inherent in a seismic performance assessment and quantify probabilistically the potential for damage. These sources stem from uncertainties in the ground motion characteristics, bridge modeling parameters (such as bearing stiffness, concrete strength, restrainer cable slack), or variations in gross geometric properties. The evaluation of general classes of retrofitted bridges yields this added complexity where there may be variation in the span length, column height, and deck width. In general, these uncertainties tend to affect the demand estimate, and uncertainties in the capacity of various components are considered by use of a probabilistic model for the capacity estimate, as opposed to a deterministic limit.
Table III. Most significant modeling parameters identified from screening study of 23 parameters, along with the probabilistic model used for developing bridge samples in fragility analysis.

<table>
<thead>
<tr>
<th>Modeling parameter</th>
<th>Probabilistic model</th>
<th>Units</th>
<th>Significance for retrofit type</th>
</tr>
</thead>
<tbody>
<tr>
<td>Loading direction</td>
<td>$U(0, 2\pi)$</td>
<td>rad</td>
<td>AB</td>
</tr>
<tr>
<td>Active abutment stiffness</td>
<td>$U(3.5, 10.5)$</td>
<td>kN/mm/pile</td>
<td>x</td>
</tr>
<tr>
<td>Deck-abutment gap</td>
<td>$N(\mu=38.1, \sigma=5.84)$</td>
<td>mm</td>
<td>x</td>
</tr>
<tr>
<td>Damping ratio</td>
<td>$N(\mu=0.045, \sigma=0.0125)$</td>
<td>Ratio</td>
<td>x</td>
</tr>
<tr>
<td>Keeper gap</td>
<td>$U(6.4, 19.1)$</td>
<td>mm</td>
<td>x</td>
</tr>
</tbody>
</table>

Note: AB, as-built; RC, restrainer cable; EB, elastomeric bearing; SJ, steel jacket; SK, shear key; $U$, uniform distribution; $N$, normal distribution.

A recent study [36] has performed detailed analysis of the sensitivity of the seismic response and fragility to various sources of uncertainty and produced a set of recommendations on the most important parameters to consider. These results have been adopted in this work. Uncertainty in the ground motions is captured by the use of the two ground motion suites previously discussed, and the angle of incidence of the two-component ground motion and the bridge is treated as a random variable. Variation in the gross geometric properties, a critical consideration, is accounted for by the use of the eight geometric samples presented in Table II. Only the most critical analytical modeling parameters identified in a prior screening study [28] are treated as variables in developing permutations of the bridge models. The screening study evaluated the statistical significance of roughly 15–16 different modeling parameters for the different MSC concrete retrofitted bridges, including uncertainties in the retrofit properties themselves. The parameters that most significantly affect the bridge response differ for each retrofit measure, but, in general, the use of the screening study reduces the number of parameters that needs to be treated probabilistically from 15 to 3 or 4 for the MSC concrete bridge class. Although probabilistic models for the 23 as-built and retrofit modeling parameters were developed in all, only those identified for use in the demand model sampling are presented in Table III. The loading direction defined by a uniform distribution is assumed to have equal likelihood of intersecting the bridge at any direction. Based on a review of the current state of practice, gaps to the keeper plate provided at the elastomeric isolation bearings range from 6.4 to 19.1 mm and are modeled with a uniform distribution recognizing the uncertainty in the realized value, yet given the limited further information available to fully characterize a probabilistic model. The distributions for active abutment stiffness (uniform), deck-abutment gap (normal), and damping ratio (normal) presented in Nielson [24] based on a review past studies of typical damping values in bridges, and CSUS bridge characteristics, are adopted herein. Further details of the screening can be found elsewhere [28].

4. DEMAND AND CAPACITY ESTIMATES

4.1. Probabilistic seismic demand models

PSDMs are constructed from the peak component responses of the 96 simulations for the various retrofitted bridge classes using the outlined methodology. PGA is used as the intensity measure for developing the relationships based on the conclusions of a recent study that identified the measure...
Figure 3. Comparison of the PSDMs for select components of the MSC concrete bridge as-built and retrofit with shear keys.

as an appropriate IM for CSUS bridge classes [37]. In many cases, the retrofit measure alters the demand model for different components.

