

Article

Seismic Damage Risk Assessment of Reinforced Concrete Bridges Considering Structural Parameter Uncertainties

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Abstract

To accurately assess the seismic risk of bridges, this study systematically conducted probabilistic seismic hazard–fragility–risk assessments using a reinforced concrete continuous girder bridge as a case study. First, the CPSHA method from China’s fifth-generation seismic zoning framework was employed to calculate the Peak Ground Acceleration (PGA) with 2%, 10%, and 63% exceedance probabilities over 50 years as 171.16 gal, 98.10 gal, and 28.61 gal, respectively, classifying the site as being with 0.10 g zone (basic intensity VII). Second, by innovatively integrating the Response Surface Method with Monte Carlo simulation, the study efficiently quantified the coupled effects of structural parameter and ground motion uncertainties, a finite element model was established based on OpenSees, and the seismic fragility curves were plotted. Finally, the risk probability of seismic damage was calculated based on the seismic hazard curve method. The results demonstrate that the study area encompasses 46 potential seismic sources according to China’s fifth-generation zoning. The seismic fragility curves clearly show that side piers and their bearings are generally more susceptible to damage than middle piers and their bearings. Over 50 years, the pier risk probabilities for the intact, slight, moderate, severe damage, and collapse are 68.90%, 6.22%, 15.75%, 7.86%, and 1.27%, while the corresponding probabilities of bearing are 3.54%, 44.11%, 25.64%, 7.74%, and 18.97%, indicating significantly higher bearing risks at the moderate damage and collapse levels. The method proposed in this study is applicable to various types of bridges and has high promotion and application value.



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Keywords: bridge; seismic hazard assessment; OpenSees; Response Surface Method; seismic fragility assessment; structural uncertainty; probability risk assessment

1. Introduction

China is located at the intersection of the Circum-Pacific Seismic Zone and the Eurasian Seismic Zone, making it one of the countries most severely affected by earthquake disasters globally [1]. The 1976 Tangshan earthquake nearly destroyed all highway and railway bridges in Tangshan City. The 2008 Wenchuan earthquake caused severe damage or collapse to 25 highway bridges in Sichuan Province. Notably, the G213 Baihua Bridge, located approximately 1.0 km from the Longmenshan active fault, experienced the collapse of its fifth span, with 90% of the remaining piers exhibiting crushing failures [2]. With the arrival of active cycles, future earthquakes may cause even more severe bridge damage. Therefore, systematically analyzing the failure mechanisms and damage degrees of bridges under different ground motions is crucial for improving seismic capacity of bridges and ability of regional disaster mitigation.

The degree of bridge seismic damage depends on two aspects. First, the intensity and probability of seismics at the bridge site (study site), which belongs to seismic hazard assessment. Factors such as seismic zones, distribution of potential seismic source zones, historical seismic records and site conditions may affect seismic hazards. Second, the seismic performance of the bridge itself, characterized by the probability of exceeding various damage levels under a given ground motion, which belongs to seismic fragility assessment, may affect seismic hazards. Factors such as bridge structural form, construction quality and protective measures influence seismic fragility. The probability of bridge seismic risk, which represents the probability of a specific bridge exceeding various damage levels over a certain period [3], is derived from the integration of seismic hazard and fragility assessments by analyzing their coupled characteristics.

Numerous scholars have made significant contributions to the field of structural seismic risk assessment. In the field of seismic hazard assessment, Cornell proposed the Probabilistic Seismic Hazard Analysis (PSHA) [4]. China's current seismic hazard assessment methodology (CPSHA) was developed based on PSHA [5]. The third to fifth generations of China's seismic ground motion zonation schemes were formulated by CPSHA [6]. For seismic fragility assessment, Vamvatsikos and Cornell introduced the Incremental Dynamic Analysis (IDA), which has since become the most widely adopted method [7]. Wang et al. proposed a fragility assessment approach that incorporated multi-dimensional performance limit states and pioneered fragility curves for multi-span continuous concrete girder bridges, explicitly accounting for pier ductility and lateral deformation [8]. Li et al. established the probability density evolution theory and applied it to nonlinear dynamic response analysis, thereby quantifying the variability in physical parameters of engineering structures [9]. In seismic risk assessment, Yin et al. and Che et al. employed the Monte Carlo method and seismic hazard curve method to assess seismic risks for a highway embankment and proposed a risk management method based on risk acceptability theory [10,11]. Abeyisiriwardena et al. developed a seismic risk analysis method for RC frame structures considering aftershock effects [12]. Their work involved convolving mainshock–aftershock fragility functions with hazard functions, extending the traditional risk integral formula to a bivariate domain, and deriving demand risk functions incorporate aftershock impacts.

Structural uncertainties are inherent in the design, construction and operation of bridges. A critical technical challenge lies in characterizing the coupled effects of structural parameter and ground motion uncertainties on bridge seismic performance. Although Chinese seismic design philosophy for bridges adheres to the principle of “no damage under minor seismic events, repairable damage under moderate seismic events, and no collapse under major seismic events” [13], clear definitions are lacking for the corresponding seismic intensity indicators (“minor”, “moderate” “major” and damage states (“no damage”, “repairable” “no collapse”); this ambiguity significantly limits the practical application of the philosophy. To address these issues, primary contributions of this study are twofold: (1) to develop an integrated probabilistic framework for bridge seismic risk assessment; (2) to introduce a novel, computationally efficient methodology that integrates the Response Surface Method with Monte Carlo simulation to effectively quantify the coupled effects of structural and ground motion uncertainties. This integrated approach thereby provides a practical and theoretically sound tool for performance-based seismic design and decision-making. The study flowchart is shown in Figure 1.

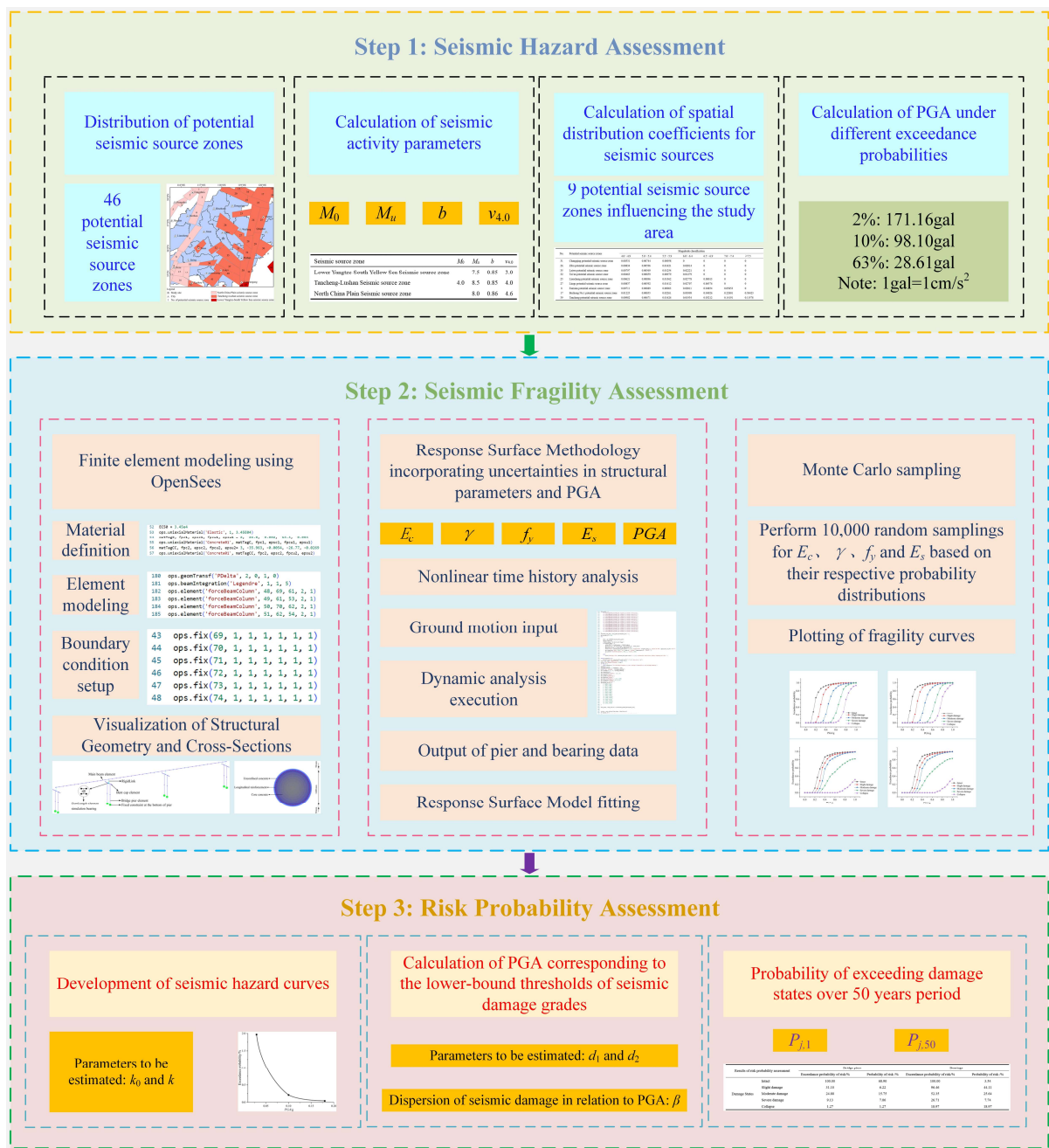


Figure 1. Study flowchart.

2. Finite Element Model Development of the Study Object

The bridge studied is located in Boshan District, Zibo City, Shandong Province, China, with a span arrangement of 3×30 m. The main beam was constructed with C50 concrete (The 28-day compressive strength of the concrete is 50 MPa), while the bridge piers and bent caps were constructed using C40 concrete (the 28-day compressive strength of the concrete is 40 MPa). HRB400 steel (the tensile strength of the reinforcing steel is 400 MPa.) was adopted for reinforcement, and pot rubber bearings were employed as supports. In the finite element modeling using OpenSees (Open System for Earthquake Engineering Simulation), the main beam and bent caps were modeled with elastic beam-column elements, the bearings were simulated using zero-length elements, the bridge piers were represented by force-based beam-column elements with fixed bases at the foundation. Given that the bridge site is characterized by stiff soil (Site Class II), where soil–structure interaction effects

are known to be minimal, this fixed-base assumption is considered appropriate, allowing the analysis to focus on the uncertainty and performance of the superstructure components. Concrete in the bridge piers was modeled with the Concrete01 material, and reinforcing steel was characterized by the Steel02 material [14–16]. The constitutive relationships of the Concrete01 and Steel02 materials are shown in Figure 2. Figure 2 is cited from the official OpenSees website at: <https://opensees.berkeley.edu/> (accessed on 27 April 2024)

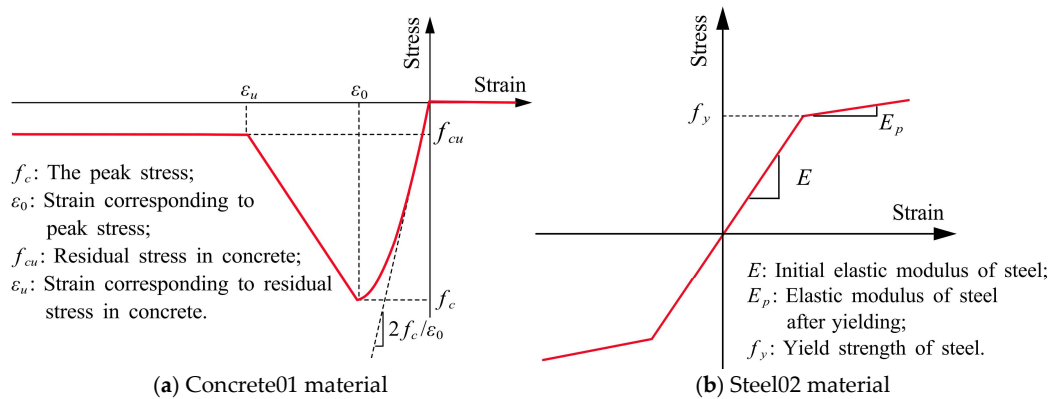


Figure 2. Constitutive relationships of material.

