Probabilistic Vulnerability Scenarios for Horizontally Curved Steel I-Girder Bridges Under Earthquake Loads

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Horizontally curved steel I-girder bridges are located in all seismic zones in the United States. Research has shown that damage can occur to steel bridge components under earthquake loads. Probabilistic-based techniques are one tool that can be used to assess more accurately the seismic vulnerability of curved bridges for various damage states and at various seismic hazard levels. To examine probabilistic-based vulnerability criteria efficiently, the study used response surface metamodels (RSMs) in conjunction with Monte Carlo simulations to generate horizontally curved steel I-girder bridge fragility curves. The generated curves were then used to evaluate bridge damage in terms of previously published structure damage states. The use of RSMs reduces the required number of computer simulations needed to generate the fragility curves. The paper summarizes the fragility curve generation procedure for a group of horizontally curved steel I-girder bridges using RSMs in association with Monte Carlo simulation. Probabilistic vulnerability scenarios are presented via application to existing horizontally curved steel bridges located in Pennsylvania, New York, and Maryland to estimate seismic demands for those bridges and to generate fragility curves.

Horizontally curved steel I-girder bridges are located throughout the United States, including areas that are considered to be seismically active. As the number of curved steel bridges located in seismic zones continues to increase, the need to better understand their seismic behavior also increases. Past research related to this topic has largely been computational and rather focused (*1–4*). To date, research has not been expanded to the level of assessing curved steel I-girder bridge seismic vulnerability for various damage states (*5*) and at various seismic hazard levels using probabilistic techniques.

Assessing seismic vulnerability at various damage states using probabilistic-based fragility curves is a common approach when examining bridge seismic performance (*6–9*). A fragility curve provides a conditional probability that a bridge will meet or exceed a certain damage level for a given ground motion intensity. Recent research in the United States has focused on developing fragility curves for straight steel bridges (*6, 7, 9*) but curved steel bridges have yet to receive extensive research focus, and, because they have appreciable torsional behavior, tools developed for evaluating straight steel bridges cannot be accurately applied to curved bridges. Therefore, any method that can efficiently and probabilistically evaluate the seismic vulnerability of curved bridges is beneficial. The research described in this paper used response surface metamodels (RSMs) in conjunction with Monte Carlo simulations to examine probabilistic-based vulnerability criteria and generate fragility curves for horizontally curved steel I-girder bridges.

RSMs allow for complex structural behavior to be characterized using less complicated computational models than traditionally have been generated [e.g., finite element models (FEMs)]. RSMs typically employ second-order polynomial functions and least-squares regression techniques to fit system responses to a response surface. Recently, RSMs have been used in connection with probabilistic approaches (e.g., first-order reliability method, Monte Carlo simulation, etc.) to generate seismic fragility curves for populations of steel and reinforced concrete civil engineering structures, but not bridges (*10, 11*).

In this study, RSMs and Monte Carlo simulations were used to develop fragility curves based on statistics supplied from a group of horizontally curved steel I-girder bridges located in Pennsylvania, New York, and Maryland. Design of experiments (DOE) approaches, additional statistical tools, and the generated RSMs were then used to efficiently predict seismic response for a range of conditions. Fragility curves for horizontally curved steel I-girder bridges were then developed using these RSMs in conjunction with Monte Carlo simulations. The more specific discussion of the scenarios of how the fragility curves were developed follows.

PROBABILISTIC VULNERABILITY SCENARIO STUDY USING RSMs

RSMs are statistically influenced polynomial functions that define a multidimensional response surface. The generated response surface can represent and replace more complicated and time-consuming computational models (e.g., FEMs). RSMs were originally developed to analyze results from physical experiments to create empirically based models of the response. They can generally be described as shown in Equation 1:

$$y(x) = f(x) + \epsilon \tag{1}$$

where

$y(x)$ = unknown function of interest,
$f(x)$ = known polynomial function of x, and
ϵ = random error, which is assumed to be normally distributed with a mean of zero and a variance of σ^2.

