Seismic fragility analysis of irregular continuous rigid frame girder bridge

Fan Chen¹,², Xiaoyu Gu¹*, Deshan Shan¹, Jun Dong¹ and Qiao Li¹

Abstract: In this paper, probabilistic seismic demand analysis is adopted to investigate the seismic fragility of a certain typical irregular highway bridge in the Chinese western mountainous areas. Considering the uncertainties of ground motion and structural parameters, the sample library for seismic fragility analysis is constructed aided by Latin hypercube sampling method, and the nonlinear time history analysis for each sample is implemented to achieve the structural dynamic response under the seismic excitation. Moreover, the curvature ductility ratio and the relative displacement are regarded as the damage indexes for pier and bearing, respectively. Furthermore, the fragility nephograms for the whole piers and fragility curves for various critical sections with different damage states are obtained based on a lognormal distribution assumption and the reliability theory. Finally, the bridge system fragility curves are generated through the first-order reliability analysis method. Fragility curves obtained from the above proposed technique can be used to evaluate the seismic performance of irregular bridges, and it can lay a basis for post-earthquake damage assessment.

Subjects: Seismology; Structural Engineering; Transportation Engineering; Life-Long Design

ABOUT THE AUTHORS

Xiaoyu Gu All the author and co-author are come from the smart bridge research group of Southwest Jiaotong University, China. One researching interesting for our group is bridge seismic fragility analysis, and the work presented in this paper is some part of our investigation.

PUBLIC INTEREST STATEMENT

Seismic fragility analysis is frequently adopted to evaluate the seismic performance of bridge structure, and the structural irregularity has a significant impact on the structural seismic performance. The bridges in Chinese western mountainous areas are generally irregular structure from the viewpoint of earthquake engineering. A certain typical irregular long-span continuous rigid-frame bridge with high-rising piers is taken as the case study in this paper to investigate its seismic fragility. The probabilistic seismic demand model and the incremental dynamic analysis are adopted to implement the fragility analysis, and the uncertainties for the ground motion and bridge structural parameters are also took into consideration. Different with the common used fragility curves, the fragility nephogram is innovatively proposed to describe the fragility of bridge component and system in this paper. The obtained fragility nephogram can provide the basis for seismic performance evaluation and post-earthquake damage assessment.
1. Introduction

The seismic zones are widely distributed and the seismic activities are frequent in the western mountain area of China (Lin, Hung, Liu, & Chai, 2010). The recent earthquakes, such as the 2008 Wenchuan Earthquake, 2010 Yushu Earthquake and 2013 Lushan Earthquake, occurred in this area. For spanning the complex terrain with high mountain, deep valley and crisscross ravines at western China, the bridge structures with long span and high-rise piers are widely adopted, especially for the three span continuous rigid frame girder bridge. Moreover, the difference between piers height is enormous, even in some cases up to 30–40m. Thus, the montane bridge structure is a typical irregular bridge structure from the viewpoint of bridge aseismic (Formisano, Di Lorenzo, Iannuzzi, & Landolfo, 2017). Nevertheless, there is no relatively reasonable and effective assessment methods for seismic performance for the bridges with more than 40m height pier (Chinese Industrial Standard JTG/T B02-01-2008) in Chinese current specification for bridge seismic design and maintenance.

With the advancement in the structural seismic theory, the seismic fragility analysis has become one of the important means to evaluate the seismic performance of bridge structure (Billah & Alam, 2015). There are two ways to develop the structural seismic fragility curves, namely empirical and analytical fragility analysis, respectively (Billah & Alam, 2015). Extensive and symmetric studies on the seismic fragility analysis of bridge structure are carried out by scholars around the world, and a comprehensive review of the different methodologies proposed for the seismic fragility analysis of highway bridges along with their features, limitations and applications is presented by Billah and Alam (2015). In this review, the key features of different available methods are summarized and listed in a series of concise tables, and the future development is also discussed.

In the irregular bridge structure with unequal pier heights, the seismic demands for each pier are usually different, while the shortest pier being subjected to maximum demand (Bi & Hao, 2011; Frankie, 2013). Fragility curves of 18 different bridge configurations with different pier height are generated by Akbari (2012), and it is figured out that the shortest-piers of the irregular bridge are extensively damaged while the higher-piers remain elastic under the action of high-intensity earthquake based on the capacity/demand ratio approach. Jara, Galván, and Olmos (2013) proposed a seismic vulnerability procedure for an irregular isolated bridge, and the impacts of two earthquake sources with different fault characteristics are discussed for the first isolated bridge with varied substructure height from 46.05 m to 70.75 m in Mexico (Jara, Madrigal, Jara, & Olmos, 2013). It is shown that the effect of the fault type on the bridge seismic vulnerability is significant. Then seismic vulnerability analysis of three typical irregular types of reinforced concrete simple supported bridges with medium-span located in the high seismicity zone of Mexico is adopted to determine the best strength and stiffness parameters of their isolation system by Jara et al. (2013). Furthermore, the strength and stiffness characteristics of substructure are taken as the interesting parameters, the seismic performance of irregular medium-span simply supported reinforced concrete bridges on soft and hard soils are discussed by Jara, Reynoso, Olmos, and Jara (2015), and the conclusion is drawn that the piers close to the tallest piers are the most affected components by the irregularity. The fragility curves with the uncertainties of earthquake, structural geometries and materials for a certain multi-frame concrete box-girder viaduct under four different kinds of irregularities are adopted by Abbasi, Zakeri, and Amiri (2015) to discuss their seismic behaviors, and it concludes that the bridge fragility increases with the degree of the altitudinal irregularity.

