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Development of a bridge-specific fragility methodology to improve the seismic resilience of bridges

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Abstract. This article details a bridge-specific fragility method developed to enhance the seismic design and resilience of bridges. Current seismic design processes provide guidance for the design of a bridge that will not collapse during a design hazard event. However, they do not provide performance information of the bridge at different hazard levels or due to design changes. Therefore, there is a need for a supplement to this design process that will provide statistical information on the performance of a bridge, beyond traditional emphases on collapse prevention. This article proposes a bridge-specific parameterized fragility method to enable efficient estimation of various levels of damage probability for alternative bridge design parameters. A multi-parameter demand model is developed to incorporate bridge design details directly in the fragility estimation. Monte Carlo simulation and Logistic regression are used to determine the fragility of the bridge or bridge component. The resulting parameterized fragility model offers a basis for a bridge-specific design tool to explore the influence of design parameter variation on the expected performance of a bridge. When used as part of the design process, these tools can help to transform a prescriptive approach into a more performance-based approach, efficiently providing probabilistic performance information about a new bridge design. An example of the method and resulting fragility estimation is presented.

Keywords: bridge-specific fragility; response surface; seismic design process; bridge resilience; probabilistic method

1. Introduction

Seismic fragility curves are statistical functions that give the probability of exceeding a certain damage state as a function of a ground motion intensity measure. Fragility curves can be used as tools to assess and mitigate the effects of earthquake ground motions on structures (Calvi *et al.* 2006, Padgett 2007, Padgett and DesRoches 2008, Kwon and Elnashai 2010, Yazgan, 2015). The fragility function can be written as $P[DS|IM=y]$ where $IM=y$ stands for the ground motion intensity measure (*IM*) taking a particular value, and *DS* is the exceedance of the damage state in question. Fragility analysis involves analyzing a structure to determine its preparedness to withstand certain ground motion intensities. This type of analysis has become extremely important in the earthquake engineering community in providing end users with information to assist in mitigating the effects of earthquake forces (Koutsourelakis 2010, Avsar *et al.* 2011, Huo and Zhang 2012, Chen and Chen 2016, Zelaschi *et al.* 2015, Mosleh *et al.* 2016a, 2016b). Fragility curves have been developed and are used in earthquake-prone regions to provide information about infrastructure performance and determine its expected performance during a likely earthquake, as well as assist agencies in making retrofitting decisions (Varum *et al.* 2011, Chang *et al.* 2012).

This article details a fragility method that can provide probabilistic performance information during the seismic design process for bridges. In illustrating the need for this type of performance information, the seismic design procedure of The California Department of Transportation (Caltrans) was reviewed (Caltrans SDC 2010). Caltrans has a seismic bridge design process for an Ordinary Bridge described in the Seismic Design Criteria (SDC). It directs the design engineer to meet minimum requirements resulting in a bridge that should remain standing in the event of a Design Seismic Hazard (DSH). Within the SDC, seismic hazards such as a design spectrum or ground motion time histories determine the demands on the bridge components and bridge system. These demands compare to the capacity of the components to ensure that the bridge meets key performance criteria. The SDC also specifies design detailing of various components, including the columns, abutments, foundations, hinge seats and bent caps. The expectation of following the guidelines set forth by the SDC during the design process is that the resulting bridge design will avoid collapse under anticipated seismic loads.

The SDC can be improved in several areas. The procedure set forth in the SDC is a prescriptive approach that does not provide probabilistic information on the bridge performance. Although the SDC produces bridge designs that will not collapse during a DSH, the collapse capacity of the structure is uncertain in itself (Luco *et al.* 2007) and is not addressed by the SDC. Furthermore, performance levels other than the collapse limit are not addressed.

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Understanding the likelihood of intermediate damage levels that affect bridge functionality and restoration of service is critical from the perspective of resilience (Bruneau *et al.* 2003). The current approach in SDC does not account for the performance of the bridge at hazard levels other than the Design Seismic Hazard. The current design process also does not directly provide information on the expected performance as a function of different design details. Therefore, there is a need for a supplement to this design process that will provide statistical information on the performance of a bridge during a Design Seismic Hazard, as well as for other hazard levels. The bridge-specific fragility method presented in this paper was developed within the context of improving the seismic design process of bridges. It helps to transform current seismic design processes from a prescriptive approach to a more probabilistic approach. Furthermore, the bridge-specific fragility method shortens the amount of effort required to obtain a fragility estimate when compared to traditional fragility methods so that a bridge designer can quickly determine if a bridge design has an acceptable response to a DSH. This research improves the design engineer's understanding of the probabilistic performance of their bridge design as a function of several design details. This is accomplished by providing design engineers with rapid access to fragility estimates during the design process.