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Transportation Research Record: Journal of the Transportation Research Board, No. 2202, Transportation Research Board of the National Academies, Washington, D.C., 2010, pp. 206–211.
DOI: 10.3141/2202-24

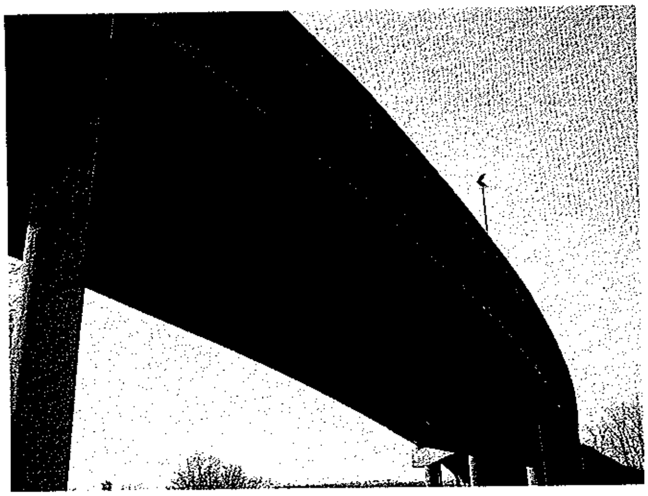

FIGURE 1 Representative curved steel I-girder bridge.

The individual errors, ϵ_i, at each observation are assumed to be independent and identically distributed. More details on RSMs as they relate to earthquake engineering can be found in Franchin et al. (*11*).

For the current study, the generated surface is used to predict seismic response and generate seismic fragilities for a select bridge group. Along with being used as a tool to address a specific research objective, an RSM can also refer to the process for developing the model and determining the polynomial coefficients. DOE techniques, which assist with determining which simulations or physical experiments should be examined when resources are limited, are usually employed to assist with determining coefficients for developing the RSMs. For the work described here, developed RSMs were used in conjunction with Monte Carlo simulations, which obtained an approximate probability distribution for the desired outcomes (i.e., seismic response) for the study group, to determine horizontally curved steel I-girder bridge fragilities for a group of existing bridges located in Pennsylvania, New York, and Maryland. Bridges that were examined were determined to be in a low to moderate seismic zone with a 10% peak ground acceleration (PGA) and 2% probability of exceedance for a 50-year recurrence interval (*12*). Figure 1 shows a representative curved steel I-girder bridge. The RSM procedure used for calculating the fragilities is shown in Figure 2 and is summarized below.

The first step in Figure 2 is defining the input and output parameters for the RSMs. The input parameters for target horizontally curved steel I-girder bridges were identified based on experience and bridge group data and consisted of macrolevel (e.g., geometric and structural) and microlevel (e.g., material) parameters. The range of each input parameter was set for the selected region of interest based on available data from the bridge group, which consisted of 99 existing bridges, and was used to generate experimental combinations based on DOE techniques.

The second step in Figure 2 starts with screening of selected structural parameters to identify statistically significant "optimal" ones that assisted with defining the RSMs. Optimal parameters were ones shown to heavily influence seismic response of target three-dimensional curved bridge models created in Open System for Earthquake Engineering Simulation (OpenSees) (*13*). DOE was then used to generate statistically significant metamodels to be examined under an ensemble of ground motions for RSM development. Central composite design (CCD) was selected to assist with determining appropriate combinations of input parameters, such as geometric-, structural-, and material-level parameters, for generation of the RSMs. The experimental application of CCD consists of a complete $2^k + 2k + 1$ statistical design, where k represents the number of input parameters and is set to −1, 0, and +1 values to represent lower-, center, and upper-level values from a given data set. The experimental CCD can then produce efficient bridge combinations at the three parameter levels