The piers were discretized into fiber elements, which were partitioned using Legendre integration. Each element was divided into five integration points along its longitudinal axis, with each integration point having the same cross-section. The five Legendre integration points along the longitudinal direction of the pier and the division of fibers at their cross-sections are shown in Figure 3.

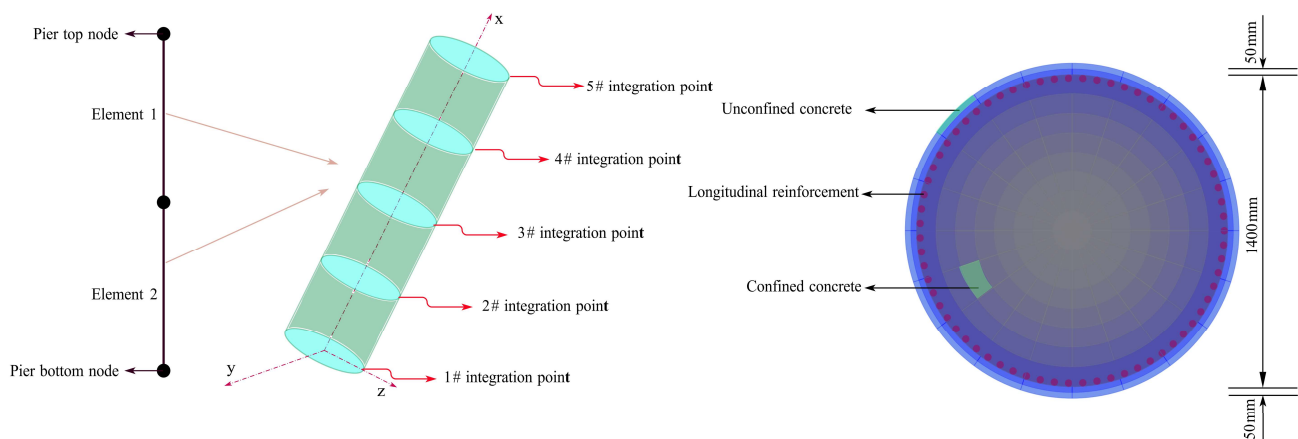


Figure 3. Legendre integration points and the fiber division of their cross-sections.

The Midas and OpenSees finite element models are shown in Figures 4 and 5, respectively.

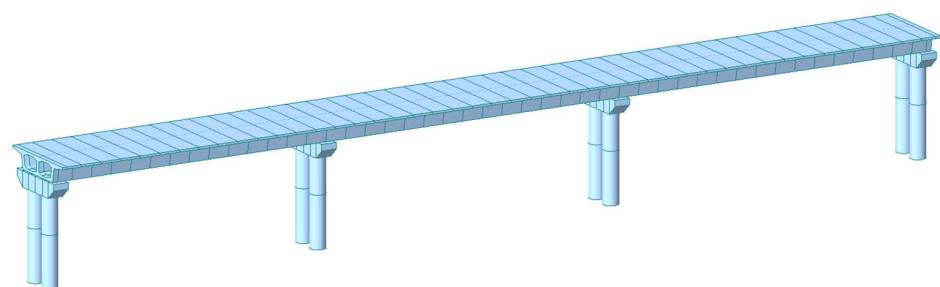


Figure 4. Midas finite element model of bridge.

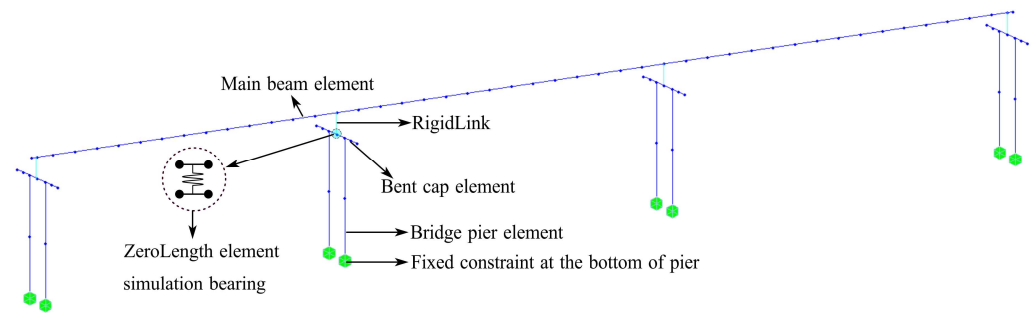


Figure 5. OpenSees finite element model of bridge.

3. Classification of Bridge Seismic Damage Levels and Selection of Damage Parameters

3.1. Classification of Bridge Seismic Damage Levels

The Standard for Classification Seismic Damage to Buildings and Structures (GB/T 24335-2009) [17] classify the seismic damage into five levels: intact, slight damage, moderate damage, severe damage and collapse [18]. This study adopts this classification for bridge and provides qualitative descriptions for each damage level, as summarized in Table 1.

Table 1. Qualitative descriptions of bridge seismic damage levels.

Seismic Damage Levels	Damage Description
Intact	The bridge components remain undamaged, and normal operation is maintained.
Slight damage	Surface-level concrete spalling or cracks are observed, normal operation can resume after minor repairs.
Moderate damage	Significant longitudinal or lateral displacement occurs, normal operation is achievable following emergency repairs.
Severe damage	Through cracks develop in the structure, partial operation can be restored with temporary reinforcement.
Collapse	Girder collapse or pier failure occurs, resulting in complete functional loss and operational shutdown.

3.2. Seismic Damage Parameters for Bridges

Bridge piers and bearings are the primary components susceptible to seismic damage. Accordingly, separate damage parameters were selected for each to assess their performance independently.

3.2.1. Seismic Damage Parameters of Bridge Piers

Due to their nonlinear response characteristics, which are mainly governed by flexural damage [19], the displacement ductility ratio was selected as the seismic damage parameter for bridge piers in this study [20]. The calculation method is shown in Equation (1).

$$\mu_d = \frac{\Delta_d}{\Delta_{cy1}} \quad (1)$$

where μ_d is the displacement ductility ratio of the pier, Δ_d is the relative displacement of the pier top, and Δ_{cy1} is the relative displacement at the pier top when the outermost longitudinal reinforcement steel first yields. The relationship between the bridge seismic levels and μ_d is summarized in Table 2.

Table 2. Relationship between bridge seismic damage levels and μ_d .

Seismic Damage Levels	Intact	Slight Damage	Moderate Damage	Severe Damage	Collapse
μ_d	$0 \leq \mu_d < \mu_{cy1}$	$\mu_{cy1} \leq \mu_d < \mu_{cy}$	$\mu_{cy} \leq \mu_d < \mu_{c4}$	$\mu_{c4} \leq \mu_d < \mu_{cmax}$	$\mu_d \geq \mu_{cmax}$

Where μ_{cy1} is the displacement ductility ratio at the first yield of the outermost longitudinal reinforcement in the bridge pier. μ_{cy} is the equivalent displacement ductility ratio, defined as the ratio of the ultimate displacement to the yield displacement of the bridge pier, where a higher value indicates superior ductility. μ_{c4} represent the displacement ductility ratio when the compressive strain of the cover concrete reaches 0.004, which corresponds to the stage of complete spalling of cover concrete and initiation of crack in the core concrete region. Finally, μ_{cmax} is the maximum ductility ratio, attained when the core concrete crushes, the longitudinal reinforcing steel yields, and the stirrups fracture. Since μ_{cy1} only accounts for the initial yield and ignores the subsequent nonlinear deformations, it is defined as $\mu_{cy1} = 1$. The calculation methods for μ_{cy} , μ_{c4} and μ_{cmax} are shown in Equation (2)–(4) [21].

$$\mu_{cy} = \frac{\Delta_{cy}}{\Delta_{cy1}} = \frac{\varphi_{y2}}{\varphi_{y1}} \quad (2)$$

$$\mu_{c4} = \frac{\Delta_{cy} + \left(\frac{0.08 \cdot L + 0.022 \cdot f_y \cdot d_s}{2} \right) \cdot (\varphi_{c4} - \varphi_{y2}) \cdot \left(L - \frac{0.08 \cdot L + 0.022 \cdot f_y \cdot d_s}{2} \right)}{\Delta_{cy1}} \quad (3)$$

$$\mu_{cmax} = \mu_{c4} + 3 \quad (4)$$

where φ_{y1} denotes the curvature at the first yield of the outermost longitudinal reinforcing steel in the bridge pier. φ_{y2} is the equivalent yield curvature, and φ_{c4} is the curvature when the compressive strain of the cover concrete reaches 0.004. L denotes the height of the bridge pier. f_y is the yield strength of the reinforcement steel. d_s is the diameter of the steel. This study adopted the following parameter values: $\varphi_{y1} = 8.164 \times 10^{-4}$ rad/m, $\varphi_{y2} = 9.233 \times 10^{-4}$ rad/m, $\varphi_{c4} = 3.421 \times 10^{-3}$ rad/m, $L = 10$ m, $f_y = 400$ MPa and $d_s = 0.03$ m. Based on these calculations, Table 2 was converted to Table 3.

Table 3. Relationship between bridge seismic damage levels and μ_d .

Seismic Damage Levels	Intact	Slight Damage	Moderate Damage	Severe Damage	Collapse
μ_d	$0 \leq \mu_d < 1$	$1 \leq \mu_d < 1.132$	$1.132 \leq \mu_d < 1.893$	$1.893 \leq \mu_d < 4.893$	$\mu_d \geq 4.893$

3.2.2. Seismic Damage Parameters of Bearings

Due to the relatively low horizontal displacement threshold of pot rubber bearings, this study selected the shear deformation d as the seismic damage parameter for bearings, following the research by Zhong et al. [22]. The relationship between bridge seismic damage level and d is shown in Table 4.