Figure 3 shows the PSDMs for transverse abutment deformations and fixed bearing deformations for the bridge retrofit with shear keys. The regression line for the as-built bridge is plotted relative to the retrofitted bridge to indicate the shift in the demand resulting from its adoption. As illustrated in this example, the bearing demands are decreased, whereas the abutment demands are increased for the same level of ground shaking. This trend is consistent with the findings of a prior deterministic evaluation of the three-dimensional seismic response of these types of retrofitted bridges [28]. In some cases, the retrofits affect both the median of the demand (Equation (5)) and the dispersion. For example, Figure 3 indicates that the use of shear keys reduces the logarithmic standard deviation associated with the transverse fixed bearing demands from 0.62 for the as-built bridge to 0.38. Results for the different retrofit measures reveal a similar trend where the demands may be increased, decreased, or unaffected in terms of the median or dispersion. In addition to the marginals for the JPSDM, correlation matrices are also constructed for each retrofitted bridge type, although most retrofits only have a slight impact on the correlation between the component demands for the MSC concrete bridge.

4.2. Limit state capacity estimates

The damage states used in this study are qualitatively described as slight, moderate, extensive, and complete damages. Estimates of the limit state capacities for each component and damage state are selected in an effort to maintain consistency across the various bridge components. Achieving a particular damage state for one component should have a similar impact on the functional performance of the entire bridge system as achieving the damage state for another component. This objective, which is more thoroughly discussed in Padgett and DesRoches [36], is considered in the construction of capacity estimates used in the development of bridge fragility curves in this work.

4.2.1. As-built components. The limit state capacities used for the as-built components are those proposed by Nielson and DesRoches [24, 30]. Physics-based limit state capacities were refined
Table IV. Capacity estimates for as-built components adapted from Nielson and DesRoches [30].

<table>
<thead>
<tr>
<th>As-built component</th>
<th>Slight</th>
<th>Moderate</th>
<th>Extensive</th>
<th>Complete</th>
</tr>
</thead>
<tbody>
<tr>
<td>Concrete column ($\mu_p$)</td>
<td>$S_c$</td>
<td>$\beta_C$</td>
<td>$S_c$</td>
<td>$\beta_C$</td>
</tr>
<tr>
<td>Elastomeric bearing expan-long</td>
<td>1.29</td>
<td>0.59</td>
<td>2.10</td>
<td>0.51</td>
</tr>
<tr>
<td>(mm)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Elastomeric bearing expan-tran</td>
<td>28.9</td>
<td>0.60</td>
<td>104.0</td>
<td>0.55</td>
</tr>
<tr>
<td>(mm)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Elastomeric bearing fixed-long</td>
<td>28.9</td>
<td>0.60</td>
<td>104.0</td>
<td>0.55</td>
</tr>
<tr>
<td>(mm)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Elastomeric bearing fixed-tran</td>
<td>28.8</td>
<td>0.79</td>
<td>90.9</td>
<td>0.68</td>
</tr>
<tr>
<td>(mm)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Abutment-passive (mm)</td>
<td>37.0</td>
<td>0.46</td>
<td>146.0</td>
<td>0.46</td>
</tr>
<tr>
<td>Abutment-active (mm)</td>
<td>9.80</td>
<td>0.70</td>
<td>37.9</td>
<td>0.90</td>
</tr>
<tr>
<td>Abutment-tran (mm)</td>
<td>9.80</td>
<td>0.70</td>
<td>37.9</td>
<td>0.90</td>
</tr>
</tbody>
</table>

through a Bayesian updating process to incorporate the results of a survey on the limits inspectors place on damage before altering the bridge functionality [36]. Capacity estimates for a range of bridge components relevant for the case study MSC concrete bridge class are considered, as shown in Table IV. It is noted that some of the capacity estimates are listed as ‘N/A’ for the components and damage states that the survey results indicated large amounts of damage could occur without leading to significant long term impacts on the functionality of the bridge. These components, therefore, do not contribute to the system fragility for that particular damage state.

4.2.2. Retrofitted components. It has been previously illustrated that various retrofits affect the seismic response and demand of the bridge. However, the use of retrofit may also alter the capacity of some components, including the retrofitted component itself. For example, new limit state capacities must be defined for the steel-jacketed columns or elastomeric isolation bearings. The capacity estimates for these components are defined with an effort to maintain the functional implications of each damage state and at limits where visual damage may be apparent to inspectors. The median values of the retrofitted component limit state capacities are assigned in most cases based on past experimental tests of the retrofits. With little additional information on the dispersion about the median, judgment is often used along with knowledge of the uncertainty in the capacity of the as-built component.