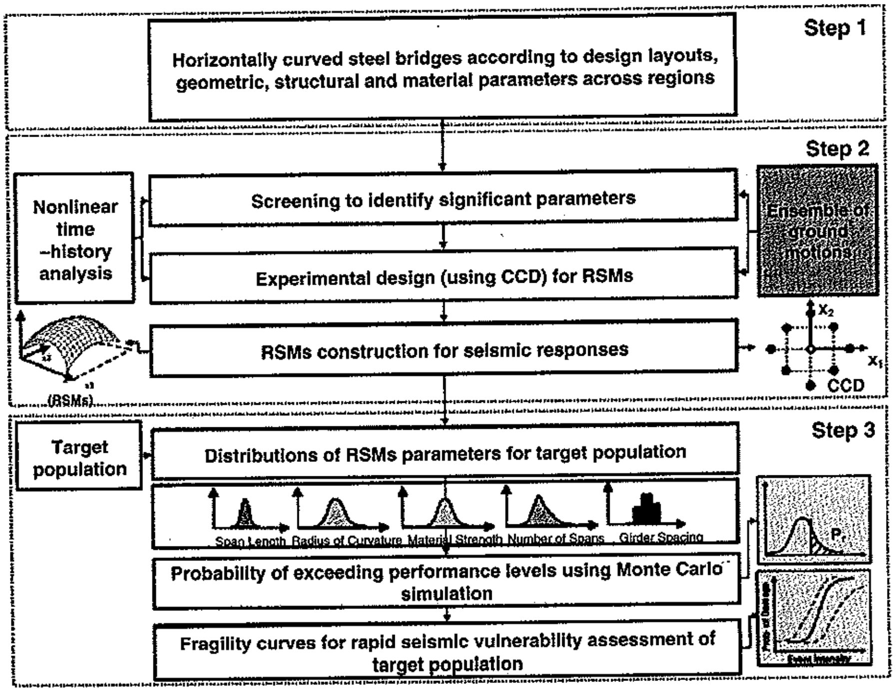

FIGURE 2 Probabilistic vulnerability scenarios development using RSMs.

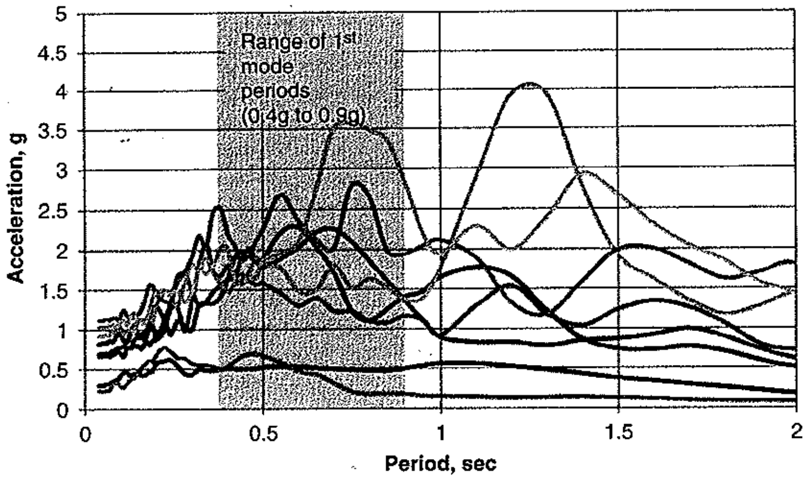

FIGURE 3 Response spectra for synthetic ground motion suites indicating first mode range for bridge group.

for each curved bridge input parameter that was considered. The representative ground motion ensemble applied to the models was randomly selected based on previously published research (*14*). Figure 3 shows representative response spectra for the synthetic ground motion suites and indicates the first mode period range for the examined bridge group.

Outputs from the OpenSees nonlinear time–history analyses for each statistically significant input combination were used to populate the seismic performance data set, with output information of interest being based on defined damage states, damage indices, or peak structural response parameters available in the literature (*5, 15*). Peak column curvature ductility and radial and tangential bearing and abutment deformations were used as the output variables. The column parameter was selected based on the bridge group statistical analysis, which indicated that the most common substructure pier type was a multicolumn pier. These quantities were selected as main evaluation parameters because they are identified as key elements for seismic bridge evaluation based on Federal Emergency Management Agency (FEMA) HAZUS-MH loss assessment criteria (*5*).