The subsequent sections of this journal article detail the bridge-specific fragility method (BSFM), key components of the method, and an example demonstrating the method for practical use. A two-span integral concrete bridge, a bridge common among California state bridges, is used to demonstrate the method herein. However, the proposed approach is relevant and applicable to other bridge types and can be extended in the future to support the design process for other typical bridge classes.

2. Bridge Specific Fragility Method (BSFM)

Rationally, the fragility curves are conditioned on single parameter, such as ground motion intensity measure. However, the single-parameter demand models and fragility curves have some limitations: (1) the inability to account for the influence of uncertainty (modeling) parameters on structural performance during earthquakes without extensive re-simulations for each new set of parameter combinations; (2) the inability to explicitly address the effect of uncertainty parameters on fragility curves (Ghosh *et al.* 2013, Mangalathu 2017, Mangalathu *et al.* 2018). These limitations can be addressed by the generation of parameterized fragility curves (Seo *et al.* 2013, Dukes 2013). The current study derives fragility curves through the use of multi-parameter probabilistic seismic demand models that offer approximating functions of bridge component responses as a function of design details as well as the hazard intensity (Seo *et al.* 2012, Seo and Linzell 2013, Dukes 2013, Ghosh *et al.* 2013, Mangalathu *et al.* 2015, Mangalathu 2017). The advantage of using a multi-parameter demand model is that the fragility for a bridge component or system will be conditioned on the ground motion intensity measure as well as the other design variables, allowing fragility curves developed specifically for a bridge with a given set of conditioning design variable quantities. Assuming that the input variables are statistically independent, a multi-parameter demand model of each bridge component (demand parameter) is constructed. Samples obtained from this demand model are compared with those of the associated limit state model to obtain the binary survival-failure vector. This vector is used to perform a logistic regression analysis to determine the regression coefficients and thus develop the multi-parameter fragility curve in the component. The bridge specific fragility method presented in this paper has the ability to efficiently incorporate bridge design details into the fragility estimation method. These details act as conditioning variables on the fragility equations and analysis. Thus, one need of this research was to find the design details that have the most effect on the responses of the different components of the two-span integral box girder bridge class (used as an illustrating case herein) to earthquake ground motions. The three basic steps involved in the generation of the BSFM is (1) the generation of the multi-parameter demand model, (2) fixing the limit states (or capacity) and (3) the generation of fragility curves by a logistic regression approach.

2.1 Multi-parameter demand model

Many demand models for traditional fragility analysis use one parameter, usually a ground motion intensity measure, in the derivation of a predictive model of seismic demand on the bridge system or component (Calvi *et al.* 2006, Kwon and Elnashai 2010, Banerjee and Chi 2013, Zelaschi *et al.* 2014, Praglath *et al*. 2015, Ramanathan *et al.* 2015, Chen and Chen 2016, Mangalathu *et al.* 2016a, Kostinakis and Morfidis 2017, Bhosale *et al.* 2017, Mangalathu *et al.* 2018b). In contrast, the multi-parameter demand model herein utilizes design parameters in addition to a ground motion intensity measure. By also using the design parameters as predictors, the demand model can be readily applied across a broad range of prospective designs and hence a probabilistic seismic demand model specific to the particular design realization bridge of interest can be developed. The multi-parameter demand model is developed using the concept of metamodels (Simpson *et al.* 2001).

Amongst the various available metamodels, the current study uses the response surface method (RSM). The RSM usually combines a factorial design, polynomial model, and least squares regression as the design of experiment, model choice and model fitting (Simpson *et al.* 2001). RSM has been used by various researchers for the fragility analysis (Towashiraporn 2004, Seo *et al.* 2012, Ghosh *et al.* 2013). RSM is the metamodel chosen to develop the multi-parameter demand model because it is a natural extension of the current single parameter probabilistic seismic demand models (PSDM) in traditional fragility analysis and is computationally less expensive compared to other surrogate models. Furthermore, it was found to have good predictive quality for the tested case.