A few studies are related to the seismic vulnerability analysis of irregular bridge as the frequently applied long-span continuous rigid-frame girder bridge with high-rise pier (Billah & Alam, 2015), especially for the bridge with more than 30 m height difference between different piers.
In this paper, we investigate the seismic performance of irregular bridge structure with larger height difference of pier, and we discuss the seismic vulnerability of continuous rigid frame girder bridge with long span and high-rise pier in western China. The uncertainties of ground motion and bridge structural parameters are also taken into account, and the nonlinear dynamic time-history analysis is adopted to obtain the structural responses under the action of ground motion for each ground motion-bridge model. The probabilistic seismic demand analysis (PSDA) is utilized to develop the seismic fragility curves of bridge structural components and bridge structural system. The seismic fragility characteristics of irregular bridge structure are also discussed.

2. Seismic vulnerability analysis theory

2.1. Probabilistic seismic demand model

The fragility curve (Billah & Alam, 2015), which is the conditional probability that the structural components or system exceed a given damage state (DS) or a specific level of engineering demand parameter (EDP) with a certain given the intensity measure (IM) parameter, is determined ultimately in the framework of performance-based earthquake engineering (PBEE) (Billah & Alam, 2015; Cui, Zhang, Ghosn, & Xu, 2018). The EDP is the bridge structural response under the seismic excitation governed by the IM obtained from the nonlinear transient finite-element analysis. Probabilistic seismic demand model (PSDM) (Billah & Alam, 2015) is a conditional probability that any of the structural components or a structural system experiences a certain level of demand (D) for a given IM level, and it is one of the results for PSDA (Billah & Alam, 2015). PSDM is adopted to develop the seismic fragility curve based on the bridge structural dynamic time-history analysis in this paper. Engineering demand parameter (EDP) is assumed to meet the requirement of lognormal distribution, the statistical regression analysis is adopted by Hamburger, Jalayer, Foutch, and Cornell (2002) to investigate the relationship between the median of seismic demand $d_{EDP}$ of bridge structure or structural system and intensity measure IM of ground motion, and the exponential relationship between $d_{EDP}$ and IM is shown as follows:

$$d_{EDP} = a IM^b$$  \[(1)\]

where $a$ and $b$ are undetermined coefficients obtained by the regression analysis. Different structural demands are corresponding to the ground motions with different intensities, and the dispersion of the structural seismic demand parameter $\beta_{EDP|IM}$ is

$$\beta_{EDP|IM} = \sqrt{\frac{\sum_{i=1}^{N} (\ln(edp_i) - \ln(a IM_i^b))^2}{N-2}}$$  \[(2)\]

where $edp_i$ are the peak values of bridge structural seismic demand under the action of $i$th ground motion with the intensity measure $IM_i$, and $N$ is the total number of ground motions. After determining the parameters of lognormal distribution by regression analysis, the PSDM can be expressed as follows:

$$P[EDP \geq edp|IM] = 1 - \phi\left(\frac{\ln edp - \ln EDP}{\beta_{EDP|IM}}\right)$$  \[(3)\]

where $\phi(\cdot)$ is standard normal cumulative distribution function. Substituting Equation (1) into Equation (3), yields

$$P[EDP \geq edp|IM] = 1 - \phi\left(\frac{\ln edp - \ln a IM^b}{\beta_{EDP|IM}}\right) = \phi\left(\frac{\ln(IM) - \mu}{\xi}\right)$$  \[(4)\]

where $\mu = (\ln edp - \ln a)/b$ is the natural logarithm of the median for the seismic intensity measure under the condition with a given engineering demand parameter $EDP$; $\xi = \beta_{EDP|IM}/b$ is logarithmic standard deviation, which is directly adopted to evaluate the applicability of the intensity
parameter for the ground motion; smaller the \( \xi \) value is, the better will be the applicability of intensity parameter for the ground motion.

