Research Article

Seismic Fragility Analysis of Multispan Continuous Girder Bridges with Varying Pier Heights considering Their Bond-Slip Behavior

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The bond-slip effect has a great influence on the seismic performance of reinforced concrete structures and ignoring it will overestimate the seismic performance of the structures. Based on the low-cyclic reversed loading experiment of a reinforced concrete column, this paper uses OpenSees to establish a nonlinear finite element model considering bond-slip and verify its correctness. In this paper, a multispan continuous girder bridge with varying pier heights is taken as an example. Considering the effect of the bond-slip behavior of steel bars, a refined finite element model based on the OpenSees platform is established to do the numerical simulation analysis. 10 seismic waves are selected from the Pacific Earthquake Engineering Research Center (PEER) according to the site condition and modulate the amplitude to 150 waves. This paper uses the incremental dynamic analysis (IDA) and the second-order reliability method to analyze the seismic fragility of bridge components and systems, respectively. Results show that the exceeding probability increases obviously when considering bond-slip, and with the increase of seismic spectral acceleration, the influence of bond-slip on the exceeding probability of components also increases; when bond-slip is considered, the difference of system fragility between the upper and lower limits under four damage states is greater than that without bond-slip.

1. Introduction

Throughout the history of bridge earthquake disasters, such as the 1994 Northridge Earthquake in the United States, the 1995 Kobe Earthquake in Japan, and the 2005 Wenchuan Earthquake in China, once the bridge as a transportation hub is damaged in the earthquake, it will lead to traffic block, and even affect the postdisaster rescue and reconstruction work. Therefore, the strengthening of bridges, the estimation of economic losses, and the performance evaluation of bridges after an earthquake as the components of bridge seismic performance research are indispensable. A fragility analysis method is one of the important component structural seismic analysis. Seismic fragility analysis originated from the early 1970s and was used for the seismic probability risk assessment of nuclear power plants [1].

Moreover, it is used to assess the fragility of postearthquake cities. In the study of bridge earthquakes, Basoz and Kirimidjian [2, 3] initially established the empirical fragility curve of bridge structures on the basis of the damage data of the Loma Prieta and Northridge earthquakes by using logistic regression analysis. The results show that the empirical vulnerability curve has a good correlation with the observed damage. Hwang et al. [4] used a nonlinear analysis method to fit the seismic demand model of bridge structures, determine the damage indices of different components, and establish the seismic fragility curve of structural systems for areas lacking seismic damage data. Karim et al. [5, 6] idealized the bridge pier into a single-degree-of-freedom system by numerical simulation method and established the theoretical fragility curve of log-normal distribution form according to the damage indices and ground motion indices

through nonlinear dynamic response analyses. Lagaros et al. [7, 8] incorporating artificial neural networks into the fragility analysis framework enhances the computational efficiency of the Monte Carlo simulation. Castaldo et al. [9] proposed the fragility curves associated with a wide range of parameter studies to study the seismic reliability of elastic structural systems equipped with friction pendulum isolators. Through a comparative analysis of seismic fragility curves, bridges are more likely to be damaged as a system than any other component [10, 11]. In reinforced concrete structures, the bond-slip between reinforcement and concrete has an important effect on the seismic performance of a structure. Bond-slip behavior is one of the most common factors that cause the damage and collapse of reinforced concrete structures in services under earthquakes [12, 13]. Bond-slip deformation is one of the components of the total lateral displacement of reinforced concrete piers, and low-cyclic reversed loading tests show that the maximum additional deformation caused by bond-slip in the base can reach more than 30% of the total deformation of pier tops [14, 15].

This paper takes the low-cyclic reversed loading experiment of the reinforced concrete column as an example, establishes a numerical model considering bond-slip based on OpenSees [16] for conducted reversed cyclic analysis, and verifies the correctness of the model by comparing the experimental results. Taking a multispan continuous girder bridge with varying pier heights as the case analysis, two different nonlinear finite element models with and without bond-slip are established, and suitable seismic waves are selected from PEER according to the site condition. This paper uses the incremental dynamic analysis and the second-order reliability method to analyze the seismic fragility of bridge components and systems, respectively, so as to study the influence of bond-slip on its seismic performance.