The response variable and the ground motion intensity measure is transformed in the lognormal space to produce a better relationship between the two (Cornell *et al.* 2002, Mangalathu *et al.* 2016b). The relationship for a single-parameter demand model is shown in Eq. (1), where Y is the component response variable, IM represents the ground motion intensity measure, and a and b are regression coefficients. Eq. (2) shows the equation for the multi-parameter demand model, which is a linear first-order regression model with lognormal transformations on the response variable and the ground motion intensity, where n is the number of variables. X_2, … , X_n represent the design parameters on which the fragility curve will be conditioned, and will be described in the following sections.

$$\ln(Y) = a + b\ln(IM) \tag{1}$$

$$\ln(Y) = \beta_0 + \beta_1 \ln(IM) + \beta_2 \ln(X_2) + ... + \beta_n \ln(X_n) \tag{2}$$

2.2 Capacity model

The capacity model, consisting of limit states which define the quantitative threshold values for different damage conditions, is important to define specifically for the structure type and expectations of the performance of the structure. Limit states in fragility analysis are defined as discrete threshold quantities of a component response that corresponds to a physical damage condition (Mackie and Stojadinović 2005). The damage states used in fragility curves have traditionally been the following four levels (Choi 2004, Mosleh 2016b, Mangalathu 2017): Slight (LS_1, hereafter), Moderate (LS_2, hereafter), Extensive (LS_3, hereafter), and Complete (LS_4, hereafter). These four categories apply to a particular component of the bridge being analyzed, such as the columns, footings, and abutments. Many fragility curves have focused on the response of one component, such as the drift of a column, to indicate the state of a bridge after an earthquake event. However, the responses of other major bridge components have emerged as significant elements in determining the fragility curve for the entire bridge (Ramanathan 2012, Mangalathu 2017). The capacity models are usually described by a two-parameter lognormal distribution with median, S_c and dispersion, β_c (Ramanathan *et al.* 2015). In theory these capacity models could also be parameterized per design parameters, but such refined probabilistic needs extensive experimental data and is beyond the scope of the current paper.

2.3 Generation of fragility curves by logistic regression

Logistic regression is used to find the best fitting model that describes the relationship between an outcome or response and a set of predictor variables (Kutner, *et al.* 2005), in this case, provides the form of the cumulative distribution function that describes the parameterized bridge failure probability given multiple input parameters. The first step involved in the development of the fragility curves is the generation of large number of demand estimates (N), usually 100,000 in the examples herein. This can be achieved by randomly generating predictor variables (IM, X_1,...,X_n) based on their probabilistic distribution through the Latin hypercube sampling technique and the estimation of demand using Eq. (2). N capacity values were also generated based on their probability distribution per limit state. The sampled points from the demand and capacity models are compared to generate binary data of components and systems that are considered failed or safe. These resultant vectors are then regressed against a matrix of the original design parameters using a logistic regression to find regression coefficients, α_k, seen in Eq. (3). This equation supplies the fragility estimate for a component, k.

$$PF_{k|IM,X_1,..X_n} = \frac{e^{\alpha_{k,0}+\alpha_{k,IM}\ln(IM)+\sum_{i=1}^{n}\alpha_{k,X_i}\ln(X_i)}}{1+e^{\alpha_{k,0}+\alpha_{k,IM}\ln(IM)+\sum_{i=1}^{n}\alpha_{k,X_i}\ln(X_i)}} \tag{3}$$

To develop the binary vector of system failure, a system abstraction must be developed that links component to system level failure. Although a range of definitions may exist depending upon the perspective on system failure (Duenas and Padgett, 2011), a series system assumption is adopted herein, i.e., if any of the components within a system fails, then the entire system has failed. Components that contribute to the lower damage states are considered to be both “primary” and “secondary” components, while only “primary” components are considered to affect the higher damage states. The validity of this assumption from the perspective of post-event functionality and resilience lies in the fact that if any one of the components is damaged to a level that would require a particular level of closure, then the system would be tagged that level.

As the fragility is based on multiple parameters, the result is a multi-dimensional fragility surface or cloud, containing the points produced by the logistic regression equation, instead of the 2-dimensional curve developed in traditional fragility methods. To graphically show the cloud in two or three dimensions, all but one or two parameters have to be deterministically defined while varying the remaining one or two parameters of interest within a range in order to graph the 2-dimensional fragility curve or 3-dimensional fragility surface. 2-dimensional fragility curves are shown in the demonstration of the BSFM with a case study given in the following section.