2.2. Structural component seismic fragility analysis

The seismic fragility of bridge structure or structural component is defined as the conditional probability that its seismic demand meets or exceeds its real seismic capacity for a given level of ground-motion intensity. Moreover, this probability can be expressed as the cumulative distribution function and the probability density function of the engineering demand parameters (Billah & Alam, 2015), from the statistics viewpoint, the cumulative distribution function of the conditional probability is defined as the fragility curves of bridge structure and structural component. The following fragility function is carried out along with the definition of structural component seismic fragility:

\[
F_{\text{fragility}} = P[EDP \geq C | IM]
\]  

(5)

where \( C \) is seismic capacity of bridge structural component. As shown in the above-mentioned PSDM, the structural seismic demand \( EDP \) can be achieved from the nonlinear time-history analysis of the bridge structure under the actions of a series of ground motions. When the distributions of \( EDP \) and \( C \) of bridge structural component are lognormal distribution, Equation (5) can be written as follows:

\[
P[EDP \geq C | IM] = \phi \left( \frac{\ln(S_d/S_c)}{\sqrt{\beta_{EDP,IM}^2 + \beta_C^2}} \right) = \phi \left( \frac{\ln(S_d/S_c)}{\beta_{\text{comp}}} \right)
\]  

(6)

where \( \beta_{EDP,IM} \) and \( \beta_C \) are the lognormal standard deviation of the seismic demand and capacity for bridge structural component, respectively; \( \phi(.) \) is standard normal cumulative distribution function; \( S_c \) is the median of structural seismic capacity in a certain limit state; and \( S_d \) is the median of structural seismic demand. \( \beta_{\text{comp}} = \sqrt{\beta_{EDP,IM}^2 + \beta_C^2} \) is the logarithmic standard deviation of the fragility function.

2.3. Structural system seismic fragility analysis

Actually, the seismic damage probability of bridge structural system is the joint probability distribution of seismic damage probabilities for different structural components. There are two conventional methods to obtain the joint distribution probability in terms of multivariate statistics: the first is the Mont Carlo method and the other is the joint probabilistic seismic demand method (Abbasi & Amiri, 2015; RamanathanPadgett & DesRoches, 2015). In the second method, after the determination of the correlation matrix for structural responses and the performance-related limit state equation of each component in the bridge structural system, the joint probability can be determined by the systemic fragility curve that can be obtained by joining all fragility curves of components (Siqueira, Sanda, Paultre, & Padgett, 2014). Both of the above-mentioned methods are computationally complex and intense to achieve the joint distribution probability. Then the first-order reliability theory (Ghosh, Rokneddin, Padgett, & Dueñas-Osorio, 2014; Rokneddin, Ghosh, Due As-Osorio, & Padgett, 2014) is adopted to figure out the damage probability of bridge structural system in this paper. Due to the mutual relationship, impacts between the different damaged components are ignored in the first-order reliability theory, and the first-order boundary of systemic damage probability can be written as follows (Morbin, Zanini, Pellegrino, Zhang, & Modena, 2015):

\[
\max_{i=1,...,m} |P(F_i)| \leq P_{\text{sys}} \leq 1 - \prod_{i=1}^{m} [1 - P(F_i)]
\]  

(7)

where \( P(F_i) \) and \( P_{\text{sys}} \) are the seismic damage probabilities of the \( i \)th component and the whole system.

2.4. Seismic fragility analysis processing

The uncertainties of ground motions and bridge structural parameters are taken into consideration, and the sample library of the seismic vulnerability model is created aided by the Latin
hypercube sampling method. Then nonlinear time history analysis for each bridge-ground motion sample in the model library is implemented to achieve the structural dynamic responses. After the damage indexes of the vulnerable bridge components are determined, the fragility curves of all the fragile components are figured out, respectively, in light of the PSDA, and then the systematic fragility curve is obtained by the first-order reliability theory. The seismic fragility analysis flowchart is figured out in light of the above discussion, as shown in Figure 1. In this figure, the key issues to ensure the accuracy of the seismic fragility analysis for the irregular bridge structure are listed as follows and will be discussed in Section 4.

(1) The damage index should be determined so that it can be used to characterize the structural damage state.

(2) Selected records of ground motion should contain all the features of ground motion as much as possible, such as soil-site type, and must be specified for a specific bridge location.

3. Typical long span irregular bridge in western China

A certain irregular continuous rigid frame girder bridge in China western region is taken as a case study to discuss the seismic vulnerability of the long-span irregular bridge structure with high-rise piers.

3.1. General description of bridge structure

Miaoziping bridge, which is located at the Duwen Expressway, is a long-span and high-rise pier bridge, the distance from the bridge site to the epicenter of Wenchuan Earthquake is 29 km; thus, the bridge was severely damaged when the earthquake occurred (Lin, Hung, Liu, and Chai, 2010). Therefore, the main bridge is taken as a case study to discuss the seismic performance of the long span and high-rise pier irregular bridge.

The span arrangement of the main girder is 125m + 220m + 125m. As shown in Figure 2, the single cell box girder with variable girder height is a three-span prestressed concrete continuous rigid frame. Identical rectangular hollow section is designated to the 2# and 3# main piers, and double-column thin-wall hollow pier is adopted by the both-sides junction piers. The concrete of...
girder, 2# and 3# piers is C60, and the concrete of the 1# and 4# is C40, respectively. The heights of 1#-4# piers are 67.45 m, 102.0 m, 99.5 m, and 85.42 m, respectively, and the maximal height difference is 34.55 m, i.e., approximately one third of the highest pier. Thus, the bridge is a typical irregular structure. Longitudinal reinforcement ratio and stirrup ratio of pier are 1.2% and 0.75%, respectively. Both longitudinal bars and stirrup are HRB335 rebar. The main girder is fixed on the top of 2# and 3# main piers, and GPZ10SX ± 200 bidirectional movable basin rubber bearings are installed on the top of 1# and 4# piers for connecting the main girder. The seismic fortification category is A-class in Chinese specification, and it means that the bridge structure falls in the category of top seismic protection structure. Moreover, the site soil of this bridge is II-type according to site soil classification in the relative Chinese specification, as shown in Table 1. As evident from the table, the equivalent shear wave velocity $v_{se}$ in site soil is adopted in Chinese specification to classify the site soil.