2. Model Validation with Experimental Results

This paper selects the results of a low-cyclic reversed loading experiment of reinforced concrete columns [17] as the basis for comparison and refers to the definition of the relevant numerical model [18–20]. Specimen design parameters are as follows: Specimen is made of C40 concrete, the compressive strength of the concrete cube is 32.1 MPa, and the elastic modulus is 33092.4 MPa. The stirrup is made of HPB300 steel bar with a yield strength of 390 MPa and tensile strength of 494 MPa. Longitudinal reinforcement is made of HRB400 steel bar with a yield strength of 525 MPa, and tensile strength of 665 MPa; the concrete cover thickness is 20 mm, the axial compression ratio is 0.4, assuming the column is fixed at the bottom and free at the top.

The experimental loading mode is the load-displacement dual-control loading method. Axial force loading is carried out first, the specimen is loaded under load controlled cyclic loading before reaching the yield state, and the initial load is calculated at 50% of the cracking load; the ultimate load should be lower than 85% of the peak load. The specimen is loaded under displacement controlled cyclic loading after yielding, the ultimate load should be lower than 85% of the

peak load, and the specimen should stop loading after reaching the ultimate state. The loading protocol is shown in Figure 1.

To study the influence of the bond-slip effect on the reinforced concrete column, the finite element model of reinforced concrete columns with and without bond-slip effect is established by using OpenSees. Fix the bottom nodes of the column to simulate the ground anchor, the column height is equivalent to 1000 mm, and 10 units are divided along the column length. The design and simplified model of the reinforced concrete column is shown in Figure 2. The concrete numerical model is simulated by Concrete01 in OpenSees (Figure 3). Mander [21] proved that stirrups can effectively confine the core concrete and improve its strength and ductility, and the effect of core concrete of the stirrup should be considered in the process of the finite element modeling. Based on the Mander's model [21] (Figure 3), this paper replaces the confined effect of stirrup on core concrete with effective uniform lateral stress to correct the stress-strain constitutive relation of core concrete so that the ultimate capacity and ductility of bridges can be evaluated more effectively. The steel numerical model is simulated by Steel02 in OpenSees (Figure 3). The Steel02 model modifies the Giuffre–Menegotto–Pinto model [22], considering the Bauschinger and isotropic reinforcement effects. Based on a fiber beam-column element, the bond-slip constitutive is simulated by Bond_SP01 in OpenSees. Figure 3 shows the stress-slip relationship curves under monotonic and cyclic loadings, respectively. S_y is determined based on Zhao and Sritharan [23] as follows:

$$S_y = 2.54 \times \left[\frac{d_b}{8437} \frac{f_y}{\sqrt{f'_c}} \times (2\alpha + 1) \right]^{(1/\alpha)} + 0.34, \quad (1)$$

where d_b is the reinforcement diameter, f_y is the yield strength of reinforcement, f'_c is the concrete compressive strength, α is the parameter of the local bond-slip relation and taken as 0.4 according to CEB-FIP Model 90 [24], b is 0.3, and R is 0.5 in the bond-slip command.

The numerical simulation hysteretic curves obtained from the calculation of the two working conditions were compared with the test results, as shown in Figure 4. The three types of hysteretic curves are relatively full, indicating that the seismic performance of the concrete column is better, and the bearing capacity and strength of the concrete column have no obvious change in the same level of cyclic loading. The hysteretic curve is plump and arcuate, indicating that the experimental reinforced concrete column has a shear failure or bond-slip, and the absence of serious pinching effect indicates that the contribution of bending failure is greater than that of shear failure, so there is little difference between loading stiffness and unloading stiffness, and the pinching effect is not obvious [25, 26]. According to the comparison of the finite element analysis results, it can be seen that the unloading/reloading stiffness of considering bond-slip is more consistent with that of the experimental. The maximum displacement of the column top in the experiment is -35.18 mm, and it in numerical simulation

FIGURE 1: Loading protocol.