3. Application of the BSFM to a common California Bridge class

The BSFM is applied here to two span single frame box girder bridges in California with single column bents to illustrate the method and its potential to support design considerations. The two span frame bridge is one of the most common bridge type in California as noted by Ramanathan (2012) and a typical layout of a two span bridge is shown in Fig. 1.

3.1 Bridge design and input parameters

The bridge design parameters considered are chosen corresponding to characteristics of a bridge that were found to be important to monitor during the design process

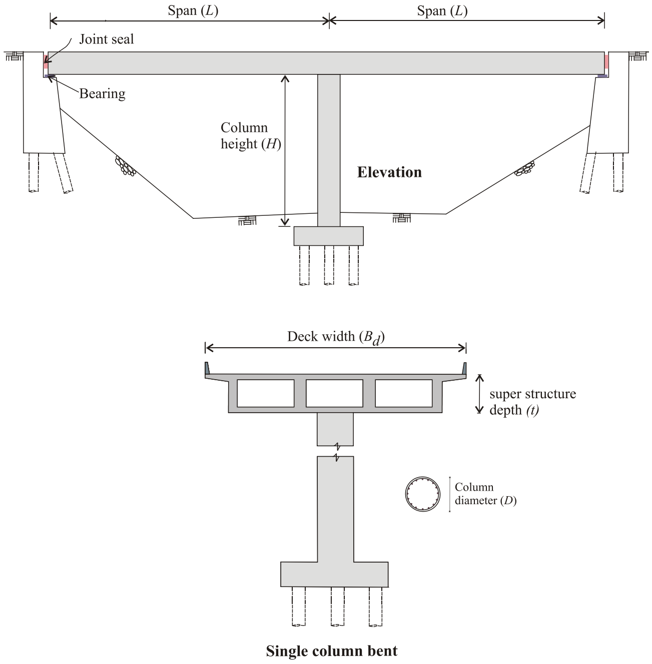


Fig. 1 Typical configuration of two-span box girder bridge

Table 1 Description of design parameters considered

Design Parameter	Effect on Bridge Behavior	Bounds		Median value
		Lower bound	Upper bound	
Longitudinal steel reinforcement ratio of the column (X_1, %)	A higher steel ratio stiffens and strengthens the column	1.0	3.0	2.0
Volumetric ratio of transverse steel reinforcement of the column (X_2, %)	Determines the difference between unconfined and confined concrete strength, which determines the capacity of the component	0.5	1.4	0.95
Aspect Ratio - Column height to column dimension ratio (X_3)	Increasing this ratio makes the structure more flexible	2.5	6.0	4.25
Superstructure depth to column dimension ratio (X_4)	Increasing the depth makes the structure more stiff	0.8	1.3	1.05
Span length to column height ratio (X_5)	Increasing the span length makes the structure more flexible	4.5	9.5	7.0

(Mackie and Stojadinović 2005, Caltrans 2010), significant in the evolution of seismic design of bridges (Sahs *et al.* 2008), as well as those suggested by the Caltrans project team (Caltrans 2012). Table 1 lists the design parameters used in this article and the effects on the seismic performance of a bridge. The design parameters are selected based on an extensive sensitivity study conducted by Dukes (2013) with the objective to use BSFM in design offices. Interested readers are directed to Dukes (2013) for a detailed discussion on the sensitivity study. The sensitivity study revealed that the five design parameters listed in Table 1 were statistically significant in predicting the responses of bridge components. Details of this sensitivity study can be found in Dukes (2013). Uniform distribution was adopted for the generation of bridge samples by Latin hypercube such that the parameter space efficiently covers the full range of viable design parameters. Uniform distribution was used for the bridge parameters based on distribution of the data obtained in the survey of bridge plans conducted by Dukes (2013). It is important to choose ground motions applicable to the site location in which the bridge may be designed. The ground motions developed by Baker *et al*. (2011) are used for the dynamic analyses and to incorporate the uncertainty in ground motion properties. The suite consists of 120 ground motions associated with moderate-to-strong earthquakes at small distances and 40 ground motions with strong velocity pulses characteristics of sites experiencing near-fault directivity effects. These motions were not developed as structure-specific or site-specific, and so are applicable to many research needs and can be tailored to fit individual user needs through pre-processing (i.e., scaling of motions) or post-processing (i.e., finding regression relationships between response of models and ground motion measure) of the ground motion characteristics (Baker *et al.* 2011). The peak ground acceleration (PGA) is considered as the IM in the current study from a practical perspective.