### 3.2. Numerical model

OpenSees (http://opensees.berkeley.edu/) is adopted to carry out the three-dimensional finite-element modeling of the long-span irregular bridge with high-rise pier, elastic beam element is used to simulate the main girder, and the self-weight and secondary dead-load are reckoned in. The elastic-plastic fiber element is adopted to simulate the piers, and the constitutive relations for the reinforcement and
concrete materials in the pier element are also defined. The constitutive relation for the concrete is described as the Kent–Scott–Park model (http://opensees.berkeley.edu/), and moreover the tensile strength of concrete is also taken into consideration, and the constitutive relation of reinforcement is defined by the Giuffre–Menegotto–Pinto model (http://opensees.berkeley.edu/).

According to Clause 6.3.7 of “Guideline for Seismic Design of Highway Bridge” (Chinese Industrial Standard JTG/T B02-01-2008, 2008), the bilinear ideal elastic-plastic spring element is adopted to simulate the basin rubber bearing. The equivalent stiffness $K_{eff}$ of bearing are determined by the shear modulus, the cross-sectional area and the thickness of the rubber layers. The elastic and plastic stiffness of bearing are $K_1$ and $K_2$, and then the yield force $F_y$ and displacement $D_y$ of the bearing are determined.

Soil–structure interaction is taken into consideration, and the dynamic impedance (shown as Table 2) at the interfaces between the piers and the foundations are derived according to the soil properties and the pile configuration (Stefanidou, Sextos, Kotsoglou, Lesgidis, & Kappos, 2017), and the expansion joints at the deck ends are modeled as a gap element. In Table 2, $K_{xx}$, $K_{yy}$ and $K_{zz}$ are the equivalent linear stiffness of pile foundation of the longitudinal, lateral and vertical direction, respectively; and $K_{rx}$, $K_{ry}$ and $K_{rz}$ are the equivalent rotational stiffness of pile foundation of the longitudinal, lateral and vertical direction, respectively.

## 4. Seismic fragility analysis of irregular highway bridge

### 4.1. Uncertainty

There are mainly two kinds of uncertainties in bridge structural model, namely epistemic and aleatory uncertainties (Au, 2014a, 2014b). In the bridge structural seismic fragility analysis, epistemic uncertainty is mainly on account of the impact of the bridge structural geometric and material properties, and the influence of ground motion is taken into consideration as the aleatory uncertainty. In this paper, both kinds of uncertainties are considered in the seismic fragility analysis of the irregular bridge structure.

#### 4.1.1. Uncertainty of ground motion

The design spectrum in the Chinese specification “Guideline for Seismic Design of Highway Bridge” (Chinese Industrial Standard JTG/T B02-01-2008, 2008) corresponding to a specific bridge site soil type

### Table 1. Site soil classification in Chinese specification

| Equivalent shear wave velocity $v_s/(m.s^{-1})$ | Classification |
| --- | --- | --- | --- | --- |
| $v_s$>500 | I | II | III | IV |
| $500 \geq v_s>250$ | <5 | ≥5 | | |
| $250 \geq v_s>140$ | <3 | 3–50 | >50 | |
| $v_s \leq 140$ | <3 | 3–15 | >15–80 | >80 |

### Table 2. Equivalent stiffness of pile foundation

| Parameter | Unit | $K_{xx}$ | $K_{yy}$ | $K_{zz}$ | $K_{rx}$ | $K_{ry}$ | $K_{rz}$ |
| --- | --- | --- | --- | --- | --- | --- | --- |
| 1# Pier | kN/m | $1.53E + 06$ | $3.89E + 06$ | $1.19E + 07$ | $3.17E + 09$ | $2.68E + 08$ | $9.31E + 09$ |
| 2# Pier | kN/m | $6.01E + 06$ | $5.13E + 06$ | $7.79E + 07$ | $6.06E + 09$ | $6.45E + 08$ | $8.89E + 09$ |
| 3# Pier | kN/m | $5.20E + 06$ | $5.65E + 06$ | $8.56E + 07$ | $6.48E + 09$ | $5.96E + 08$ | $7.79E + 09$ |
| 4# Pier | kN/m | $2.46E + 06$ | $4.24E + 06$ | $3.86E + 07$ | $4.12E + 09$ | $6.27E + 09$ | $1.01E + 10$ |
is taken as a target spectrum, and 100 ground motion records with average soil shear wave velocity
between 250 m/s and 500 m/s are chosen from the PEER (Pacific Earthquake Engineering Research
Center) Ground Motion Database (https://ngawest2.berkeley.edu/). The site-to-source distance of all
the chosen ground motion records is larger than 30 m, and without a velocity pulse of high energy. The
peak ground acceleration (PGA) is changed from 0.1 g to 1.2 g. The response spectra of all the selected
ground motion records are plotted in Figure 3. The mean of dynamic amplification factors for all the
selected ground motion records and specification recommendation spectrum corresponding to II-
type site-soil is shown in Figure 3. As shown in the figure, the mean spectrum is close to the
recommended spectrum. Moreover, the chosen ground motion records are taken as the longitudinal
input to investigate the longitudinal seismic fragility of bridge structure.