Limit state capacities for slight through complete damage for the steel-jacketed columns are assessed based on tests of steel-jacketed circular columns by Chai et al. [38] and Priestley et al. [39]. For example, the moderate damage corresponds approximately to the point when yielding of the jacket and visual bulging may occur, whereas complete damage corresponds to the estimated ultimate curvature ductility demand of a typical jacketed column. Although the elastomeric isolation bearings replace more vulnerable as-built bearings, there are still limits on their displacement capacities. Inferences are made as to the limit on shear strain and displacement capacities based on past testing [40–43]. For example, slight damage is approximated as the point at which tests of typical CSUS bearings and pedestals have revealed potential yielding of the anchor bolts and cracking of the pedestals; extensive damage corresponds to an anticipated yielding of the steel shims, which would require replacement following the event. Although the seat extenders do not affect the demand placed on the bridge, they do alter the limit state capacity for the complete damage state by increasing the capacity to sustain deck displacements and bearing deformations before unseating occurs. The shear keys and restrainer cables do not affect the limit state capacities,
Table V. Capacity estimates for retrofitted components.

<table>
<thead>
<tr>
<th>Retrofitted component</th>
<th>Slight $S_c$</th>
<th>$\beta_C$</th>
<th>Moderate $S_c$</th>
<th>$\beta_C$</th>
<th>Extensive $S_c$</th>
<th>$\beta_C$</th>
<th>Complete $S_c$</th>
<th>$\beta_C$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Elastomeric isolation bearings-long (mm)</td>
<td>140.00</td>
<td>0.60</td>
<td>210.00</td>
<td>0.55</td>
<td>279.00</td>
<td>0.59</td>
<td>489.00</td>
<td>0.65</td>
</tr>
<tr>
<td>Elastomeric isolation bearings-trans (mm)</td>
<td>140.00</td>
<td>0.79</td>
<td>210.00</td>
<td>0.68</td>
<td>279.00</td>
<td>0.73</td>
<td>489.00</td>
<td>0.66</td>
</tr>
<tr>
<td>Steel-jacketed column ($\mu_s$)</td>
<td>9.35</td>
<td>0.59</td>
<td>17.70</td>
<td>0.51</td>
<td>26.10</td>
<td>0.64</td>
<td>30.20</td>
<td>0.65</td>
</tr>
<tr>
<td>Elastomeric bearing fixed-long w/SE (mm)</td>
<td>28.90</td>
<td>0.60</td>
<td>104.00</td>
<td>0.55</td>
<td>136.00</td>
<td>0.59</td>
<td>339.00</td>
<td>0.65</td>
</tr>
<tr>
<td>Elastomeric bearing expan-long w/SE (mm)</td>
<td>28.90</td>
<td>0.60</td>
<td>104.00</td>
<td>0.55</td>
<td>136.00</td>
<td>0.59</td>
<td>339.00</td>
<td>0.65</td>
</tr>
</tbody>
</table>

Note: SE, seat extender.

although they alter the seismic demand. Further details on the assumptions and development of capacity estimates for the retrofitted components can be found in Padgett [28]. The limit state capacities, presented in terms of a median and dispersion, derived for use in this study, are shown in Table V.

5. FRAGILITY CURVES FOR RETROFITTED BRIDGE COMPONENTS AND SYSTEM

5.1. Component vulnerability pre- and post-retrofit

Evaluating the fragility of critical components before and after retrofit provides valuable insight on the effectiveness of different retrofit measures and potential reasons for system impacts. The component fragilities can be evaluated in closed form following Equation (4). Findings from the PSDM analysis propagate through the component fragility assessment, where some retrofits may have either a positive or negative impact on the component.

Figure 4 shows the fragility curves for the slight damage state for four different components. The component fragilities for the as-built bridge (bold) are shown relative to the retrofitted bridges. For this particular damage state and bridge class, it is illustrated that the use of elastomeric isolation bearings as a retrofit reduces the bearing vulnerability, while increasing the abutment fragility. Other damage states and components reveal different notable trends. For example, at the moderate damage state, the restrainer cables slightly decrease the expansion bearing vulnerability, yet considerably increase the abutment fragility in active action. This is because of the increase in forces transferred through the cables when in tension. Similarly, as Figure 4 also shows for the slight damage state, the shear keys lead to a reduction in the transverse vulnerability of the fixed bearings, yet have a negative impact on the abutments in the transverse direction. This is a result of the transfer of forces to the abutment when resisting the transverse deck displacement.

The steel jackets essentially eliminate the vulnerability of the columns, yet have virtually no impact on the vulnerability of any other component of the bridge system. If the columns dominated the system fragility, this may be an ideal selection for a retrofit measure. However, it is seen in Figure 5 that there are a number of critical components that contribute to the fragility of the MSC concrete girder bridge system. The four damage states from slight through complete damage are shown to illustrate the vulnerability of different components for the as-built bridge. The relative contribution of each component changes as a function of the damage state, yet no single component is solely representative of the bridge system fragility estimate. Employing different retrofit measures alters the relative vulnerability of the different components and hence the system fragility.
This reveals the importance of considering a retrofit’s impact on multiple components when developing system fragility curves, as proposed in the presented methodology herein.