3.2 Analytical modeling

Although a detailed explanation of the modeling of the bridge can be found elsewhere (Mangalathu 2017) the general approach is presented herein. Three dimensional numerical modeling is carried out with the help of the finite element package OpenSees (Mazzoni *et al.* 2006) incorporating both geometric and material nonlinearities. The superstructure of bridges is modeled as a spine with elastic beam-column elements since it is expected that the deck will remain elastic during earthquake loading. Transverse deck elements are modeled using elastic beam-column elements (rigid and massless) and are connected to the columns using rigid links to ensure the moment and force transfer between members. The columns and bent caps are modeled using fiber sections applied to nonlinear beam column elements, and foundations with rotational and translational springs. Fiber sections in OpenSees allows the user to clearly define the confined and unconfined properties of the concrete component, as well as the steel reinforcement. The bilinear contact element developed by Muthukumar and DesRoches (2006) is used to model the pounding between the deck and the abutments. The elastomeric bearing pads that support the superstructure at the abutments and in-span hinges are modeled with translational bilinear spring elements developed by Nielson (2005) with an elastic-plastic material model in the transverse and longitudinal directions. Zero length elements capturing the response of the abutment back fill soil and bi-directional force (abutment piles or frictional surface) are connected in parallel and are connected to the transverse deck elements in the case of diaphragm abutments. The passive response of the abutment backwall is simulated

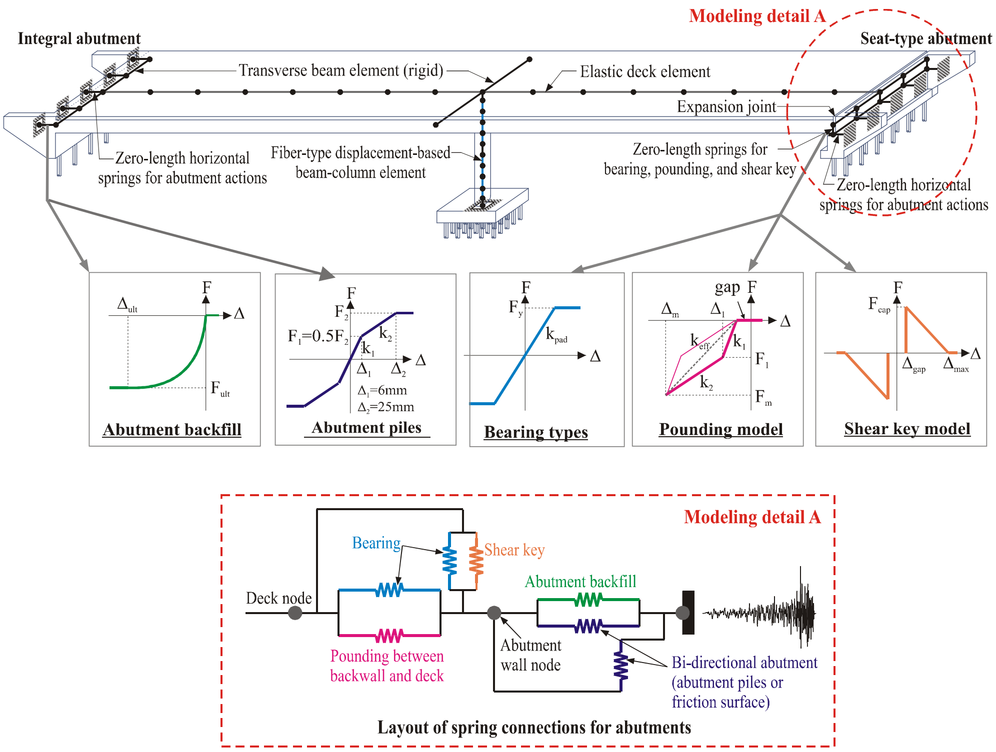


Fig. 2 Numerical models for various bridge components (Mangalathu *et al*. 2016a)

Table 2 Limit state models for various components (Ramanathan *et al*. 2015).

Component	LS_1 (slight)		LS_2 (moderate)		LS_3 (extensive)		LS_4 (complete)	
	S_c	β_c	S_c	β_c	S_c	β_c	S_c	β_c
Column curvature ductility, μ_ϕ	1	0.35	4	0.35	8	0.35	12	0.35
Abutment gap	1	0.35	3	0.35	14	0.35	21	0.35
Bearing movement	1	0.35	4	0.35	NA	NA	NA	NA
Joint seal	1	0.35	5	0.35	NA	NA	NA	NA

using the hyperbolic soil model proposed by Shamsabadi and Yan (2008). Trilinear springs stemming from the recommendations of Choi *et al.* (2004) are used to model the piles. More details of the component modeling is given by Dukes (2013). Fig. 2 illustrates the numerical models of various bridge components.