The final RSM polynomial function was computed from the selected output and input combination using least-squares regression. The final mathematical second-order polynomial RSM can be expressed as shown in Equation 2:

$$y = \beta_0 + \sum_{i=1}^{k} \beta_i x_i + \sum_{i=1}^{k} \beta_{ii} x_i^2 + \sum_{i=1}^{k-1} \sum_{j>i}^{k} \beta_{ij} x_i x_j + \epsilon \qquad (2)$$

where

y = dependent variable, such as seismic response;
x_i, x_j = independent input parameters, such as radius of curvature;
β_0, β_i, β_{ii}, and β_{ij} = coefficients to be estimated; and
k = number of input variables.

β parameters are calculated using least-squares regression to fit the response surface approximations to empirical data or to data generated from the nonlinear time–history analyses.

The final step in Figure 2 involved estimating the desired seismic response parameters (e.g., peak tangential bearing deformations) of a selected portfolio of curved bridges using the developed, and statistically significant, RSM functions. Because the resulting RSMs were already known to be statistically significant based upon the initial analyses completed to generate them, the process eliminated a large number of additional nonlinear time–history analyses to develop the curves. The probability of the chosen response to exceed certain, predefined, damage limit states was then extracted from the distribution of the simulation results using Monte Carlo simulations. This probability value was conditioned relative to a specific earthquake intensity level (e.g., PGA) and represented 1 point on a fragility curve. Repetition of the process over a specified range of earthquake intensities provided probabilities of exceedance for a range of intensity levels, and the entire fragility curve was created.

RSM CONSTRUCTION

The RSM construction process is shown in detail in Figure 4. A combination of CCD and nonlinear time–history analyses was used for generating the curved bridge RSMs. Because both single-variable effects and the effects of interaction of variables were considered, a three-level CCD approach was employed with each variable considered at three values, its upper bound (+1), its center value (0), and its lower bound (−1). Variables that were considered were selected based on the statistical inventory analyses, and Table 1 displays the five optimal parameters and their corresponding upper-, center, and lower-level values obtained from a statistical analysis of the bridge group design plans.

As mentioned above, a randomly extracted suite of synthetic ground motions was applied in OpenSees to representative curved bridge models containing the optimal parameters. The suite of synthetic ground motions was scaled to have an average PGA of 0.1g, 0.55g, and 1.0g to examine seismic response for a broader range of earthquake scenarios. The three-level CCD spaces for the five parameters from Table 1 and a single earthquake intensity level parameter (X_{eq}) are shown in Table 2. Forty-five bridge models were generated corresponding to the combinations of the optimal parameters and earthquake intensity level parameters outlined in Table 2 by running a CCD experimental design in the commercially available JMP statistical analysis program (*16*).

To demonstrate how a response computation was achieved for each experimental CCD design combination, one specific combination is