FIGURE 2: Blueprint and simplified model of the specimen.

without considering bond-slip and with considering bond-slip are -32.66 mm and -35.343 mm, respectively.

3. Case Study

3.1. Bridge Description. In this study, incremental dynamic analysis is adopted to analyze the fragility of a multispan continuous beam bridge with a high pier. Figure 3 shows the bridge elevation layout. C50 concrete is adopted for the single-cell box section of the main beam, which is 3.35 m high and 12.2 m wide. The section area of the main beam is

9.52 m^2 , the torsional inertia moment of deck section J is 25.76 m^4 , the bending moment of the inertia of cross-section in z -axis I_z is 14.29 m^4 , and I_y is 82.42 m^4 in the y -axis direction. The concrete cover thickness of piers is 0.09 m, 1[#], 4[#], and 5[#] piers poured with C40 concrete, and the axial compression ratio is 0.238; 2[#] and 3[#] piers poured with C50 concrete, and the axial compression ratio is 0.197. Longitudinal reinforcement is made of HRB335; the stirrup is made of R235. The geological condition of the bridge site is good, and the spread footing foundation is surrounded by weakly weathered rocks, which is used C25 concrete. The

FIGURE 3: Finite element model and schematic diagram of the bridge.

FIGURE 4: Hysteretic curve.

seismic fortification intensity of the area where the bridge is located is 7°, the earthquake is divided into the first group, and the ground type is II.

3.2. Finite Element Model. The main girder of bridges is less likely to enter the plastic stage under earthquake actions; therefore, the main girder in this study is simulated by the elastic beam-column element. The element mass per unit length includes the self-weight and secondary loads. Take the 2[#] rigid frame pier as an example. The pier section is hexagonal, its top is fixed with the main girder, the bottom is a 1.5 m thick spread footing foundation, and the concrete cover thickness is 9 cm, considering the confining effect of the core concrete by the stirrup; using the Mander model

cross-section can be divided into confined and unconfined concrete, which is located in the core concrete and the concrete cover, respectively, as shown in Figure 3. In the finite element model, the pier body is divided into 37 nonlinear beam-column elements, and each element is provided with three integration points, except that the length of element 37 is 0.44 m. The other 36 elements are all 1 m. Fix the spread footing foundation at the bottom of the rigid frame pier without considering the interaction between pile and soil. The corner and displacement deformations caused by bond-slip are simulated by the zero-length section element. The consolidation of the pier top and beam is simplified to a zero-length element. According to Berry's proposed criteria [27], the section of rigid frame piers is divided into quadrilateral sections. This case contains high

piers that are prone to bending failure; therefore, the nonlinear beam-column element is used to simulate the plastic deformation of the pier end.

1[#], 4[#], and 5[#] piers are connected with the main beam by QZ12500 bridge spherical bearing, 2[#] and 3[#] piers are connected with the main girder by rigid connection, and abutments adopt QZ6000 bridge spherical bearing. Large vertical stiffness is defined to simulate vertical constraints. The horizontal stiffness value of QZ12500 is 125000 kN/m, and the horizontal stiffness value of QZ6000 is 60000 kN/m. The details of the bearing are shown in Figure 3. The left abutment is equipped with a modular expansion device of 2240 mm, and the right abutment is provided with an MTL-320 expansion device. As shown in Figure 3, the effect between the abutment and the main girder is simplified to the longitudinal and transverse stiffness [28], and two rigid arm elements with the same width as the main beam are established at the beam end. The abutment is simulated by the zero-length element. This study only explores the longitudinal seismic response of the bridge under the action of earthquakes, ignoring the effect of the lateral fill, assuming that lateral and vertical as fully rigid, considering the influence of the expansion joint, and simulating by the hyperbolic gap material constitutive model in the longitudinal bridge upward. The material constitutive model is consistent with the foregoing. For 2[#] and 3[#] piers, S_y is 0.525 mm; for 1[#], 4[#], and 5[#] piers, S_y is 0.585 mm; b is 0.3, and R is 0.5 in the bond-slip command.