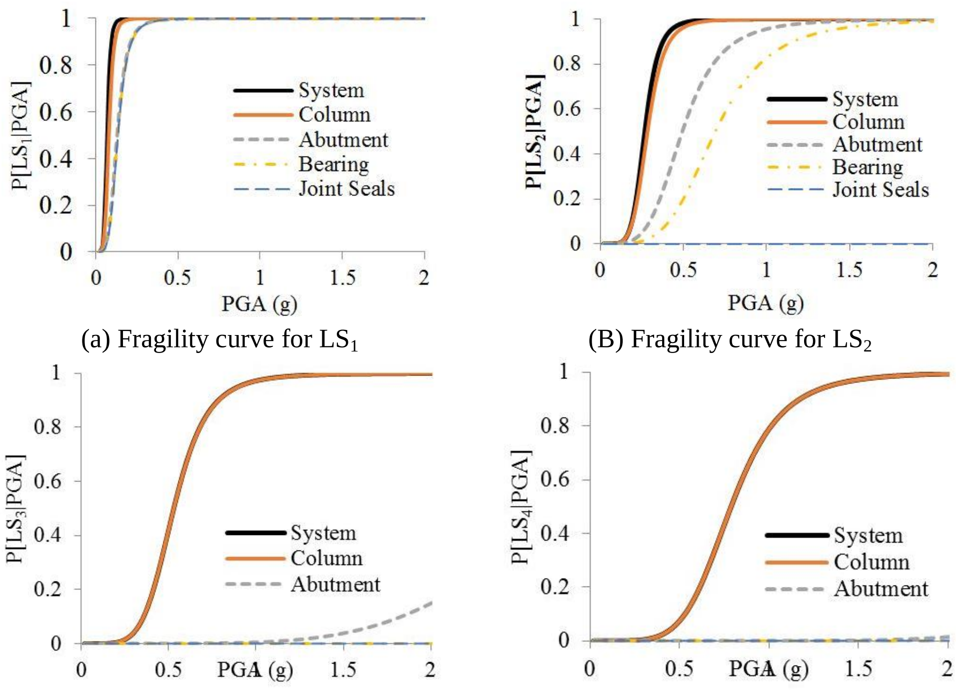
3.3 Fragility curves

Having identified the parameters of interest, statistically significant yet nominally identical 3-D bridge models are generated by sampling across the range of parameters using Latin Hypercube Sampling. One hundred and sixty analytical bridge models are generated consistent with the number of ground motions and are paired randomly. Non-linear time history analysis (NLTHA) is carried out on each bridge model and the peak component responses are recorded to determine the relationship between the peak demands and the input parameters. Four engineering demand parameters (EDPs) including column curvature ductility (μ_ϕ), abutment gap (δ_d), bearing movement (δ_a), and joint seal displacement (δ_p) are considered in the current study, and the associated limit states are shown in Table 2. S_c values were obtained through a literature review of component damage states and corresponding demand, as well as a consensus among Caltrans engineers, as these values were designed to be specific to the bridge inventory used as the sample. β_c is assigned as 0.35 in a subjective manner due to lack of sufficient information and adopted across all components and damage states (Mangalathu 2017). The column curvature response and the abutment gap displacement response act as the primary components, as they affect the vertical stability and load carrying capacity of bridge. Extensive or complete damage of these components might lead to the closure of the bridge. Bearing movement and joint seal are considered secondary components as failure of these components will not force the closure of bridge. However, it might lead to restrictions on the travel speed and traffic conditions on the bridge. Readers are referred to Ramanathan *et al.* (2015) for a more detailed discussion on the primary and secondary components and their effect on the various damage states.