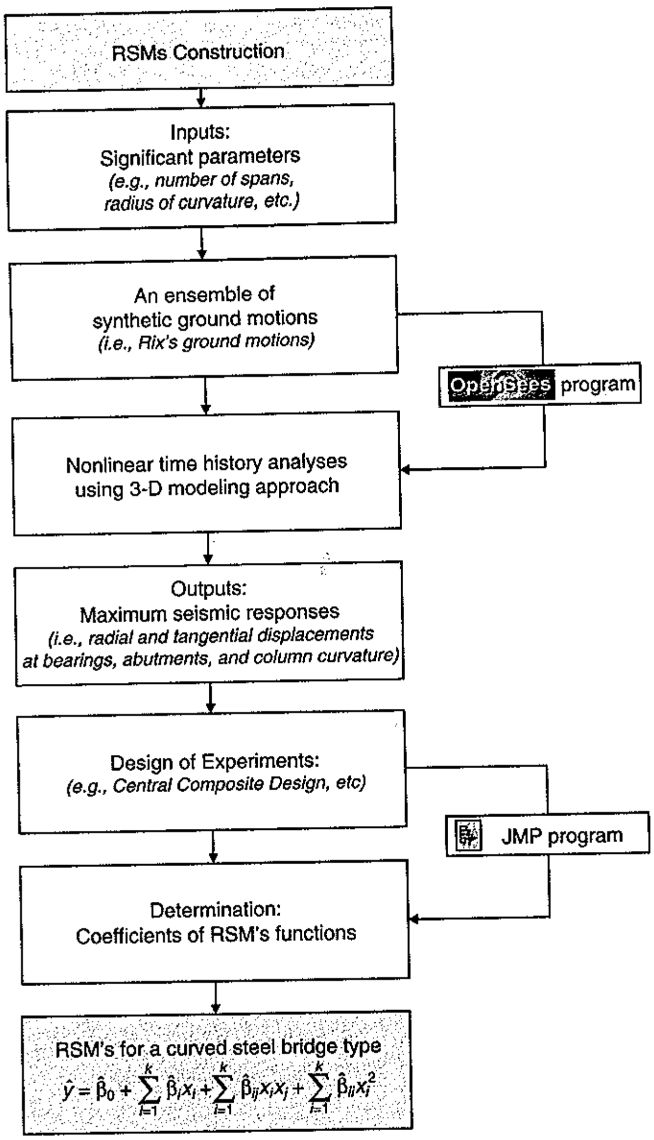

FIGURE 4 RSM construction process.

discussed. It contained the following input variables: $X1 = -1, X2 = 1, X3 = -1, X4 = -1, X5 = -1$, and $X_{eq} = -1$. This variable set was interpreted as a simple span curved steel bridge with a maximum span length of 91.4 m (300 ft), a radius of curvature of 240 m (787.7 ft), a girder spacing of 1.46 m (4.8 ft), a cross-frame spacing of 2.7 m (8.8 ft), and a suite of synthetic ground motions with an average PGA of 0.1g. All other curved steel bridge parameters that are deemed less influential are fixed at their mean values as identified from statistical examination of the curved bridge plan group. Nonlinear time–history analyses were performed of the bridge representing these data points in OpenSees, and maximum column curvature and maximum radial and tangential deformations at the bearings and abutments were computed for the time history. Mean and standard deviations of the deformations and curvature ductilities were then extracted. This process was repeated for each bridge pattern in Table 2 with resulting values used to generate the RSMs via statistical analyses.

With the use of a least-square regression analysis of each CCD table, RSM functions consisting of the optimal parameters in association with given PGAs were developed. The generated RSM functions for the output parameters can be shown symbolically and mathematically in Equation 2. The equation is composed of two response surface models, $\hat{y}_\mu$ and $\hat{y}_\sigma$. In these models, a normal distribution is assumed and the first term predicts an expected or mean value of the critical seismic response due to a suite of synthetic ground motions, while the second term represents earthquake-to-earthquake dispersion for the response computations and, consequently, incorporates randomness with respect to earthquake excitations.

$$\hat{y}_i = \hat{y}_{\mu|i} + N\left[0, \hat{y}_{\sigma|i}\right] \quad (3)$$

where

$i = 1, \ldots, 5$;
$\hat{y}_{1,\ldots,5}$ = maximum tangential–radial deformation at abutment–bearing and curvature ductility of column;
$\hat{y}_{\mu|1,\ldots,5}$ = mean value of maximum tangential–radial deformation at abutment–bearing and curvature ductility of column; and
$N\left[0, \hat{y}_{\sigma|1,\ldots,5}\right]$ = normal distribution plus one standard deviation for maximum tangential deformation at an abutment.

FRAGILITY CURVE GENERATION

A brief summary of fragility curve generation is provided here with information for the curves being taken from plans of the existing horizontally curved steel I-girder bridges located in Pennsylvania, New York, and Maryland. Bridges selected for the work that is shown differed with respect to the five optimal parameters listed in Table 1.

To develop fragility curves, the seismic demands of each component (e.g., column ductility curvature) were computed by using its RSMs. Each demand was evaluated against a corresponding performance level using S_c, defined as the median value of an intensity measure for the chosen performance level selected from information provided by FEMA (5). Table 3 lists descriptions of the performance