3.3. Selected Ground Motion Records. At present, there are two kinds of input methods: the synthetic ground motion method and the existing ground motion collection method. The former ignores the seismic characteristics of the response spectrum and cannot fully reflect the random characteristics of actual ground motion. However, the Pacific Earthquake Engineering Research Center (PEER) provides rich ground motion records and fully considers the randomness of ground motion and response spectrum; the ground motion index mainly includes peak acceleration (PGA) and spectral acceleration (SA), and the SA is selected as the ground motion index in this paper in order to make the ground motion sufficiently discrete. The spectrum-compatibility criterion selected in this paper refers to related research [29–32], and when selecting ground motion, the three factors of amplitude, spectrum characteristics, and duration should be considered comprehensively. The details are as follows.

(1) The peak acceleration of the selected ground motion should be consistent with the peak value of frequent and rare earthquakes at the bridge site; (2) according to the ground type, the average shear wave velocity (V_{s30}) corresponding to ground type II is selected to make the response spectrum of the selected ground motion consistent with the actual response spectrum as far as possible (as shown in Table 1); (3) the duration of the seismic wave should be 5 to 10 times of the basic period of the bridge, so as to ensure that the pier can receive enough seismic damage energy; (4) the epicentre distance (R) should be more than 20 km to exclude the near-

field vibration records. The response spectrum and details are shown in Figure 5 and Table 2.

According to equation (2), the SA of 10 actual ground motion records is randomly generated into 10 groups of 150 ground motion records within the range of 0.01 g–1.0 g.

$$a_g^{(i)}(t) = k_i a(t), \quad (2)$$

where $a_g^{(i)}(t)$ is the ground motion record after the i th amplitude modulation; $a(t)$ is the original ground motion record; k_i is the amplitude modulation coefficient.

4. Fragility Analysis

Seismic fragility refers to the conditional probability that the damage degree of the structure exceeds the specific damage state under the action of different earthquake intensities. The seismic vulnerability curve is expressed as follows [33]:

$$P_f = P[DI \geq C|IM], \quad (3)$$

$$P_f = P_r\left(\frac{S_c}{S_d} \leq 1\right) = P_r\left(\ln\left(\frac{S_c}{S_d}\right) \leq 0\right), \quad (4)$$

$$S_c = \ln(\mu_c, \beta_c), \quad (5)$$

$$S_d = \ln(\mu_d, \beta_d), \quad (6)$$

$$P_f = \Phi\left(\frac{\ln \mu_d - \ln \mu_c}{\sqrt{\beta_c^2 + \beta_d^2}}\right), \quad (7)$$

where P_f is the exceeding probability, IM is the ground motion intensity measure, which takes the SA, DI is the damage index, C is the seismic response, S_c is the structural capability, and S_d is the structural requirement. According to classical reliability theory, the two are often assumed to obey the log-normal distribution of two parameters [5, 33]. μ_c and β_c represent the mean and logarithmic standard deviation of the seismic capacity, respectively. μ_d and β_d represent the mean and logarithmic standard deviation of structural demand, respectively. Equations (5) and (6) are substituted by equation (4) to obtain equation (7). SA is 0.4 when it is an independent variable [34].

Cornell et al. [35] assumed that an exponential relationship exists between μ_d and IM (equation (8)). The log of both sides of equation (8) should be taken to obtain the probabilistic seismic demand model, as shown in equation (9), and substitute equation (8) with equation (6) to obtain the seismic fragility function, as shown in equation (10).

$$\mu_d = e^a \cdot IM^b, \quad (8)$$

$$\ln(\mu_d) = a + b \cdot \ln(IM), \quad (9)$$

$$P_f = \Phi\left(\frac{a + b \ln(IM) - \ln \mu_c}{\sqrt{\beta_c^2 + \beta_d^2}}\right), \quad (10)$$

TABLE 1: The average shear wave velocity (V_{s30}).

V_{s30} (m/s)	<150	150–260	260–510	>510
Ground type	I	II	III	IV

where a and b are the statistical regression coefficients, and the remaining unknown variables are the same as the study above.