Table 4. Relationship between bridge seismic damage levels and d .

Seismic Damage Levels	Intact	Slight Damage	Moderate Damage	Severe Damage	Collapse
d/mm	$0 \leq d < 25$	$25 \leq d < 50$	$50 \leq d < 75$	$75 \leq d < 90$	$d \geq 90$

4. Probabilistic Seismic Hazard Assessment of the Study Site

4.1. Overview of the CPSHA Method

(1) The methodology adopted the principle of three-level potential seismic source zone division and constructed the corresponding seismic activity model. This model comprises seismic statistical zones (seismic zones) and potential seismic source zones of background seismic activity (background sources), and potential seismic source zones of tectonic structures (tectonic sources), which are spatially superimposed [23]. The seismic statistical zone serves as the bottom layer and the statistical unit for seismic activity parameters. The background source, as the middle layer, reflects the characteristics of background seismic activity within the seismic zones and differences in seismic tectonic models. The tectonic source, as the top layer, reflects the characteristics of moderate-to-strong seismic activities associated with local tectonic structures. The representation of spatial inhomogeneity in seismic activity varies across different magnitude ranges: for small-to-moderate magnitudes, it is jointly represented by tectonic sources and background sources, whereas for moderate-to-strong magnitudes, it is represented solely by tectonic sources.

(2) The seismic magnitudes in seismic zone follow the Gutenberg–Richter relationship, as shown in Equation (5) [24].

$$\lg N = a - b \cdot M (M_0 \leq M \leq M_u) \quad (5)$$

where N is the number of earthquakes with magnitudes greater than or equal to M , M is the seismic magnitude; M_0 is the threshold magnitude, defined as the minimum magnitude that causing damage; M_u is the magnitude upper limit, which is the maximum possible seismic magnitude; a and b are statistical parameters, where a reflects the level of seismic activity over a specific period, and b reflects the proportional relationship between large and small earthquakes. The probability density function of the magnitude distribution in seismic zones can be derived from Equation (5), as shown in Equation (6) [25].

$$f(M) = \frac{\beta \cdot e^{-\beta \cdot (M-M_0)}}{1 - e^{-\beta \cdot (M_u-M_0)}} (M_0 \leq M \leq M_u) \quad (6)$$

where $\beta = b \cdot \ln 10$.

(3) The seismic activity in a seismic zone over a period follows a Poisson distribution, implying that the probabilistic characteristics of seismic activity in the zone can be determined by a parameter v . The two-state Poisson model assumes that seismic activity in the seismic zone has “active” and “inactive” states, making it more consistent with the actual occurrence pattern of earthquakes. The probability P_K of K earthquakes occurring in the seismic zone over the next T years as shown in Equation (7) [26].

$$P_K = \frac{(v \cdot T)^K}{K!} \cdot e^{-v \cdot T} \quad (7)$$

(4) Seismic activity within a seismic zone is non-uniformly distributed, whereas it is uniformly distributed within a potential seismic source zone. The probability that an earthquake in magnitude bin m_j occurs in the i -th potential seismic source zone is denoted as f_{i,m_j} . Given that the area of the i -th potential seismic source zone is A_i , the probability $P(m_j|i)$ of an earthquake in magnitude bin m_j occurring at any point within the i -th potential seismic source zone as shown in Equation (8) [27].

$$P(m_j|i) = f_{i,m_j} / A_i \quad (8)$$

(5) Equation (9) was derived based on Equation (5) to Equation (8).

$$P(m_j) = \frac{2}{\beta} \cdot f_m(m_j) \cdot \text{sh}(\beta \cdot \Delta m / 2) = \frac{2 \cdot e^{-\beta \cdot (m_j - M_0)}}{1 - e^{-\beta \cdot (M_u - M_0)}} \cdot \text{sh}(\beta \cdot \Delta m / 2) \quad (9)$$

where $P(m_j)$ is the probability that an earthquake within the seismic zone lies within the magnitude bin $m_j \pm 0.5\Delta m$, and $\text{sh}(\beta \cdot \Delta m / 2)$ denoted the hyperbolic function with $\beta \cdot \Delta m / 2$ as its argument. Assuming that an earthquake of magnitude m_j occurs randomly in the k -th seismic zone, the probability of generating a $\text{PGA} \geq y_q$ at a given site is expressed by Equation (10).

$$P(\text{PGA} \geq y_q)_k = \sum_{j=1}^{N_m} \sum_{i=1}^{N_{ks}} \int \int P(\text{PGA} \geq y_q | m_j, (x, y)_{ki}) \cdot \frac{2 \cdot e^{-\beta_k \cdot (m_j - M_0)}}{1 - e^{-\beta_k \cdot (M_u - M_0)}} \cdot \frac{f_{ki, m_j}}{A_{ki}} \cdot \text{sh}\left(-\frac{\beta_k}{2} \cdot \Delta m\right) dx dy \quad (10)$$

where N_m is the total number of magnitude bins, N_{ks} is the number of potential seismic source zones in the k -th seismic zone, y_q is the specified PGA value, and $dx dy$ is the cell area of the gridded potential seismic source zones. By integrating the effects of N_z seismic zones in the area using the total probability formula, the seismic hazard probability can be calculated using Equation (11).

$$P(\text{PGA} \geq y_q) = 1 - \exp \left[- \sum_{j=1}^{N_m} \sum_{i=1}^{N_{ks}} \sum_{k=1}^{N_z} \int \int P(\text{PGA} \geq y_q | m_j, (x, y)_{ki}) \cdot \frac{2 \cdot e^{-\beta_k \cdot (m_j - M_0)}}{1 - e^{-\beta_k \cdot (M_u - M_0)}} \cdot \frac{f_{ki, m_j}}{A_{ki}} \cdot \text{sh}\left(-\frac{\beta_k}{2} \cdot \Delta m\right) dx dy \right] \quad (11)$$

(6) The PGA attenuation relationship shown in Equation (12) is adopted [28].

$$\lg \text{PGA} = A + B \cdot M + C \cdot \lg(R + D \cdot e^{E \cdot M}) \quad (12)$$

where M is the surface wave magnitude, R is the epicentral distance, and A, B, C, D and E are coefficients, with values shown in Table 5.

Table 5. Coefficients of the PGA attenuation relationship.

Direction	A		B		C	D	E
	$M \geq 6.5$	$M < 6.5$	$M \geq 6.5$	$M < 6.5$			
Long axis	−0.363	1.437	0.791	0.513	−2.103	2.088	0.399
Short axis	−1.147	0.712	0.788	0.502	−1.825	0.944	0.447

Where the Long axis refers to the principal direction in the spatial distribution of ground motion parameters, along which these parameters vary gradually and exhibit a wider zone of influence. That is, ground motion parameter attenuation along the Long Axis occurs at relatively low gradient with increasing distance from the seismic source. In contrast, the Short axis represents the principal direction in which ground motion parameters change more rapidly and have a narrow influence range. Along the Short axis, the values decay more sharply with increasing distance from the seismic source.

(7) According to the conversion coefficient method of the fifth-generation seismic ground motion parameter zoning scheme, Equation (13) is adopted to convert bedrock PGA to site class II PGA [29].

$$\text{PGA}_{II} = F_a \cdot \text{PGA}_I \quad (13)$$

where PGA_J is the bedrock PGA, PGA_{II} is the PGA for class II sites, F_a is the conversion coefficient, which is calculated using Equation (14) [30].

$$F_a = \begin{cases} 1.25 & PGA_J \leq 62.5 \text{ gal} \\ 1.25 - (PGA_J - 62.5)/1250 & 62.5 \text{ gal} < PGA_J \leq 375 \text{ gal} \\ 1 & PGA_J > 375 \text{ gal} \end{cases} \quad (14)$$

where $1 \text{ gal} = 1 \text{ cm/s}^2$.

4.2. Case Study

Based on the fifth-generation seismic ground motion parameter zoning scheme, the study area (extended 200 km from the study site) was influenced by the Tancheng–Lushan seismic source zone, the Lower Yangtze–South Yellow Sea seismic source zone, and the North China Plain seismic source zone. These three source zones encompass 28, 4 and 14 potential seismic source zones, respectively, totaling 46 potential seismic source zones, as shown in Figure 6.

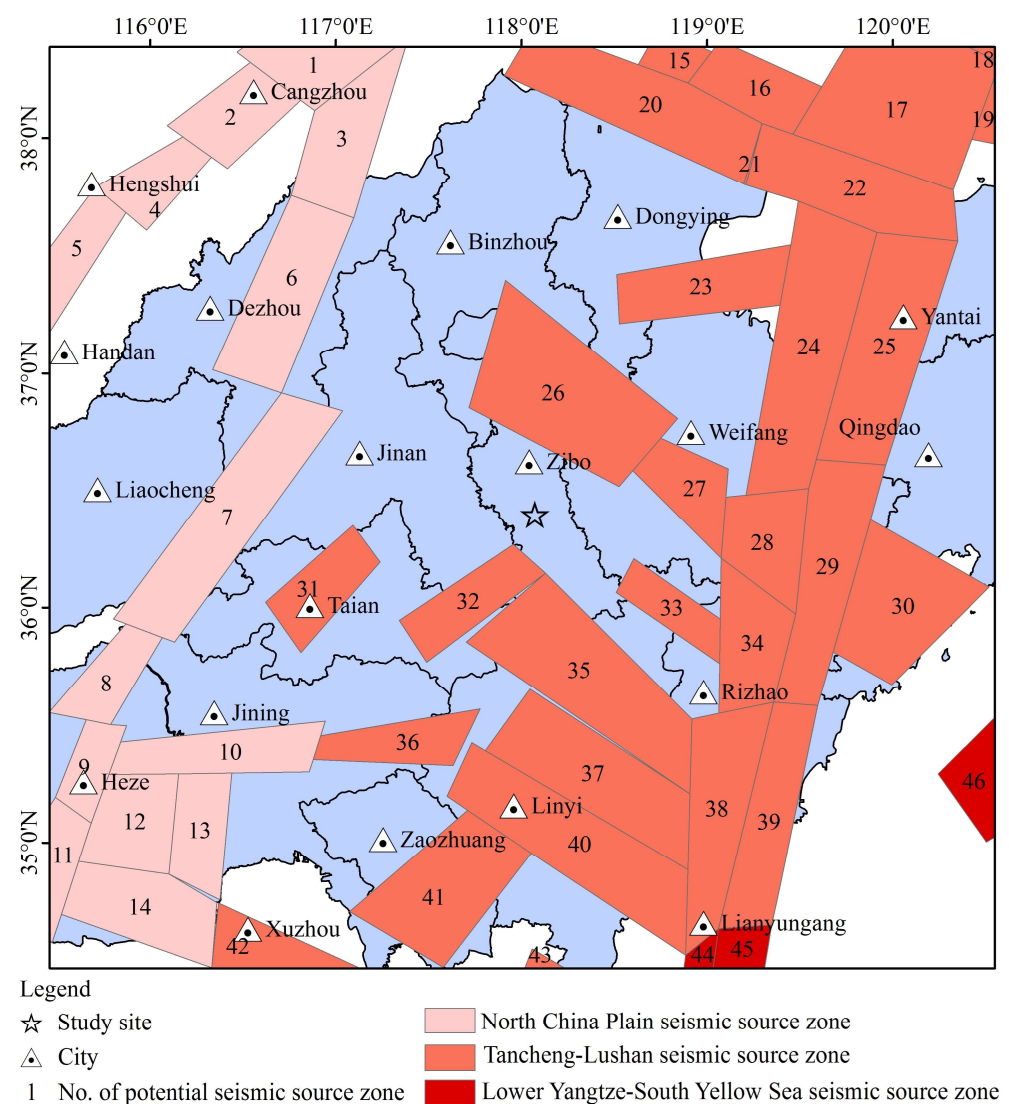


Figure 6. Distribution of seismic zones and potential seismic source areas.