TABLE 1 Optimal Parameters for RSM Models

ID	Most Significant Parameter	Lower Bound −1	Center Bound 0	Upper Bound 1
$X1$	Number of spans	1	2	3
$X2$	Maximum span length, m (ft)	15.2 (50.3)	53.3 (174.9)	91.4 (300)
$X3$	Radius of curvature, m (ft)	240 (787.4)	1,866 (6,122.0)	3,492 (11,456.7)
$X4$	Girder spacing, m (ft)	1.46 (4.8)	2.35 (7.7)	3.23 (10.6)
$X5$	Cross-frame spacing, m (ft)	2.7 (8.8)	5.005 (16.4)	7.31 (24)
X_{eq}	Peak ground acceleration (g)	0.1	0.55	1

TABLE 2 Sample CCD Spaces

Pattern	$X1$	$X2$	$X3$	$X4$	$X5$	X_{eq}
1	−1	1	−1	−1	−1	−1
2	−1	1	1	−1	1	−1
3	−1	1	1	−1	−1	1
4	0	0	0	0	0	1
...	...	...	...	...	....	...
41	−1	−1	1	1	1	−1
42	−1	−1	−1	−1	1	−1
43	0	0	0	0	0	0
44	0	−1	0	0	0	0
45	0	0	0	−1	0	0

NOTE: −1 = lower bound, 0 = center bound, and +1 = upper bound.

states. For the representative combination described previously—a single-span bridge with a maximum span length of 91.4 m (300 ft), a radius of curvature of 240 m (787.7 ft), a girder spacing of 1.46 m (4.8 ft), and a cross-frame spacing of 2.7 m (8.8 ft)—slight damage was estimated at the abutment and bearing for tangential deformations while moderate damage was estimated for radial deformations at the bearing and for the columns based on their ductility curvature values. Monte Carlo simulations using 10,000 trial runs, deemed an efficient number to accurately estimate an exceedance probability from previous research (*8*), were performed on the RSMs associated with previously discussed optimal parameters. To appropriately compute seismic demands using the RSM simulations, each optimal parameter's probability density function (PDF), having a mean and standard deviation determined from a statistical analysis, was applied to the corresponding RSMs. PDF plots for each of the five optimal parameters from Table 1 are shown in Figure 5. Based on these PDFs, a discrete distribution was used for the number of spans (*X1*), a normal distribution was used for cross-frame spacing (*X5*), and lognormal distributions were used for the remaining parameters. Uncertainty with respect to nonoptimal parameters related to bridge capacity, which included concrete compressive strength, steel strength, and other similar parameters, was not considered in the seismic fragility analysis.

A representative set of seismic fragility curves is shown in Figure 6 for one of the output parameters that was examined, tangential

TABLE 3 Performance Levels (*5*)

Performance Level	Description
Slight	Minor cracking and spalling to the abutment, cracks in shear keys at abutments, minor spalling and cracks at hinges, minor spalling at the column (damage requires no more than cosmetic repair) or minor cracking to the deck.
Moderate	Any column experiencing moderate cracking (shear cracks) and spalling (column structurally still sound), moderate movement of the abutment (<2 in.), extensive cracking and spalling of shear keys, any connection having cracked shear keys or bent bolts, keeper bar failure without unseating, rocker bearing failure or moderate settlement of the approach.
Extensive	Any column degrading without collapse—shear failure—(column structurally unsafe), significant residual movement at connections, or major settlement approach, vertical offset of the abutment, differential settlement at connections, shear key failure at abutments.
Complete	Any column collapsing and connection losing all bearing support, which may lead to imminent deck collapse, tilting of substructure due to foundation failure.

deformation at the bearings. These fragility curves were generated by using computed maximum tangential deformations at the bridge supports. They detail the likelihood of different damage levels being reached as a function of PGA with each curve being defined based on information from FEMA (*5*). As shown in the figure, the fragility curves for each damage level reach an exceedence probability of 1 at different PGAs, with the slight damage curve reaching 1 for PGAs between $0.24g$ and $1g$, the moderate curve reaching 1 for PGAs between $0.6g$ and $1g$, and so on. Via comparisons between this curve and other similar curves generated for the previously described output parameters, components that are more sensitive to damage at lower PGAs can be identified. Details on other curves that were generated can be found in Seo (*17*).