5.2. Fragility curves for case study bridge class

The insight gained from the component fragility curves leads to a need to evaluate the overall system vulnerability and impact of retrofit. The system fragility curves are developed by considering the contribution of multiple vulnerable components through the methodology proposed in Section 2. Fragility curves depicting the overall bridge vulnerability and relative impact of various retrofits on the performance of the bridge system are presented for the case study MSC concrete bridge class. The fragility curves for slight through complete damage for the bridge retrofit with steel column jackets, elastomeric isolation bearings, restrainer cables, shear keys, or seat extenders relative to the as-built bridge are shown in Figure 6. Although combinations of retrofit are often performed, this study does not evaluate the effect of combinations of retrofit measures on the overall system fragility. Future work will assess the effects of combinations on further reducing the vulnerability of bridges.

From Figure 6 it is evident that different retrofit measures appear to be more effective for different damage states in terms of reducing the probability of damage for a given PGA. For example, the elastomeric bearings reduce the system vulnerability for the slight damage state,
whereas the seat extenders are more effective for the complete damage state. Again, this is a result of the influence of the retrofit measures on the seismic capacity and demand placed on various components, which was evident when viewing the component fragility curves. The median values (PGA) of the fragility curves for the different retrofit measures normalized by the as-built fragility are plotted in Figure 7. It is noted that this does not account for changes in dispersion which may be important considerations, along with other factors such as cost, constructability, etc. However, a larger normalized median value is indicative of a retrofit measure that results in a larger reduction in system vulnerability.

The importance of developing system level retrofitted bridge fragility curves is again recognized. For example, if the impact of restrainers on the longitudinal vulnerability of the expansion bearings were the only consideration made, the fragility would be assumed to be improved by 25% at the moderate damage state on the basis of median value shift. However, by assessing the system vulnerability, it is observed that the actual fragility is only improved by 1% relative to the as-built bridge because of the contribution of other components such as the fixed bearings, abutments, etc. On the other hand, evaluating the impact of retrofitting with elastomeric isolation bearings based solely on reduced demands on the substructure would lead to a 42% under-estimation of the system improvement at the extensive damage state. This is because of the contribution of improved bearing fragility in addition to the columns, among other items. Hence, the importance of capturing not only the enhanced performance of the targeted seismic response quantity and component vulnerability but also system level impact of bridge retrofit is emphasized.
Figure 6. Fragility curves for retrofitted MSC concrete bridge.

Figure 7. Comparison of median value of the fragility for various retrofit measures normalized by the as-built median value PGA. (Note: The normalized median for elastomeric isolation bearings at the slight damage state is off the scale at 2.9.)
6. CONCLUSIONS

This work has presented an approach to developing systems level fragility curves for retrofitted bridge classes through an analytical methodology. Emphasis is placed on capturing the impact of retrofit on multiple key components when assessing the bridge fragility in its retrofitted condition. Although seismic retrofit measures tend to be targeted at individual component response quantities, this study has shown that the fragility of other components may be indirectly affected in either a positive or a negative way. Alternatively, there may be additional sources of vulnerability, which are not addressed by a given retrofit scheme, yet should be captured when assessing the bridge fragility. This is accomplished by the development of joint PSDMs and capacity estimates which capture the influence of retrofit on both the dynamic response and limit state capacities for the system. Component level fragility curves may be developed for illustration purposes to evaluate the impact of a particular retrofit on the columns, bearings, or abutments; however, retrofitted bridge system fragility curves may be directly evaluated.

A case study bridge class, the MSC concrete girder bridge, retrofit with five different measures is used to exemplify the methodology and recommended implementation details, including analytical modeling, uncertainty treatment, limit state capacities, etc. The retrofit measures considered include steel column jackets, elastomeric isolation bearings, restrainer cables, shear keys, and seat extenders. Three-dimensional analytical models are utilized and subjected to two-component synthetic ground motions. Preliminary screening studies are used to identify the most critical variable modeling parameters for treatment in bridge sample simulation. The impact of retrofit on PSDMs was illustrated to note the shift in demand resulting from the use of different retrofit measures, and limit state capacities for retrofitted components were proposed. The results reveal that the most effective retrofit measure in reducing probable damage is a function of the damage state of interest. Additionally, the potential to over-estimate or under-estimate system fragility by only evaluating the impact of retrofit on the targeted component was revealed.

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