The multi-parameter demand model is developed with the bridge design variables (Table 1) and EDPs using the response surface method as described in section 2. The demand values are compared with the randomly generated capacity values and logistic regression is carried out on the survival-failure vector. Table 3 gives the logistic regression coefficients for the system and component level damage for various damage states estimated using Eq. (3). The main advantage of the proposed methodology and the generated fragility curves is that a designer could produce fragility curves that are specific to the design bridge. The other

Table 3 Logistic regression equation coefficients for single-column bent bridges

Component	Limit State	$\alpha_{k,IM}$	$\alpha_{k,IM}$	$\alpha_{k,x1}$	$\alpha_{k,x2}$	$\alpha_{k,x3}$	$\alpha_{k,x4}$	$\alpha_{k,x5}$
System	LS_1	13.93	6.99	-167.6	39.35	0.84	0.47	0.53
	LS_2	5.26	6.30	-216.9	4.15	0.72	0.48	0.41
	LS_3	2.47	5.61	-274.6	-1.40	0.51	0.53	-0.05
	LS_4	0.42	5.55	-273.1	4.56	0.48	0.54	-0.15
Column	LS_1	14.77	5.82	-296.6	7.24	0.47	0.49	-0.33
	LS_2	6.71	5.76	-290.2	-1.54	0.49	0.53	-0.21
	LS_3	2.50	5.59	-274.7	-1.89	0.50	0.53	-0.06
	LS_4	0.43	5.54	-273.2	4.73	0.48	0.54	-0.15
Gap at Abutment	LS_1	2.87	4.46	4.80	5.55	0.70	0.23	1.47
	LS_2	-2.46	4.34	-3.96	0.31	0.68	0.21	1.33
	LS_3	-11.22	5.06	-5.30	11.41	0.77	0.21	1.28
	LS_4	-16.69	6.56	-1.99	-7.08	0.94	0.26	2.29
Bearing Movement	LS_1	2.96	4.43	1.64	7.67	0.73	0.21	1.42
	LS_2	-3.96	4.26	-3.44	-2.03	0.68	0.21	1.30
	LS_3	-102.6	0.00	0.00	0.00	0.00	0.00	0.00
	LS_4	-102.6	0.00	0.00	0.00	0.00	0.00	0.00
Joint Seals Movement	LS_1	3.40	4.44	-5.72	-7.60	0.70	0.22	1.37
	LS_2	-102.6	0.00	0.00	0.00	0.00	0.00	0.00
	LS_3	-102.6	0.00	0.00	0.00	0.00	0.00	0.00
	LS_4	-102.6	0.00	0.00	0.00	0.00	0.00	0.00



(a) Fragility curve for LS_1 (B) Fragility curve for LS_2

(c) Fragility curve for LS3 (d) Fragility curve for LS4

Fig. 3 System and component level fragility curves for single-column bridges with respect to PGA for four damage states

advantage of this method is that bridge-specific fragility curves can be produced without the need to create the curves deterministically with new simulations for each new bridge design.

Fig. 3 shows fragility curves for the bridge system and components for single-column bent bridges with PGA as the IM, using the regression coefficients from Table 3 and median values of the design parameters from Table 1. As indicated in Fig. 3, the bridge system is always as vulnerable as or more vulnerable than any of the contributing components, as is the nature of a series system. The performance of the column controls the performance of the bridge system for the lower bridge system damage state where both the primary and secondary components contribute to the bridge system fragility. The trend continues at the higher bridge system damage states, where only the primary components contribute to the fragility. At these higher damage states, the abutment gap response is