According to the fifth-generation seismic ground motion parameter zoning scheme, the values of M_0 , M_u , b and the annual average occurrence rate $v_{4.0}$ for earthquakes above magnitude 4.0 in each seismic zone are shown in Table 6.

Table 6. Seismic activity parameters.

Seismic Source Zone	M_0	M_u	b	$v_{4.0}$
Lower Yangtze–South Yellow Sea seismic source zone	4.0	7.5	0.85	3.0
Tancheng–Lushan seismic source zone		8.5	0.85	4.0
North China Plain seismic source zone		8.0	0.86	4.6

There are 9 potential seismic source zones affecting the seismic hazard of the study area, and their seismic spatial distribution coefficients are shown in Table 7.

Table 7. Seismic spatial distribution coefficients.

No.	Potential Source Zones	Magnitude Classification						
		[4.0, 4.9]	[5.0, 5.4]	[5.5, 5.9]	[6.0, 6.4]	[6.5, 6.9]	[7.0, 7.4]	≥ 7.5
31	Changqing potential seismic source zone	0.00331	0.00744	0.00958	0	0	0	0
26	Zibo potential seismic source zone	0.00834	0.00596	0.01421	0.02014	0	0	0
35	Laiwu potential seismic source zone	0.00797	0.00569	0.01254	0.02221	0	0	0
32	Tai'an potential seismic source zone	0.00463	0.00859	0.00979	0.01279	0	0	0
25	Liaocheng potential seismic source zone	0.00621	0.00896	0.01362	0.02778	0.08013	0	0
27	Linqu potential seismic source zone	0.00837	0.00592	0.01412	0.02707	0.06576	0	0
8	Fanxian potential seismic source zone	0.00711	0.00889	0.00963	0.02011	0.04854	0.05853	0
17	Bozhong No.1 potential seismic source zone	0.01225	0.00855	0.02261	0.03999	0.04926	0.22801	0.58423
39	Tancheng potential seismic source zone	0.00982	0.00671	0.01426	0.01934	0.03212	0.14191	0.11578

The fifth-generation seismic ground motion parameter zoning scheme specifies the PGA with a 10% probability of exceedance over 50 years as the design basis earthquake for general structures. Using the seismic safety assessment software SECR2019, the PGA values for the study site corresponding to 2%, 10% and 63% probabilities of exceedance over 50 years are calculated as 171.16 gal, 98.10 gal and 28.61 gal, respectively. These results place the study site within the 0.10 g zone, which corresponds to basic seismic intensity zone VII.

5. Seismic Fragility Assessment of the Study Object Based on the Monte Carlo Method and Response Surface Method

5.1. Uncertainty Analysis of Structural-Seismic Motion Samples

5.1.1. Probability Distribution and Normalization of Random Variables

In seismic analysis, uncertainties in bridge structures primarily arise from variations in material properties, geometric characteristics, and construction practices. Furthermore, ground motion intensity significantly influences structural responses [31]. Therefore, this study selected five random variables for fragility assessment: concrete elastic modulus E_c , damping ratio γ , steel yield strength f_y , steel elastic modulus E_s , and PGA. Among them, E_c , f_y and E_s follow log-normal distributions [32–34], whereas γ follows a normal distribution [35].

The Central Composite Design (CCD) method is a widely used approach for constructing response surface models due to its simplicity, efficiency, and reasonable number of required trials [36]. The random variables were normalized using the CCD method as shown in Equation (15). The probability distributions and corresponding statistical parameters of these random variables are summarized in Table 8.

$$x_i = \frac{\xi_i - (\xi_{i,\max} + \xi_{i,\min})/2}{(\xi_{i,\max} - \xi_{i,\min})/2} \quad (15)$$

where ξ_i , $\xi_{i,\min}$ and $\xi_{i,\max}$ denote the value, minimum value and maximum value of the i -th random variable, respectively; x_i is the normalized value of the i -th random variable, where $x_i = 1, 0$ and -1 represent ξ_i taking the maximum value, mean value and minimum value, respectively. $i = 1, 2, 3, 4$ and 5 represents the variables E_c , γ , f_y , E_s and PGA , respectively.

Table 8. Probability distributions and statistical parameters of random variables.

Random Variables	Probability Distribution	Variables Before and After Normalization	Minimum Value	Mean Value	Maximum Value	Standard Deviation	Coefficient of Variation
E_c/MPa	Log-normal distribution	ξ_1	2.99×10^4	3.25×10^4	3.51×10^4	2600	0.080
		x_1	-1	0	1		
γ	Normal distribution	ξ_2	0.035	0.05	0.065	0.015	0.300
		x_2	-1	0	1		
f_y/MPa	Log-normal distribution	ξ_3	372	400	428	28	0.070
		x_3	-1	0	1		
E_s/MPa	Log-normal distribution	ξ_4	1.94×10^4	2.00×10^4	2.06×10^4	6000	0.030
		x_4	-1	0	1		
PGA/g		ξ_5	0.2	0.6	1.0		
		x_5	-1	0	1		

5.1.2. Selection of Ground Motion Inputs

Due to the high lateral stiffness of the bridge, this study only considered longitudinal ground motion input. Fifteen measured ground motions were selected based on the target response spectrum, as listed in Table 9, with their acceleration response spectrum shown in Figure 7. These ground motions were carefully selected to be representative of the seismic hazard at the site, encompassing a range of magnitudes and source-to-site distances. The PGA of the measured ground motions was adjusted to 20 groups with intervals of 0.05 g , ranging from a minimum of 0 to a maximum of 1.0 g . The natural ground motions are amplitude-scaled to these values, resulting in a total of 300 ground motion records.

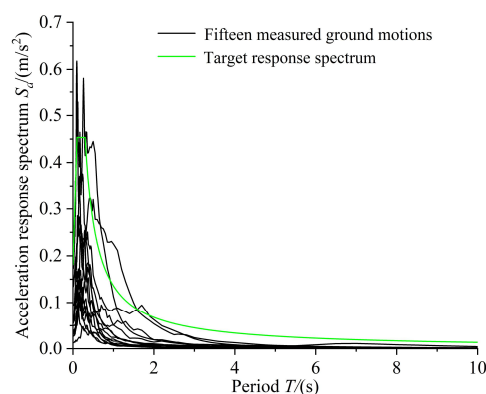


Figure 7. Acceleration response spectrum.

Table 9. Measured ground motion information.

No.	Earthquake	Year	Station Name	<i>M</i>	<i>R/km</i>	$v_{s30}/\text{m}\cdot\text{s}^{-1}$
1	Imperial Valley-04	1953	El Centro Array #9	5.50	15.11	213.44
2	Lytle Creek	1970	Castaic-Old Ridge Route	5.33	103.23	450.28
3	Lytle Creek	1970	Cedar Springs Pumphouse	5.33	21.33	477.22
4	Lytle Creek	1970	Lake Hughes #1	5.33	90.25	425.34
5	Lytle Creek	1970	Puddingstone Dam (Abutment)	5.33	29.49	421.44
6	Lytle Creek	1970	Santa Anita Dam	5.33	42.14	667.13
7	Lytle Creek	1970	Wrightwood-6074 Park Dr	5.33	10.70	486.00
8	San Fernando	1971	Cedar Springs Pumphouse	6.61	92.25	477.22
9	San Fernando"	1971	Colton-So Cal Edison	6.61	96.81	301.95
10	San Fernando	1971	Fort Tejon	6.61	59.52	394.18
11	San Fernando	1971	Gormon-Oso Pump Plant	6.61	43.95	308.35
12	San Fernando	1971	Tehachapi Pump	6.61	61.75	669.48
13	San Fernando	1971	Wrightwood-6074 Park Dr	6.61	61.64	486.00
14	Point Mugu	1973	Port Hueneme	5.65	15.48	248.98
15	Hollister-03	1974	Gilroy Array #1	5.14	9.99	1428.14

Note, v_{s30} is the average shear wave velocity within 30 m below ground.

5.2. Development of Response Surface Model

5.2.1. Introduction to Response Surface Model

The Response Surface Method is a statistical approach used to analyze the relationship between response variables and multiple input variables [37,38]. Owing to the nonlinear response of bridges under seismic action, a quadratic polynomial is typically selected as the response surface function [39,40], as expressed in Equation (16).

$$\hat{y} = a_0 + \sum_{i=1}^k a_i \cdot x_i + \sum_{i=1}^{k-1} \sum_{j>1}^k a_{ij} \cdot x_i \cdot x_j + \sum_{i=1}^k a_{ii} \cdot x_i^2 + \varepsilon \quad (16)$$

where k is the number of input variables, x_i and x_j represent the values of input variables, a_0 , a_i , a_{ii} and a_{ij} are undetermined coefficients estimated using the CCD method, and ε is the error term, which follows a normal distribution with a mean of zero.

For the finite element model of the bridge, conducting $N = 2^k + 2k + 1$ times of IDA yields a system of linear equations for the unknown coefficients a_0 , a_i , a_{ii} and a_{ij} . Using the least squares method to obtain the unbiased estimates of these undetermined coefficients, and validate the resulting response surface model using the coefficient of determination R^2 and the adjusted coefficient of determination R_a^2 . Following common practice in response surface modeling, at least one of R^2 and R_a^2 is required to exceed 0.9 [41], as defined in Equations (17) and (18).

$$R^2 = \frac{\alpha^T X^T Y - (E^T Y)^2 / N}{Y^T Y - (E^T Y)^2 / N} \quad (17)$$

$$R_a^2 = 1 - \frac{(1 - R^2)(N - 1)}{(N - k)} \quad (18)$$

where α denotes the vector of undetermined coefficients, X represents the design matrix of variable, Y is the true response vector, E is the unit vector.

5.2.2. IDA Analysis Based on the CCD Method

This study selected $k = 5$, so the CCD method was used to design $N = 43$ groups of IDA. Each group comprised 15 ground records, yielding a total of 645 analyses. The mean values and standard deviations of the seismic damage parameters for the side piers, middle piers, and their corresponding bearings in each group were recorded, as summarized in Table 10.