4.1.2. Uncertainty of bridge structural parameters
The uncertainties of material properties, component behaviors, and load effects are directly affected
by the seismic performance of the long-span continuous rigid frame girder bridge with high-rise pier;
thus, these kinds of uncertainties must be taken into consideration in the seismic fragility analysis.
Based on the structural properties of long-span continuous rigid frame bridge with high-rise pier, the
bridge damages survey reports about the Wenchuan earthquake (Lin et al., 2010) and existed
researches (Caltrans, 2010). The uncertain structural parameters and their respective distributional
characteristic parameters are determined for the fragility analysis, as shown in Table 3.

4.2. Determination of structural damage index
In accordance with FEMA (1999), FEMA (2008), and other researches (Sevim, Atamturktur,
Altunişik, & Bayraktar, 2016; Torbol, Gomez, & Feng, 2013), the structural seismic damage is
classified into four types: minor damage, moderate damage, severe damage, and collapse. Moreover, engineering demand parameters and their damage indexes for each component are determined as follows.

4.2.1. Damage index of pier
For an irregular continuous rigid frame girder bridge, the intermediate 2# and 3# piers are single column piers, and their top sections cannot be rotated freely due to the constraints of the superstructures and adjacent piers. Hence, the internal force distribution of pier column is more complex. The time-history of the contraflexure point of 2# pier under a certain ground motion record with 34.46km epicentral distance from the CHICHI/CHY087 station in the CHI-CHI earthquake (Deepu, Prajapat & Ray-Chaudhuri, 2014) is shown in Figure 4, and its corresponding statistical distribution is shown in Figure 5. As shown in these two figures, the contraflexure points are varied from 0 m to 96 m, and the mean and median of contraflexure points are 13.93 m and 9.33 m, respectively. This indicates that the computational position of contraflexure point is unstable; thus, it is unfeasible to regard this kind of pier as a cantilever pier in the seismic performance analysis. On the other hand, the displacement of the top section of pier is not synchronal with the curvature of critical section, and there is no consistent balance between each material damage and each deformation (Siqueira et al., 2014). Therefore, the displacement and displacement ductility ratio are not applicable to the damage index of pier for this particular bridge structural type.

The envelope curves of maximal curvature along the pier height of 1# ~ 4# pier under the action of 50 selected ground motions are figured out, and only envelope curves of curvature for 1# and 2# piers are shown in Figure 6. The curvature envelope curves for 3# and 4# piers are similar to that of the 1# and 2# piers. As shown in the figure, it can be deduced that even though the higher-order has impact on the distribution mode of pier curvature, the probable positions of plastic damage for side piers are located at the bottom regions. Meanwhile, the probable positions of plastic damage are located at the top and bottom regions in the case of intermediate piers. Furthermore, this
phenomenon is same as actual seismic damage of high-rise pier (Lin et al., 2010). Therefore, the curvature of pier is regarded as the damage index to describe the damage characteristics in this paper. It means that the critical sections of piers in the seismic fragility analysis are the top and bottom section of intermediate 2# and 3# piers and the bottom sections of both-side 1# and 4# piers. The description of pier damage state and calculation method of its damage indices are shown in detail in the literature (Sevim et al., 2016; Torbol et al., 2013), and the calculated results of damage indexes for each pier are shown in Table 4.

4.2.2. Damage index of bearing

Bridge bearing is one of the vulnerable components in bridge structure, and its related damage is one of the common bridge structural damages in seismic activity. Thus, the damaged bearing has a direct impact on the bridge structural normal operation state, and it is necessary to investigate the seismic fragility of bearing. In accordance with the existing researches (Billah & Alam, 2015), the damage extent of bearing is directly related to its deformation, and the deformation of bearing is frequently regarded as a measure of its damage extent index. Combining the “Pot Bearings for Highway Bridge” specification (JT/T4-2004) with the real bridge condition and relative parameters of selected bearing, the relative displacement is taken as the damage index for bearings.

Pot bearings are installed on the tops of junction piers for the discussed irregular bridge structure in this paper, and the allowable longitudinal displacement for bridge bearings is 0.2 m from the design point of view. Even though few investigations for quantifying the seismic damage extent of pot bearing are carried out, the quantification of the bearing damage index is also figured out in accordance with the related literature (Sevim et al., 2016; Siqueira et al., 2014; Chen et al., 2018).