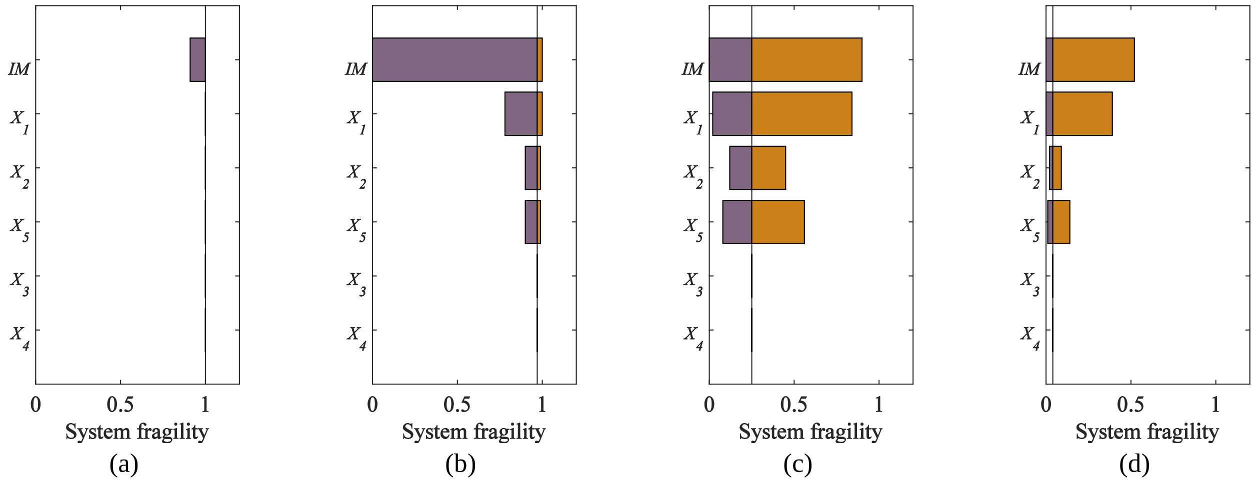


Fig. 4 Sensitivity of the fragility estimate to the design parameters for each damage state: (a) LS_1, (b) LS_2, (c) LS_3, (d) LS_4

not as vulnerable as the column response. So for this bridge, it is clear that the column is the most vulnerable component of the bridge in regards to checking the bridge performance at any damage state. Such relative vulnerabilities can change for different design parameter combinations.

This fragility analysis method can be useful in a seismic design context as it gives the design engineer performance based information on their design. They can use this performance based information during the course of their design to determine the acceptability of the design to performance criteria. With the BSFM, a probabilistic analysis giving the expected performance of a new bridge design can be readily integrated into the bridge design process. As a validation of this new method, a comparison of the results of BSFM with the more established fragility analysis method (Ramanathan 2012) using Monte Carlo simulation showed a good correlation. More details about the comparison of BSFM with other fragility methods can be found in (Dukes 2013).

4. Effect of bridge design parameters on the fragility

Presented in this section is a sensitivity study investigating the effects of the design parameters on the fragility estimation. Each design parameter is varied one at a time while keeping the others at a median value (Table 1). The results of the sensitivity analysis are shown as tornado diagrams in Fig. 4. Although the fragility is most sensitive to variation in the IM, the bridge design parameters longitudinal ratio of steel reinforcement (X_1), volumetric ratio of transverse reinforcement (X_2), and span length to column height ratio (X_5) all have a significant influence especially in limit states LS_3 and LS_4. The aspect ratio of columns (X_3) and superstructure depth to column dimension ratio (X_4) have less impact on the system fragility in all of the limit states considered.

5. Conclusions

The seismic bridge design process used in California details the minimum requirements of a bridge design that will result in a structure able to withstand the Design Seismic Hazard (DSH) level without collapse. This emphasis on collapse prevention for a target hazard level is not unique to California and represents the current state of practice. However, the process does not include a way to determine the expected performance of the bridge at the DSH level or at other hazard levels. This research introduced a bride-specific fragility method that provides probabilistic fragility information describing the performance of a new bridge design. Design details included in the fragility method were those found to have a significant effect on the response of the bridge during an earthquake.

The Bridge Specific Fragility Method (BSFM) presented advances analytical or simulation based fragility analyses to derive parameterized fragility models that can help to reveal the fragility of a particular design permutation or set of permutations with efficiency. The traditional probabilistic seismic demand model (PSDM) was modified to accommodate design parameters as input variables, creating a multi-parameter PSDM. These models utilize a Latin hypercube sampling for the sampling strategy with a response surface method for model formulation. Bridge component capacity models are adopted that correspond to Caltrans specific inspection and repair levels for use in deriving bridge component and system fragility models. Parameterized fragility estimates are generated following logistic regression for bridge design specific application.

An illustration of the method is provided using a common bridge type found in California, the two span concrete box girder bridge. The values of the bridge design details used in the example corresponded to the median values of a bridge sample taken by Dukes (2013). Sample 2D fragility curves are shown for primary and secondary bridge components as well as the overall bridge system. This example shows the utility of the method and illustrates how the information can be used in the seismic design process to rapidly provide a more performance-based analysis of a new bridge design. The sensitivity study reveals that longitudinal ratio of steel reinforcement, volumetric ratio of transverse reinforcement, span length to column height ratio are the most significant ones affecting the system fragility. Although BSFM is demonstrated for single frame Box girder bridge, the methodology is relevant and applicable to other structural systems. However, a sensitivity study to identify important parameters is required before applying the methodology to other structural systems. Furthermore, opportunities exist to refine the adopted capacity models such that they are also more refined as a function of the design parameters, including extensive experimental testing required to adequately validate such models.

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