Table 10. Results of IDA.

No.	x_1	x_2	x_3	x_4	x_5	Bridge Piers				Bearings			
						μ_d of Side Piers		μ_d of Middle Piers		d of Side Pier Bearings/mm		d of Middle Pier Bearings/mm	
						Mean Value	Standard Deviation	Mean Value	Standard Deviation	Mean Value	Standard Deviation	Mean Value	Standard Deviation
1	-1	-1	-1	-1	-1	0.24	0.22	0.24	0.22	27.33	25.07	26.51	24.39
2	-1	-1	-1	-1	1	1.10	1.01	1.08	0.99	103.75	95.29	102.16	93.87
3	-1	-1	-1	1	-1	0.22	0.20	0.22	0.20	23.97	21.95	21.55	19.80
4	-1	-1	-1	1	1	1.08	0.99	1.07	0.98	99.46	91.30	98.45	90.41
5	-1	-1	1	-1	-1	0.22	0.22	0.22	0.22	24.05	22.06	21.30	19.60
6	-1	-1	1	-1	1	1.08	1.00	1.06	0.98	99.87	91.73	99.33	91.27
7	-1	-1	1	1	-1	0.20	0.18	0.20	0.19	19.30	17.68	18.10	16.63
8	-1	-1	1	1	1	1.05	0.97	1.04	0.95	97.57	89.56	96.36	88.49
9	-1	1	-1	-1	-1	0.20	0.19	0.20	0.19	24.33	22.29	20.94	19.24
10	-1	1	-1	-1	1	1.08	0.99	1.07	0.98	100.16	91.82	99.01	90.76
11	-1	1	-1	1	-1	0.20	0.18	0.20	0.18	19.74	18.07	18.45	16.90
12	-1	1	-1	1	1	1.06	0.97	1.03	0.95	97.89	89.66	96.43	88.35
13	-1	1	1	-1	-1	0.20	0.19	0.20	0.18	20.08	18.40	17.96	16.48
14	-1	1	1	-1	1	1.05	0.96	1.04	0.95	98.11	89.94	96.84	88.77
15	-1	1	1	1	-1	0.18	0.17	0.18	0.17	16.52	15.10	14.82	13.58
16	-1	1	1	1	1	1.02	0.94	0.99	0.91	94.96	86.98	94.63	86.70
17	1	-1	-1	-1	-1	0.22	0.20	0.21	0.20	23.81	21.84	20.84	19.18
18	1	-1	-1	-1	1	1.08	0.99	1.07	0.98	100.33	92.15	98.87	90.85
19	1	-1	-1	1	-1	0.20	0.18	0.20	0.18	20.56	18.83	17.73	16.29
20	1	-1	-1	1	1	1.05	0.96	1.03	0.95	98.41	90.34	96.94	89.03
21	1	-1	1	-1	-1	0.20	0.18	0.20	0.19	20.06	18.40	17.81	16.39
22	1	-1	1	-1	1	1.05	0.96	1.04	0.95	98.11	90.14	96.51	88.68
23	1	-1	1	1	-1	0.19	0.17	0.18	0.17	15.63	14.32	15.17	13.94
24	1	-1	1	1	1	1.00	0.92	1.00	0.92	95.46	87.63	94.52	86.80
25	1	1	-1	-1	-1	0.20	0.18	0.19	0.18	20.03	18.35	17.93	16.46
26	1	1	-1	-1	1	1.05	0.97	1.04	0.95	97.92	89.76	96.54	88.49
27	1	1	-1	1	-1	0.20	0.18	0.19	0.18	16.17	14.78	14.30	13.09
28	1	1	-1	1	1	1.02	0.94	1.00	0.91	95.61	87.58	93.92	86.05
29	1	1	1	-1	-1	0.18	0.17	0.18	0.17	15.85	14.52	14.95	13.72
30	1	1	1	-1	1	1.02	0.93	1.00	0.92	95.10	87.18	94.31	86.45
31	1	1	1	1	-1	0.15	0.13	0.16	0.15	11.96	10.93	11.57	10.60
32	1	1	1	1	1	0.99	0.91	0.98	0.90	93.54	85.68	92.03	84.32
33	1	0	0	0	0	0.65	0.59	0.64	0.59	56.09	51.46	54.89	50.38
34	-1	0	0	0	0	0.67	0.62	0.66	0.60	57.40	52.66	55.74	51.16
35	0	1	0	0	0	0.66	0.60	0.64	0.59	56.21	51.51	54.71	50.14
36	0	-1	0	0	0	0.67	0.61	0.66	0.60	57.59	52.87	56.12	51.56
37	0	0	1	0	0	0.66	0.60	0.65	0.60	56.12	51.49	54.65	50.16
38	0	0	-1	0	0	0.67	0.62	0.66	0.61	57.48	52.74	56.03	51.43
39	0	0	0	1	0	0.66	0.60	0.65	0.60	55.96	51.31	54.47	49.97
40	0	0	0	-1	0	0.67	0.62	0.66	0.61	57.53	52.81	56.08	51.50
41	0	0	0	0	1	1.05	0.96	1.03	0.95	98.07	89.96	96.53	88.57
42	0	0	0	0	-1	0.20	0.18	0.20	0.19	20.13	18.44	17.92	16.46
43	0	0	0	0	0	0.66	0.61	0.66	0.61	56.63	51.95	55.94	51.07

The R^2 and R_d^2 values of the mean and standard deviation of the seismic damage parameters of the side piers, middle piers, and their corresponding bearings were calculated using Equations (17) and (18), as shown in Table 11.

Table 11. Calculation results of R^2 and R_d^2 .

Parameters	Bridge Piers				Bearings			
	μ_d of Side Piers		μ_d of Middle Piers		d of Side Pier Bearings/mm		d of Middle Pier Bearings/mm	
	Mean Value	Standard Deviation	Mean Value	Standard Deviation	Mean Value	Standard Deviation	Mean Value	Standard Deviation
R^2	0.9998	0.9998	0.9998	0.9997	0.9998	0.9998	0.9998	0.9998
R_d^2	0.9997	0.9997	0.9995	0.9995	0.9997	0.9997	0.9997	0.9997

From Table 11, both R^2 and R_a^2 exceed 0.9, confirming that the CCD method effectively captures trends in the simulation data and accurately reflects the influence of each random variable on the seismic damage of the bridge.

5.2.3. Solution of Response Surface Model

By integrating the data from Table 10 with Equation (16), the response surface model was solved. The response surface functions for the mean and standard deviation of the displacement ductility ratio u_d of side piers are shown in Equation (19) and Equation (20), respectively.

$$\begin{aligned} u_{d,m} = & 0.663 - 0.018x_1 - 0.011x_2 - 0.013x_3 - 0.011x_4 + 0.424x_5 \\ & - 0.004x_1^2 + 0.001x_2^2 + 0.001x_3^2 + 0.001x_4^2 - 0.039x_5^2 \\ & + 0.001x_1x_2 - 0.002x_1x_3 - 0.001x_1x_4 - 0.004x_1x_5 \\ & - 0.001x_2x_3 + 0.001x_2x_4 - 0.001x_2x_5 \\ & - 0.003x_3x_4 - 0.003x_3x_5 - 0.004x_4x_5 \end{aligned} \quad (19)$$

$$\begin{aligned} u_{d,\sigma} = & 0.607 - 1.294 \times 10^{-2}x_1 - 1.059 \times 10^{-2}x_2 - 1.088 \times 10^{-2}x_3 - 1.147 \times 10^{-2}x_4 + 3.897 \times 10^{-1}x_5 \\ & - 1.737 \times 10^{-3}x_1^2 - 1.737 \times 10^{-3}x_2^2 + 3.263 \times 10^{-3}x_3^2 + 3.263 \times 10^{-3}x_4^2 - 3.674 \times 10^{-2}x_5^2 \\ & + 1.563 \times 10^{-3}x_1x_2 - 3.438 \times 10^{-3}x_1x_3 - 3.125 \times 10^{-4}x_1x_4 - 2.813 \times 10^{-3}x_1x_5 \\ & - 1.563 \times 10^{-3}x_2x_3 + 1.563 \times 10^{-3}x_2x_4 - 9.375 \times 10^{-4}x_2x_5 \\ & - 2.188 \times 10^{-3}x_3x_4 - 3.438 \times 10^{-3}x_3x_5 - 1.563 \times 10^{-3}x_4x_5 \end{aligned} \quad (20)$$

The response surface functions for the mean and standard deviation of the displacement ductility ratio u_d of middle piers are shown in Equation (21) and Equation (22), respectively.

$$\begin{aligned} u_{d,m} = & 0.653 - 1.147 \times 10^{-2}x_1 - 1.265 \times 10^{-2}x_2 - 1.118 \times 10^{-2}x_3 - 1.118 \times 10^{-2}x_4 + 4.176 \times 10^{-1}x_5 \\ & - 2.487 \times 10^{-3}x_1^2 - 2.487 \times 10^{-3}x_2^2 + 2.513 \times 10^{-3}x_3^2 + 2.513 \times 10^{-3}x_4^2 - 3.749 \times 10^{-2}x_5^2 \\ & + 9.375 \times 10^{-4}x_1x_2 - 3.125 \times 10^{-4}x_1x_3 - 3.125 \times 10^{-4}x_1x_4 - 2.188 \times 10^{-3}x_1x_5 \\ & - 3.125 \times 10^{-4}x_2x_3 - 3.125 \times 10^{-4}x_2x_4 - 2.188 \times 10^{-3}x_2x_5 \\ & - 1.563 \times 10^{-3}x_3x_4 - 3.438 \times 10^{-3}x_3x_5 - 4.688 \times 10^{-3}x_4x_5 \end{aligned} \quad (21)$$

$$\begin{aligned} u_{d,\sigma} = & 0.602 - 1.029 \times 10^{-2}x_1 - 1.206 \times 10^{-2}x_2 - 9.118 \times 10^{-3}x_3 - 1.088 \times 10^{-2}x_4 + 3.812 \times 10^{-1}x_5 \\ & - 6.046 \times 10^{-3}x_1^2 - 6.046 \times 10^{-3}x_2^2 + 3.954 \times 10^{-3}x_3^2 + 3.954 \times 10^{-3}x_4^2 - 3.105 \times 10^{-2}x_5^2 \\ & + 1.250 \times 10^{-3}x_1x_2 - 6.250 \times 10^{-4}x_1x_3 - 3.908 \times 10^{-17}x_1x_4 - 2.500 \times 10^{-3}x_1x_5 \\ & - 1.250 \times 10^{-3}x_2x_3 + 6.250 \times 10^{-4}x_2x_4 - 1.875 \times 10^{-3}x_2x_5 \\ & - 1.250 \times 10^{-3}x_3x_4 - 3.750 \times 10^{-3}x_3x_5 - 3.125 \times 10^{-3}x_4x_5 \end{aligned} \quad (22)$$