CONCLUSIONS

The study described herein presented a probabilistic-based approach to generating fragility curves for a group of horizontally curved steel bridge structures using RSMs in association with Monte Carlo

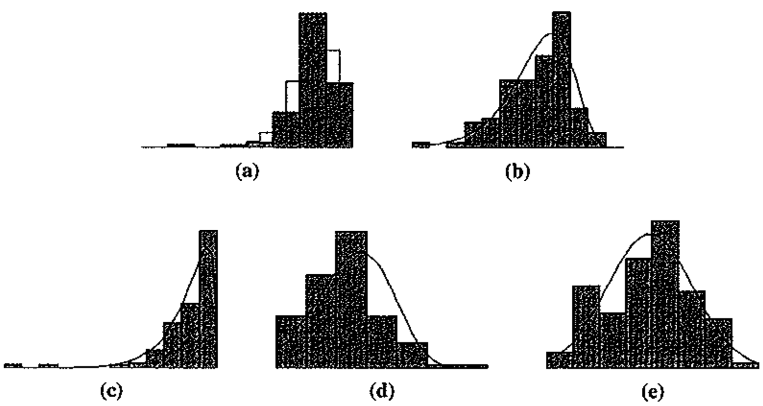

FIGURE 5 PDFs for optimal parameters: (*a*) number of spans (*X1*), (*b*) maximum span length (*X2*), (*c*) radius of curvature (*X3*), (*d*) girder spacing (*X4*), and (*e*) cross-frame spacing (*X5*).

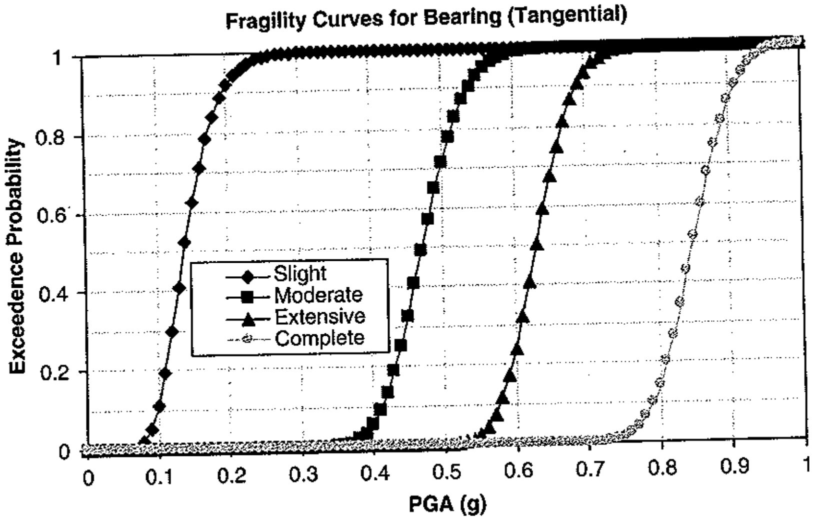

FIGURE 6 Representative seismic fragility curves for bearing tangential deformation.

simulations. The benefits of using this approach to examine seismic vulnerability for a group of existing bridges are related to the computational efficiency with which the fragility curves can be generated via the elimination of a large number of finite element analyses. Owners may be assisted with decision-making processes related to bridge seismic retrofitting as a result of the generated fragilities. They can help highlight the sensitivity of critical curved bridge components to ranges of seismic demands. The application of this method was demonstrated via the generation of a representative sample set of fragility curves for bearing tangential deformations for the bridges that were studied. Bearing tangential deformations appeared to be somewhat susceptible to the examined seismic loads at most damage states for the bridges that were studied, with slight bearing damage possibly occurring during low to moderate seismic events. Based on these findings, an investigation that involves retrofitting bearings to address the anticipated tangential displacements may be warranted. Results from generated fragility curves also revealed that radial bearing deformations were found to be the most vulnerable of the parameters that were studied for the curved steel bridges that were examined. So, in addition to considering retrofitting bearings to better resist anticipated seismic tangential deformations, consideration should be also be given to radial deformations.

ACKNOWLEDGMENT

The authors acknowledge the Korea Institute of Energy and Resources Technology for funding this work.

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The Seismic Design and Performance of Bridges Committee peer-reviewed this paper.