Table 4. Calculated value of damage index for bridge components with different damage states

| Structural position | Damage index | Unit | Damage states |
|---------------------|--------------|------|---------------|
|                     |              |      | Minor | Moderate | Severe | Collapse |
| Bottom section of 1# Pier | Curvature | – | 0.00037 | 0.00052 | 0.00152 | 0.00302 |
| Top sections of 2#, 3# piers | Curvature | – | 0.00013 | 0.00016 | 0.00268 | 0.00415 |
| Bottom sections of 2#, 3# piers | Curvature | – | 0.00009 | 0.00012 | 0.00216 | 0.00389 |
| Bottom section of 4# Pier | Curvature | – | 0.00036 | 0.00050 | 0.00139 | 0.00272 |
| Bearing of 1# and 4# Piers | Displacement | m | 0.2 | 0.4 | 0.6 | 0.8 |
Torbol et al., 2013). In this paper, four different relative displacements, such as 0.2 m, 0.4 m, 0.6 m and 0.8 m, respectively, as shown in Table 3, are regarded as the limit states corresponding to the four damage states in the seismic fragility analysis of bearing.

4.3. Structural probabilistic seismic demand analysis

The random sampling method is adopted to construct the sample library of ground motion-bridge structure, and the nonlinear time-history analysis for each sample of ground motion-bridge structure is carried out by OpenSees finite-element software. Then the analysis of structural probabilistic seismic demand is implemented based on computational results. Specific analysis processing is as follows.

1. In light of uncertainty parameters shown in Table 3, the Latin hypercube sample method (Billah & Alam, 2015) is adopted to obtain the samples of these parameters, and then the bridge structural sample library is established. In view of the computational scale and adopted sampling method, 100 samples are engaged to generate 100 samples of bridge structure.

2. 100 selected ground motion records and 100 bridge structural samples are paired one by one to construct the sample library of the ground motion-bridge structure. Then nonlinear dynamic time-history analysis for each sample in the library is carried out, and the maximal dynamic responses of the seismic critical sections for each pier and bearing are obtained. Moreover, seismic critical section and component damage index, as shown in Table 5, are determined for long-span continuous rigid frame girder bridge with high-rise pier.

3. The maximal dynamic responses and corresponding peak value of ground motion are determined, and their scatterplot is figured out in the logspace. Then the coefficients $a$, $b$ in Equation (1) and $\beta_{EDP,IM}$ in Equation (2) are obtained by the regression analysis.

On basis of the above analysis process, the seismic demand analysis for each component in the bridge structure is carried out, and the relationship between the seismic demand model of each component and intensity measure of ground motion is also determined. The computational results for each component under the action of ground motion are shown in Table 6.

4.4. Bridge structural seismic fragility analysis

4.4.1. Seismic fragility analysis of components

In order to investigate the distribution of seismic fragility along with the pier height, the fragility nephogram is adopted to present the variation characteristics of seismic damage probability changes with the intensity measure of ground motion (such as PGA) for each cross-section of the entire pier. Moreover, the damage diffusion range and the damage probability of pier under the action of a certain given ground motion with specific intensity measure can be obtained directly from the fragility

| Table 5. Seismic critical section and component damage index table |
|-----------------------|------------------|------------------|
| No. | Position of critical section | Damage index | Ground motion |
|-----|--------------------------------|-------------|--------------|
| 1   | 1# Pier bottom section         | Curvature   | $\phi_{1-a}$ |
| 2   | 2# Pier top section           | Curvature   | $\phi_{2-t}$ |
| 3   | 2# Pier bottom section        | Curvature   | $\phi_{2-b}$ |
| 4   | 3# Pier top section           | Curvature   | $\phi_{3-t}$ |
| 5   | 3# Pier bottom section        | Curvature   | $\phi_{3-b}$ |
| 6   | 4# Pier bottom section        | Curvature   | $\phi_{4-b}$ |
| 7   | 1# Pier bearing               | Relative displacement | $\Delta_{1-z}$ |
| 8   | 4# Pier bearing               | Relative displacement | $\Delta_{4-z}$ |
nephogram. Furthermore, the structural damage probability is presented by different colors in the nephogram, and evolution processing of seismic damage for the entire pier can be manifested very well by such kind of presentation. The relationship between the variation of seismic intensity and the diffusion speed of pier damage can be explored from the fragility nephogram also. Hence, the fragility nephogram is a simple, intuitive, articulate and understandable way to show the distribution characteristic of seismic fragility for the entire pier. Therefore, the fragility nephogram is adopted to investigate the seismic fragility analysis of the complete pier.

Fragility curves for all cross-sections of the four piers are achieved according to Equation (6), and the fragility nephogram of the whole pier is composed of all fragility curves for all cross-sections associated to the same pier.

The fragility nephograms of 1# side-pier with the minor and moderate damage states are shown in Figure 7, and the PGA of ground motions is varied from 0 g to 1.2 g in this figure. As shown in Figure 7, the bottom region of side-pier is most prone to damage, and the pier will be damaged gradually along the pier from the bottom to top with the increasing PGA. The seismic damage probability of the pier bottom within 25 m height for the 1# pier with minor damage state is over 50% when PGA is higher than 1.2 g; meanwhile the seismic damage probability of the pier bottom within 15 m height for this side pier with moderate damage state is over 50% as well.