The response surface functions for the mean and standard deviation of the shear deformation d of side pier bearings are shown in Equation (23) and Equation (24), respectively.

$$\begin{aligned} d_\mu = & 56.812 - 1.466x_1 - 1.502x_2 - 1.608x_3 - 1.580x_4 + 38.965x_5 \\ & - 8.989 \times 10^{-2}x_1^2 + 6.511 \times 10^{-2}x_2^2 - 3.489 \times 10^{-2}x_3^2 - 8.989 \times 10^{-2}x_4^2 + 2.265x_5^2 \\ & - 8.375 \times 10^{-2}x_1x_2 - 3.000 \times 10^{-2}x_1x_3 + 1.375 \times 10^{-1}x_1x_4 + 4.363 \times 10^{-1}x_1x_5 \\ & + 5.750 \times 10^{-2}x_2x_3 + 5.500 \times 10^{-2}x_2x_4 + 3.238 \times 10^{-1}x_2x_5 \\ & - 1.375 \times 10^{-2}x_3x_4 + 3.650 \times 10^{-1}x_3x_5 + 3.513 \times 10^{-1}x_4x_5 \end{aligned} \quad (23)$$

$$\begin{aligned} d_\sigma = & 52.119 - 1.343x_1 - 1.430x_2 - 1.473x_3 - 1.472x_4 + 35.755x_5 \\ & - 8.024 \times 10^{-2}x_1^2 + 4.976 \times 10^{-2}x_2^2 - 2.524 \times 10^{-2}x_3^2 - 8.024 \times 10^{-2}x_4^2 + 2.060x_5^2 \\ & - 7.781 \times 10^{-2}x_1x_2 - 2.594 \times 10^{-2}x_1x_3 + 1.266 \times 10^{-1}x_1x_4 + 4.009 \times 10^{-1}x_1x_5 \\ & + 5.219 \times 10^{-2}x_2x_3 + 4.969 \times 10^{-2}x_2x_4 + 2.553 \times 10^{-1}x_2x_5 \\ & - 1.344 \times 10^{-2}x_3x_4 + 3.347 \times 10^{-1}x_3x_5 + 3.122 \times 10^{-1}x_4x_5 \end{aligned} \quad (24)$$

The response surface functions for the mean and standard deviation of the shear deformation d of middle pier bearings are shown in Equation (25) and Equation (26), respectively.

$$\begin{aligned}
d_{\mu} = & 55.381 - 1.463x_1 - 1.439x_2 - 1.345x_3 - 1.425x_4 + 39.280x_5 \\
& + 3.786 \times 10^{-3}x_1^2 + 1.038 \times 10^{-1}x_2^2 + 2.879 \times 10^{-2}x_3^2 - 3.621 \times 10^{-2}x_4^2 + 1.914x_5^2 \\
& + 5.750 \times 10^{-2}x_1x_2 + 1.238 \times 10^{-1}x_1x_3 + 1.150 \times 10^{-1}x_1x_4 + 3.050 \times 10^{-1}x_1x_5 \\
& + 1.106 \times 10^{-1}x_2x_3 + 6.813 \times 10^{-2}x_2x_4 + 2.706 \times 10^{-1}x_2x_5 \\
& + 1.006 \times 10^{-1}x_3x_4 + 2.744 \times 10^{-1}x_3x_5 + 1.956 \times 10^{-1}x_4x_5
\end{aligned} \quad (25)$$

$$\begin{aligned}
d_{\sigma} = & 50.792 - 1.344x_1 - 1.385x_2 - 1.236x_3 - 1.328x_4 + 36.033x_5 \\
& + 1.310 \times 10^{-2}x_1^2 + 9.310 \times 10^{-2}x_2^2 + 3.810 \times 10^{-2}x_3^2 - 2.190 \times 10^{-2}x_4^2 + 1.758x_5^2 \\
& + 5.313 \times 10^{-2}x_1x_2 + 1.144 \times 10^{-1}x_1x_3 + 1.069 \times 10^{-1}x_1x_4 + 2.813 \times 10^{-1}x_1x_5 \\
& + 1.031 \times 10^{-1}x_2x_3 + 6.438 \times 10^{-2}x_2x_4 + 2.075 \times 10^{-1}x_2x_5 \\
& + 9.438 \times 10^{-2}x_3x_4 + 2.525 \times 10^{-1}x_3x_5 + 1.763 \times 10^{-1}x_4x_5
\end{aligned} \quad (26)$$

5.3. Plotting of Seismic Fragility Curves for the Research Object

5.3.1. Calculation of Seismic Exceedance Probability Based on Monte Carlo Method

Using the probability distributions of E_c , γ , f_y and E_s , 10,000 bridge samples were generated via the Monte Carlo Simulation. The PGA range was divided into 20 intensity levels from 0 to 1.0 g with an interval of 0.05 g. By inputting the 10,000 bridge samples across those 20 PGA levels through Equations (19) to (26), the corresponding seismic damage parameter values were obtained [42].

Combining Tables 3 and 4, the number of bridge samples that reached and exceeded each seismic damage level under different PGA intensity levels, along with the corresponding exceedance probabilities were calculated using the method shown in Equation (27) [43].

$$P_f = N(\mu \geq LS_f) / 10,000 \quad (27)$$

where P_f denotes the exceedance probability of each level of seismic damage under different PGA intensity, where $f = 1, 2, 3, 4$ and 5 representing the intact, slight damage, moderate damage, severe damage and collapse states, respectively $N(\mu \geq LS_f)$, represents the number of bridge samples reaching or exceeding the f -th level of seismic damage.

The calculated exceedance probability for each seismic damage level of the bridge piers and pier bearings is shown in Tables 12–15, respectively.

Table 12. Exceedance probabilities of various seismic damage levels for side piers of the research object.

PGA/g	P_1 of Side Piers	P_2 of Side Piers	P_3 of Side Piers	P_4 of Side Piers	P_5 of Side Piers
0.00	0	0	0	0	0
0.05	0	0	0	0	0
0.10	0	0	0	0	0
0.15	0.0008	0	0	0	0
0.20	0.1154	0.0003	0	0	0
0.25	0.3887	0.1028	0.0004	0	0
0.30	0.6621	0.3119	0.1017	0.0003	0
0.35	0.7752	0.6144	0.2891	0.0892	0
0.40	0.8821	0.7282	0.5881	0.1778	0
0.45	0.9122	0.8267	0.6892	0.2871	0
0.50	0.9282	0.8861	0.7902	0.3901	0
0.55	0.9428	0.9122	0.8236	0.4578	0
0.60	0.9661	0.9318	0.8618	0.5129	0
0.65	0.9707	0.9459	0.8891	0.5782	0.0004
0.70	0.9788	0.9577	0.9208	0.6278	0.0278
0.75	0.9826	0.9681	0.9418	0.6839	0.0472
0.80	0.9878	0.9781	0.9565	0.7208	0.0987

Table 12. *Cont.*

PGA/g	P_1 of Side Piers	P_2 of Side Piers	P_3 of Side Piers	P_4 of Side Piers	P_5 of Side Piers
0.85	0.9918	0.9819	0.9708	0.7677	0.132
0.90	0.9926	0.9912	0.9812	0.8001	0.197
0.95	0.9938	0.9933	0.9911	0.8219	0.277
1.00	0.9968	0.9948	0.9922	0.8302	0.329

Table 13. Exceedance probabilities of various seismic damage levels for middle piers of the research object.

PGA/g	P_1 of Middle Piers	P_2 of Middle Piers	P_3 of Middle Piers	P_4 of Middle Piers	P_5 of Middle Piers
0.00	0	0	0	0	0
0.05	0	0	0	0	0
0.10	0	0	0	0	0
0.15	0.0008	0	0	0	0
0.20	0.1108	0.0003	0	0	0
0.25	0.3711	0.1011	0.0004	0	0
0.30	0.5782	0.2881	0.1011	0.0003	0
0.35	0.7231	0.6101	0.2871	0.0873	0
0.40	0.8652	0.7218	0.5822	0.1725	0
0.45	0.9016	0.8211	0.6865	0.2842	0
0.50	0.9188	0.8812	0.7842	0.3881	0
0.55	0.9392	0.9082	0.8202	0.4566	0
0.60	0.9612	0.9288	0.8588	0.5019	0
0.65	0.9684	0.9415	0.8878	0.5725	0.0004
0.70	0.9721	0.9566	0.9181	0.6225	0.0255
0.75	0.9811	0.9623	0.9402	0.6811	0.0461
0.80	0.9821	0.9702	0.9556	0.7188	0.0977
0.85	0.9861	0.9735	0.9661	0.7608	0.129
0.90	0.9877	0.9821	0.9772	0.7942	0.188
0.95	0.9901	0.9877	0.9812	0.8128	0.256
1.00	0.9912	0.9901	0.9892	0.8218	0.318

Table 14. Exceedance probabilities of various seismic damage levels for side pier bearings of the research object.

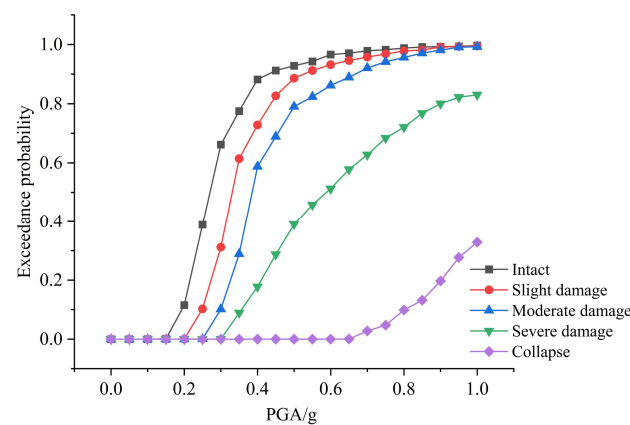
PGA/g	P_1 of Side Pier Bearings	P_2 of Side Pier Bearings	P_3 of Side Pier Bearings	P_4 of Side Pier Bearings	P_5 of Side Pier Bearings
0.00	0	0	0	0	0
0.05	0.0021	0	0	0	0
0.10	0.0028	0	0	0	0
0.15	0.3122	0.0004	0	0	0
0.20	0.6651	0.0181	0	0	0
0.25	0.8342	0.2118	0.0003	0	0
0.30	0.9028	0.5542	0.0074	0	0
0.35	0.9374	0.7758	0.0618	0.0001	0
0.40	0.9529	0.8869	0.2529	0.0006	0
0.45	0.9708	0.9328	0.5081	0.0038	0
0.50	0.9823	0.9544	0.7118	0.0296	0.008
0.55	0.9914	0.9682	0.8215	0.1172	0.0032
0.60	0.9933	0.9732	0.8891	0.3028	0.0129
0.65	0.9951	0.9838	0.9322	0.5182	0.0571
0.70	0.9968	0.9907	0.9588	0.7088	0.1682
0.75	0.9978	0.9958	0.9762	0.8618	0.3822
0.80	0.9987	0.9986	0.9881	0.9442	0.6621
0.85	0.9994	0.9991	0.9916	0.9731	0.7824
0.90	1	0.9999	0.9982	0.9918	0.8627
0.95	1	1	0.9997	0.9926	0.9318
1.00	1	1	1	0.9965	0.9688

Table 15. Exceedance probabilities of various seismic damage levels for middle pier bearings of the research object.