The fragility nephograms of 2# intermediate pier with the minor and moderate damage states are shown in Figure 8. The bottom and top regions of this intermediate pier are the vulnerable regions, and the pier will be damaged gradually along the pier from the both ends towards middle with the increasing PGA. Meanwhile, under the action of same intensity measure of ground motion, the seismic damage probability of the pier bottom is higher than that of the pier top.

Comparing the nephograms of 1# side-pier and 2# intermediate pier, it shows that the bottom of the intermediate pier is the most vulnerable region in the whole bridge, and the damage range of

### Table 6. Probabilistic seismic demand model of bridge component

| Seismic demand parameter | Regression equation | $R^2$ | $\theta_{EDP/im}$ |
|--------------------------|---------------------|-------|-------------------|
| $\Delta_{1.2}$           | $\ln(\Delta_{1.2}) = 0.8953 \ln(\text{PGA}) - 1.125$ | 0.574 | 0.688 |
| $\phi_{1.2}$             | $\ln(\phi_{1.2}) = 1.010 \ln(\text{PGA}) - 8.136$ | 0.331 | 1.104 |
| $\phi_{2.4}$             | $\ln(\phi_{2.4}) = 0.785 \ln(\text{PGA}) - 8.981$ | 0.422 | 0.663 |
| $\phi_{2.7}$             | $\ln(\phi_{2.7}) = 0.377 \ln(\text{PGA}) - 9.756$ | 0.341 | 0.897 |
| $\Delta_{3.4}$           | $\ln(\Delta_{3.4}) = 0.759 \ln(\text{PGA}) - 9.019$ | 0.514 | 0.530 |
| $\phi_{3.7}$             | $\ln(\phi_{3.7}) = 0.375 \ln(\text{PGA}) - 9.762$ | 0.359 | 0.612 |
| $\Delta_{4.8}$           | $\ln(\Delta_{4.8}) = 0.883 \ln(\text{PGA}) - 1.114$ | 0.489 | 0.676 |
| $\phi_{4.8}$             | $\ln(\phi_{4.8}) = 0.971 \ln(\text{PGA}) - 8.311$ | 0.316 | 1.111 |

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Figure 7. Fragility nephogram of 1# pier.
the bottom section of the intermediate pier is significantly wider than that of the side pier bottom. Furthermore, the diffusion speed of seismic damage for intermediate pier bottom is rapid than that for the side pier bottom with the increasing intensity measure of ground motion. Therefore, it is important to take the fragility situations of top section of intermediate pier and the bottom sections of side and intermediate piers into account in the seismic fragility analysis for the long-span continuous rigid frame girder bridge with high-rise pier.

4.4.2. Comparative analysis of component fragility

As shown in Section 4.4.1, the critical sections in the seismic fragility analysis are included in the bottom sections of side piers and the top and bottom sections of the intermediate piers. Moreover, the bearings are also seismically vulnerable (Lin et al., 2010). Therefore, the above-mentioned critical sections and bearings are taken into account while performing the seismic fragility analysis of bridge structural components.

In light of the PSDA in Section 4.3, the seismic fragility curves of all critical components are achieved by Equation (6). The seismic fragility curves of all components with minor and severe damage states are shown in Figure 9, and the ground motion intensities PGA are ranged from 0 g to 1.2 g in this figure. Even though all fragility curves of all seismic critical sections shown in Figure 9 are in similar shapes, different damage states correspond to different damage probabilities. From the fragility curves for all damage states, it can be inferred that the bearing on the top of both side piers are the most vulnerable components and the top sections of intermediate piers are least prone to be damaged under the action of ground motion. While comparing the specific situation of fragility for each component, the medians (Deepu, Prajapat & Ray-Chaudhuri, 2014) of the ground motion intensity measure with log-normal distribution is adopted to describe the component fragility, which means that the ground motion intensity corresponding 50% exceedance probability is taken as the index to evaluate the fragility of different components with different damage states. Thus, smaller the PGA median of the component, more fragile the component is.
are shown in Table 7, and the standard deviations of structural demand model are also listed in this table.

For more better analyzing the fragility of each component, the histogram of the PGA medians corresponding to the seismic damage probabilities for all the four damage states is shown in Figure 10. Moreover, the histogram of the components with 2.5 g or more PGA are not included in this figure for the reason that these components are invulnerable. The PGA medians are ranged from 0.26 g to 0.87 g for all critical components within the minor damage states. Also the bearings on the top of 4# and 1# piers are most likely to be damaged, followed by bottom sections of 2#, 3#, 1# and 4# piers, top sections of 2# and 3# piers. In addition, the fragility curves of both bottom sections for the two intermediate piers are almost identical.

The PGA medians of bearings on the top of 1# and 4# piers with moderate damage state are 0.57g and 0.51g, respectively, and these two bearings are the most vulnerable components for the moderate damage state. As far as the bottom sections of piers is concerned, the bottom sections of 2# and 3# are the fragile components, followed by the bottom sections of 1# and 4# sider piers. It means that the bottom sections of intermediate piers are more responsive to the moderate damage.

For the severe and collapse damage states, the seismic damage probabilities of the top and bottom sections for the 2# and 3# pier are small, and the bearings on the top of side piers are still the most vulnerable components. Furthermore, the seismic damage probabilities of all critical components are sequenced as the bearings on the top of 4# and 1# side piers, bottom sections of 1#, 4#, 2# and 3# piers, and the top section of 2# and 3# piers.
The discussed bridge structure in this paper is a typical irregular bridge with larger height differences between different piers, and the section property, reinforcement and concrete grade of each pier are different from each other. Moreover, the maximal axial forces carried by different critical sections are also different. All these specific configurational and mechanical characteristics of irregular bridge structure give rise to their different seismic responses. On the other hand, damage indices achieved from the moment-curvature analysis corresponding to damage states of all piers are significantly different, as shown in Table 3. Therefore, the fragility situations of the pier bottom sections with severe and collapse damage states are different from that of the pier bottom piers with minor and moderate damage states.

Comparing with the seismic fragility of regular bridge structure, the seismic fragility of irregular long span bridge structure with high-rise pier is more complex. The structural irregularity and high-order mode shapes resulted in the significant discrimination of seismic fragility between different piers, and the distributions of seismic fragility for different components with different damage states are different. Thus, the different seismic fragility distribution for different component is specified for the irregular bridge structure (Ghosh et al., 2014; Rokneddin et al., 2014; Siqueira et al., 2014).

4.4.3. Seismic fragility analysis of bridge structural system

In accordance with the seismic fragility analysis of each component discussed in Section 4.4.2, the fragility curve of irregular bridge structural system is derived by Equation (7). The bridge structural systemic fragility curves for minor and collapse damage states are shown in Figure 11, and the characteristic parameters of seismic fragility curves for bridge structural system are shown in Table 7.

From the comparison of Figures 9 and 11, it can be inferred that there is an obvious discrimination between the component and system fragilities corresponding to different damage states. As shown in Table 8, the seismic damage probability of bridge structural system is larger than that of all the structural components. Therefore, it is difficult to present the seismic performance of the irregular long span bridge with high-rise pier only depending on the seismic fragility analysis of the bridge structural components.

The fragility curve of bridge structural system can be obtained by the first-order reliability method, but the bandwidth between the upper and lower boundaries of the obtained fragility curves (Au, 2014a, 2014b) is relatively large. The bandwidth increases firstly and then decreases with the increasing PGA. The relative discrepancies of PGA medians corresponding to the upper and lower boundaries for all the four damage states are 41%, 65%, 35% and 35%, respectively.
5. Conclusion

In light of the PSDA, bridge structural component and system seismic fragility analysis for the typical irregular continuous rigid frame girder bridge in montane western China is discussed in this paper, and the main conclusion is listed as follows:

The fragility nephogram can present intuitively the variable characteristics of the seismic damage probability with various PGA for each cross-section of the whole pier, and can also reveal the damage diffusion and probability for the whole pier. As far as the irregular bridge structure is concerned, there are significant differences between seismic fragilities for different structural components with different damage states; its most vulnerable component is the bearings on the top of sider piers; and the bottom section of the 2# intermediate pier is most likely to be damaged slightly and moderately, and the bottom section of the 1# side pier are more vulnerable to be damaged severely and might be collapsed. Under the action of the ground motion with PGA less than 0.6 g, the severe damage and collapse states can hardly occur. Thus, the seismic performance of long-span irregular continuous rigid frame bridge with high-rise pier is favorable. Furthermore, the system fragility for long-span bridge with high-rise pier is significantly larger than the component fragility, and it is difficult to evaluate its systemic seismic performance just relying on the seismic fragility analysis of the bridge structural components.

Table 8. Fragility curve parameters of bridge system

| Damage state | Parameters | System |  |
|--------------|------------|--------|---|
|              |            | Upper boundary | lower boundary |
| Minor        | Median     | 0.207   | 0.291  |
|              | Standard deviation | 0.548 | 0.768  |
| Moderate     | Median     | 0.331   | 0.547  |
|              | Standard deviation | 0.548 | 0.768  |
| Severe       | Median     | 0.636   | 0.859  |
|              | Standard deviation | 0.524 | 0.880  |
| Collapse     | Median     | 0.822   | 1.112  |
|              | Standard deviation | 0.524 | 0.880  |

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Author details

Fan Chen1,2 E-mail: 22564658358@qq.com
Xiaoyu Gu1 E-mail: 838319102@qq.com
Deshan Shan1 E-mail: sshan@swjtu.edu.cn
Jun Dong1 E-mail: dj07swjtu@163.com
Qiao Li1 E-mail: 37321@vip.163.com
1 Bridge Engineering Department, Civil Engineering School, Southwest Jiaotong University, Chengdu, Sichuan 610031, China.
2 Power China Road Bridge Group Limited Company, Beijing 100048, China.

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