PGA/g	P_1 of Middle Pier Bearings	P_2 of Middle Pier Bearings	P_3 of Middle Pier Bearings	P_4 of Middle Pier Bearings	P_5 of Middle Pier Bearings
0.00	0	0	0	0	0
0.05	0.0011	0	0	0	0
0.10	0.0019	0	0	0	0
0.15	0.2871	0.0003	0	0	0
0.20	0.6341	0.0168	0	0	0
0.25	0.7711	0.1475	0.0002	0	0
0.30	0.8574	0.4682	0.0058	0	0
0.35	0.8991	0.7147	0.0521	0.0001	0
0.40	0.9288	0.8318	0.2177	0.0005	0
0.45	0.9481	0.8874	0.4838	0.0021	0
0.50	0.9618	0.9478	0.6688	0.0189	0.0007
0.55	0.9786	0.9608	0.8081	0.1098	0.0025
0.60	0.9824	0.9728	0.8691	0.2575	0.0118
0.65	0.9902	0.9811	0.9219	0.4671	0.0451
0.70	0.9936	0.9901	0.9489	0.6577	0.1287
0.75	0.9974	0.9948	0.9681	0.8429	0.2881
0.80	0.9988	0.9977	0.9779	0.9177	0.5908
0.85	0.9999	0.9991	0.9862	0.9528	0.7521
0.90	1	0.9997	0.9972	0.9762	0.8489
0.95	1	0.9999	0.9995	0.9914	0.9118
1.00	1	1	0.9999	0.9948	0.9337

5.3.2. Seismic Fragility Curves of Bridge Piers

The seismic fragility curves of the side piers and middle piers were plotted, respectively, with the PGA on the horizontal axis and the exceedance probabilities of different seismic damage levels on the vertical axis, as shown in Figures 8 and 9.

**Figure 8.** Seismic fragility curves of side piers.

From Figures 8 and 9 show that as PGA increases, the exceedance probabilities of side piers and middle piers developing from intact state to collapse gradually rise, indicating a continuous increase in the fragility of bridge piers. At low PGA levels, the probability of slight damage is nearly zero, confirming minimal seismic effects on bridge piers. For slight and moderate damage states, the probability of damage increases rapidly until the PGA reaches approximately 0.3 g, beyond this value, the growth rate slows down and stabilizes. The fragility curves for slight and moderate damage are very close, which is attributed to the small difference in load-carrying capacity between these states, reflecting similar yield and equivalent yield curvatures in the pier reinforcement.

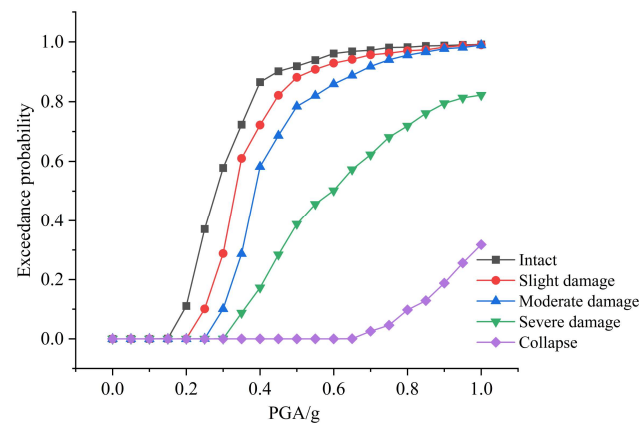


Figure 9. Seismic fragility curves of middle piers.

When the PGA gradually increases to about 0.5 g, severe damage begins to become distinctly apparent. The gap between the curves for moderate and severe damage indicates that the piers have a ductile behavior during this transition, which reduces the probability of severe damage and indicates a substantial difference between the curvature at equivalent yield and that at concrete crushing. Only when the PGA increases further do the piers begin to show complete collapse, and at the same PGA level, the exceedance probability of complete collapse is much lower than that of slight, moderate, or severe damage.

5.3.3. Seismic Fragility Curves of Bearings

Taking PGA as the horizontal axis and the exceedance probabilities of different seismic damage levels as the vertical axis, the seismic fragility curves for the side pier bearings and middle pier bearings of the research object were, respectively, plotted, as shown in Figures 10 and 11.

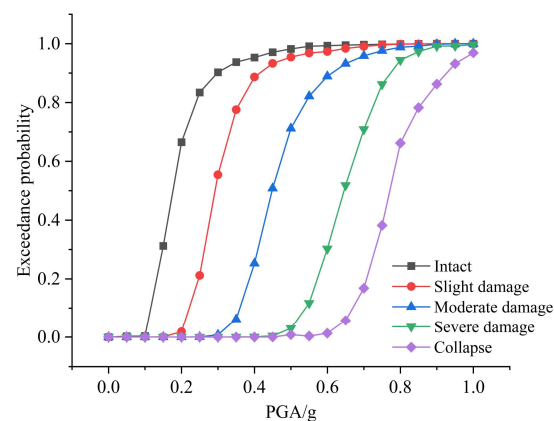


Figure 10. Seismic fragility curves of side pier bearings.

From Figures 10 and 11, as PGA increase, the exceedance probabilities of bearings developing from an intact state to slight, moderate, severe damage, and collapse gradually rise, indicating growing fragility of bearings. At low PGA, the probability of slight damage is nearly zero, suggesting negligible seismic demands on the bearings. For slight and moderate damage, the probability of damage rises rapidly until the PGA reaches approximately 0.4 g; beyond which the rate increase slows and stabilizes. The fragility curves of slight and moderate damage lie close together, due to the small difference in load-carrying capacity between these damage states, indicating only minor changes in mechanical properties.

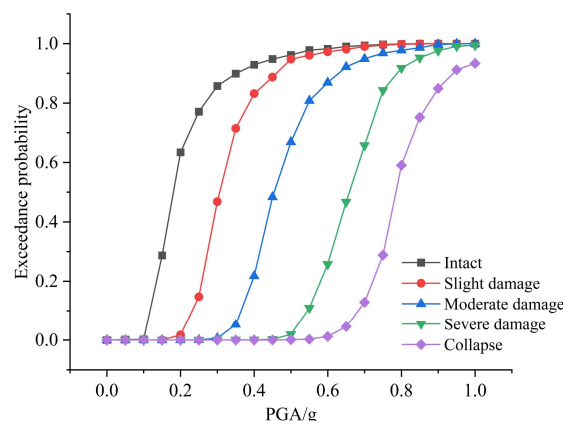


Figure 11. Seismic fragility curves of middle pier bearings.

When the PGA increases to about 0.6 g, severe damage begins to appear. The notable separation between the moderate and severe damage curves, primarily the bearings, has a ductile behavior during this transition, which substantially reduces the probability of severe damage and indicates significant changes in mechanical properties between these damage stages. Only when the PGA further increases does the bearing exhibit collapse, and at the same PGA level, the exceedance probability of collapse is much lower than that of the other damage states.

5.4. Comparative Analysis and Methodological Enhancement

Conventional seismic fragility analyses often simplify or omit the explicit treatment of parametric uncertainties, which can lead to an incomplete assessment of structural fragility. For instance, the fragility assessment of RC frames by Abeyesiriwardena et al. did not incorporate the uncertainties in key structural properties [12]. In contrast, the method proposed in this study provides a significant enhancement by systematically quantifying the coupled effects of uncertainties from four structural parameters and PGA. This is achieved by the integration of the Response Surface Method with large-scale Monte Carlo simulation, resulting in fragility curves that offer a more realistic and reliable representation of seismic performance, thereby establishing a more robust basis for risk assessment.

6. Seismic Probability Risk Assessment of the Research Object Based on Seismic Hazard Curve Method

6.1. Basic Theory

The seismic risk probability is mathematically defined as the convolution of seismic hazard and fragility, representing the integrated probability of exceeding each level of seismic damage under all seismic events over the next t years, as shown in Equation (28) [44].

$$P_{j,t} = \sum_{\text{all } x_i} P(R_{j-} \leq S | \text{PGA} = x_i) \cdot P_t(\text{PGA} = x_i) \quad (28)$$

where $P_{j,t}$ is the risk probability of exceeding the j -th damage level over the next t years; R_{j-} represent the value of the seismic damage parameter corresponding to the j -th damage level; S is the value of the seismic damage parameter of the bridge under seismic action; x_i is the PGA that the research site may encounter over the next t years. Due to the continuity of PGA values, Equation (28) can be rewritten as Equation (29).

$$P_{j,t} = \int P(R_{j-} \leq S | \text{PGA} = x_i) \cdot f(x) dx \quad (29)$$

where $f(x)$ is the probability density function of PGA at the site over t years. The risk probability that the bridge will exceed the j -th level of seismic damage in the next 1 year as shown in Equation (30).

$$P_{j,1} = \int P(R_{j-} \leq S | \text{PGA} = x_i) dH(x) \quad (30)$$

where $H(x)$ is the annual distribution function of PGA at the site, whose shape is the seismic hazard curve, as shown in Equation (31) [45].

$$H(x) = P(\text{PGA} \geq x) \approx k_0 \cdot x^{-k} \quad (31)$$

where k_0 and k are parameters derived from the seismic hazard assessment results.

Based on the results of seismic fragility assessment, Equations (32) and (33) can be obtained.

$$P_{j,1}(R_{j-} \leq S | \text{PGA} = x_i) = \phi \left[\frac{\ln(S/R_{j-})}{\beta} \right] = 1 - \phi \left[\frac{\ln(R_{j-}/S)}{\beta} \right] \quad (32)$$

$$S_i = d_1 \cdot x_i^{d_2} \quad (33)$$

where β is the dispersion parameter relating seismic damage to ground motion intensity; d_1 and d_2 are undetermined coefficients.

Equation (30) can be rewritten as Equation (34).

$$P_{j,1} = \int P(R_{j-} \leq S | \text{PGA} = x_i) dH(x) = \int \left[1 - \phi \left(\frac{\ln(R_{j-}/d_1 \cdot x^{d_2})}{\beta} \right) \right] dH(x) \quad (34)$$

By performing integral transformation on Equation (34), Equation (35) can be obtained.

$$P_{j,1} = \int \frac{d \left\{ 1 - \phi \left\{ \frac{\ln[R_{j-}/(d_1 \cdot x^{d_2})]}{\beta} \right\} \right\}}{dx} \cdot H(x) dx \quad (35)$$

The standard normal cumulative distribution function yields Equation (36).

$$\frac{d \left\{ 1 - \phi \left\{ \frac{\ln[R_{j-}/(d_1 \cdot x^{d_2})]}{\beta} \right\} \right\}}{dx} = \frac{d_2}{x \cdot \beta} \cdot \phi \left\{ \frac{\ln[R_{j-}/(d_1 \cdot x^{d_2})]}{\beta} \right\} \quad (36)$$

Then, Equation (35) can be further rewritten as Equation (37).

$$P_{j,1} = \int \frac{d_2}{x \cdot \beta} \cdot \phi \left\{ \frac{\ln[R_{j-}/(d_1 \cdot x^{d_2})]}{\beta} \right\} \cdot H(x) dx = \int_0^\infty \frac{d_2}{x \cdot \beta} \cdot \frac{1}{\sqrt{2\pi}} \cdot \exp \left[\frac{1}{2} \cdot \left(\frac{\ln R_{j-} - \ln d_1 - d_2 \ln x}{\beta} \right)^2 \right] \cdot k_0 \cdot x^{-k} dx \quad (37)$$

Simplifying Equation (37) yields Equation (38).

$$P_{j,1} = k_0 \cdot \left(\frac{R_{j-}}{d_1} \right)^{-\frac{k}{d_2}} \cdot \exp \left(\frac{k^2 \cdot \beta^2}{2 \cdot d_2^2} \right) \quad (38)$$

From Equation (33), Equation (39) is derived.

$$PGA_{j-} = \left(\frac{R_{j-}}{d_1} \right)^{\frac{1}{d_2}} \quad (39)$$

where PGA_{j-} is the critical PGA corresponding to R_{j-} , substituting Equation (39) into Equation (38) gives Equation (40).

$$P_{j,1} = k_0 \cdot (PGA_{j-})^{-k} \cdot \exp\left(\frac{k^2 \cdot \beta^2}{2 \cdot d_2^2}\right) \quad (40)$$

Referring to Equation (37), Equation (40) can be transformed into Equation (41).

$$P_{j,1} = H(PGA_{j-}) \cdot \exp\left(\frac{k^2 \cdot \beta^2}{2 \cdot d_2^2}\right) \quad (41)$$

6.2. Case Analysis

As shown in Tables 12–15, under the same PGA levels, the seismic damage of the side piers is more severe than middle piers. Therefore, only the side piers and their bearings were considered in the probability risk assessment. According to the results of the IDA, for the bridge piers, the parameters are $\beta = 0.8567$, $d_1 = 1.082$ and $d_2 = 0.995$, for the bearings, the parameters are $\beta = 0.8353$, $d_1 = 76.853$, and $d_2 = 0.99$.

According to the results of seismic hazard assessment, the PGA values with different exceedance probabilities for the research site over the next 1 year are shown in Table 16.

Table 16. PGA with different exceedance probabilities over 1 year.

Exceedance Probability Over 1 Year/%	0.0404	0.2105	1.9689
PGA/g	0.1747	0.1001	0.0292

Using Equation (31) and Table 16, the seismic hazard curve was plotted, as shown in Figure 12 and Equation (42).

$$H(x) = 0.0012 \cdot x^{-2.113} \quad (42)$$

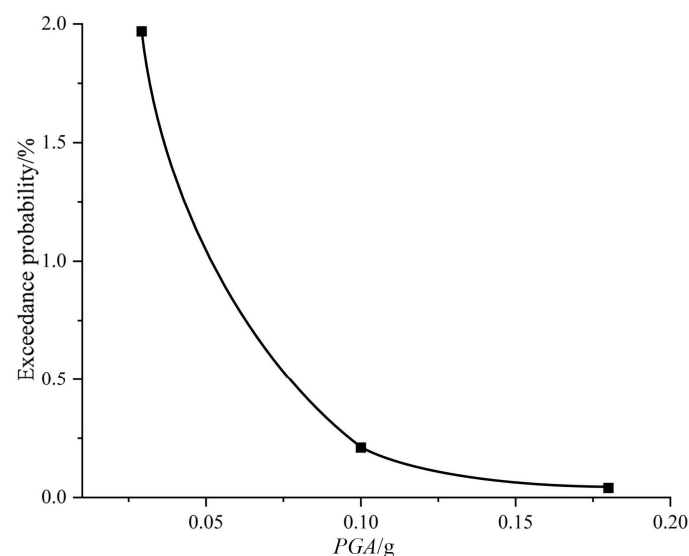


Figure 12. Seismic hazard curve.

Using Equation (39) and Tables 3 and 4, PGA_j was calculated, as shown in Tables 17 and 18.

Table 17. PGA_j corresponding to $\mu_{d,max}$.

$\mu_{d,max}$	1.000	1.132	1.893	4.893
PGA_j	0.9238	1.0464	1.7545	4.5566

Table 18. PGA_j corresponding to d_{max} .

d_{max}/mm	25	50	75	90
PGA_j	0.3216	0.6478	0.9756	1.1729

Combining Equation (41) and (42), the risk probabilities of the bridge piers exceeding each level of seismic damage over 1 year are calculated as follows, slight damage $P_{2,1} = 0.007424$, moderate damage $P_{3,1} = 0.005705$, severe damage $P_{4,1} = 0.001914$ and collapse $P_{5,1} = 0.000255$. For the bearings, the probabilities are: slight damage $P_{2,1} = 0.064627$, moderate damage $P_{3,1} = 0.014716$, severe damage $P_{4,1} = 0.006195$ and collapse $P_{5,1} = 0.004198$.

Based on the Poisson distribution, a seismic occurrence probability prediction model is constructed. According to the assumptions of the Poisson distribution, it is considered that the probability of seismic events occurring each year and their impacts are independent of each other. The conversion method for the risk probabilities of the bridge piers and bearings exceeding each level of seismic damage over 1 year and over 50 years is shown in Equation (43).

$$P_{j,50} = 1 - (1 - P_{j,1})^{50} \quad (43)$$

Following calculations, the risk probabilities of the bridge piers and bearings exceeding each level of seismic damage over 50 years are shown in Table 19.

Table 19. Assessment results of seismic damage risk probability for the research object.

Results of Probability Risk Assessment		Bridge Piers		Bearings	
		Probability of Exceedance Risk/%	Risk Probability/%	Probability of Exceedance Risk/%	Risk Probability/%
Damage States	Intact	100.00	68.90	100.00	3.54
	Slight damage	31.10	6.22	96.46	44.11
	Moderate damage	24.88	15.75	52.35	25.64
	Severe damage	9.13	7.86	26.71	7.74
	Collapse	1.27	1.27	18.97	18.97

7. Conclusions

This study systematically conducted seismic hazard, seismic fragility, and probability risk assessments using a reinforced concrete continuous girder bridge as a case study. First, the PGA with different exceedance probabilities at the study site was calculated using the CPSHA method from the fifth-generation seismic ground motion parameter zoning scheme. Second, the Monte Carlo method and Response Surface Method were integrated to characterize uncertainties in bridge structural parameters and PGA, and seismic fragility curves were plotted. Third, the risk probability was calculated based on the seismic hazard curve method, providing a theoretical basis for bridge seismic design and performance assessment. The research results were as follows.

(1) The study area is influenced by the Tancheng–Lushan, the Lower Yangtze–South Yellow Sea, and the North China Plain seismic source zones, involving a total of 46 potential seismic source zones. The PGAs with 2%, 10%, and 63% exceedance probabilities over the next 50 years at the site are 171.16 gal, 98.10 gal, and 28.61 gal, respectively, indicating that the site is in the 0.10 g seismic intensity zone corresponds to basic intensity zone VII.

(2) As PGA increases, both side piers and middle piers show clear pattern of progression from an intact state to slight, moderate, severe, and eventually collapse. The exceedance probabilities for all levels of damage rise consistently with increasing PGA, strongly confirming that the intensity of ground motion is the core factor determining the extent of bridge pier damage. At the same PGA level, the exceedance probabilities for slight, moderate, severe, and collapse damage in side piers are generally higher than those in middle piers, confirming their higher seismic fragility. When the PGA is in the range of 0.4–0.6 g, the exceedance probability of moderate damage in side piers is significantly higher than that in middle piers, reflecting that side piers are more prone to moderate damage under moderate-intensity earthquakes. At high PGA levels (>0.8 g), the exceedance probability of collapse for side piers is also notably greater than for middle piers, indicating that side piers face a higher collapse risk. Based on these findings, seismic design for bridges should prioritize enhancing the seismic performance of side piers, using measures such as optimizing the structural configuration of side piers, strengthening anti-sliding and anti-fall beam measures at bearings, and reasonably installing seismic restraint devices to improve overall bridge seismic reliability.

Similarly, as PGA increases, both side and middle pier bearings exhibit progression from intact to slight, moderate, severe damage, and collapse, with the exceedance probabilities for all damage levels increasing with PGA. This fully demonstrates that ground motion intensity is the key factor affecting bearing damage. At the same PGA level, the exceedance probabilities for all damage levels in side pier bearings are generally higher than those in middle pier bearings, and their fragility curves lie above those of middle pier bearings. This indicates that side pier bearings are more susceptible to earthquake damage and show a more significant trend of damage escalation as PGA increases. In the range of 0.4–0.6 g, the exceedance probabilities of moderate damage for side pier bearings are significantly higher, and at $\text{PGA} > 0.8$ g, their collapse probabilities are also significantly greater.

(3) Over the next 50 years, the risk probabilities for the bridge piers to remain intact suffer slight, moderate, severe damage and collapse, at 68.90%, 6.22%, 15.75%, 7.86% and 1.27%, respectively. For the bearings, the corresponding probabilities are 3.54%, 44.11%, 25.64%, 7.74% and 18.97%, indicating that the overall seismic damage risk of bearings is higher than that of piers, especially for moderate damage and collapse. The high seismic risk of bearings necessitates targeted mitigation methods: (a) optimize design by adopting high-capacity isolation bearings and strengthening anti-unseating devices; (b) enhance inspection by incorporating bearings into routine and post-earthquake monitoring programs; and (c) promote retrofitting by installing external dampers or replacing conventional bearings with isolation or damping systems to improve overall resilience.

(4) The proposed seismic hazard–fragility–probability risk assessment system is applicable to various bridge structures and has high promotional and application value. While the specific risk probabilities are case-sensitive, the overall framework is generalizable to various bridge structures. Its application requires tailoring the finite element model and damage criteria to the specific structural system under investigation. However, there are still limitations: (a) omission of soil–structure interaction, potentially underestimate local risk; (b) lack of refined modeling bearing anchor bolts and stirrup spacing in plastic hinge regions, and unquantified impacts of construction quality variations; (c) only horizontal seismic motion input is used, ignoring multi-dimensional seismic motion effects on bearing

shear deformation and pier flexural-shear failure; and (d) the validity of the finite element model relies on well-established modeling techniques rather than direct validation against experimental data from a physically identical bridge test. While the constitutive models and element formulations used have been extensively validated in the literature, future work should include a comprehensive experimental comparison to further corroborate the findings.

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