4.1. Analysis of Pier Fragility. The fragility analysis of the pier usually adopts the displacement ductility and drift angle or pier top drift ratios as the pier damage index, and bridge damage is divided into five states [36, 37]. As the pier is mainly damaged by bending in this example, the displacement ductility ratio is adopted to define the damage state [36], which is defined as follows:

$$\mu_d = \frac{\Delta}{\Delta_{cy1}}, \quad (11)$$

where Δ is the relative displacement at the top of the pier, and Δ_{cy1} is the relative displacement of the pier when the longitudinal reinforcement reaches the first yield. From the displacement ductility ratio at the first yield of reinforcement bar, equivalent yield displacement ductility ratio, the displacement ductility ratio with the compressive strain of concrete reaches 0.004, and the maximum displacement ductility ratio; four damage states are obtained, namely slight, moderate, extensive, and complete damage states. The damage index calculation process is as follows:

$$\begin{aligned} \mu_{cy1} &= \frac{\Delta_{cy1}}{\Delta_{cy1}} = 1, \\ \mu_{cy} &= \frac{\Delta_{cy}}{\Delta_{cy1}} = \frac{(1/3)\phi_y L^2}{(1/3)\phi'_y L^2} = \frac{\phi_y}{\phi'_y}, \\ \theta_{p4} &= L_p \times (\phi_{c4} - \phi_y), \\ \Delta_{c4} &= \Delta_{cy} + \Delta_{p4} = \frac{1}{3}\phi_y L^2 + \theta_{p4} \left(L - \frac{1}{2}L_p \right), \\ \mu_{c4} &= \frac{\Delta_{c4}}{\Delta_{cy1}}, \\ \mu_{cmax} &= \mu_{c4} + 3 [38], \end{aligned} \quad (12)$$

where Δ_{cy} is the relative displacement of the pier top when the pier section reaches effective yield, Δ_{c4} is the relative displacement of the pier top when the compressive strain in concrete is 0.004, ϕ'_y is the curvature at the first yield, ϕ_y is the effective yield curvature; ϕ_{c4} is the curvature at the compressive strain in concrete which is 0.004, L is the distance from the plastic hinge section to the point of contraflexure, and L_p is the plastic hinge length [39].

On the basis of the above calculation, the damage indices of the piers under different ultimate states can be obtained (Table 3).

FIGURE 5: Acceleration spectra.

Nonlinear dynamic time-history analysis is used to analyze the seismic response of piers under two different conditions. The probabilistic demand model of each pier is shown in Table 4. In this paper, 1[#], 4[#] piers have similar height, same concrete type, connection with the main beam, and similar bending resistance during the service period, as do 2[#], 3[#] piers. Hence, only 1[#], 2[#], and 5[#] piers are represented to draw the fragility curve, as shown in Figure 6. (KL in the figure indicates that bond-slip is considered, while KL/0 means that bond-slip is not considered).

Table 4 shows that when bond-slip is not considered, the slope of the fitted equation of piers is smaller than when the bond-slip is considered, that the speed of the seismic demand response accelerates with the increase of earthquake intensity and that the gap between two conditions is also widening. The seismic demand response of 2[#] and 3[#] piers is significantly lower than that of 1[#], 4[#], and 5[#] piers because they are rigid frame and high flexible ones.

Figure 6 illustrates that the exceeding probability of the pier is higher when the bond-slip is considered than when bond-slip is not considered. The influence of considering the bond-slip on the exceeding probability of piers increases with the increase of SA. Under earthquake action of the same SA, the exceeding probability of 5[#] pier (16.55 m) is lower than that of 1[#] pier (9.68 m). When the cross-section of the pier is the same, the higher pier members are more flexible, the permissible displacement is larger, and the ability of the dissipating energy is stronger. 2[#] pier is rigid frame with large stiffness and small deformation; hence, under the earthquake action of the same SA, their exceeding probability is significantly lower.

4.2. Analysis of Bearing Fragility. The displacement of the bearing beyond the permissible displacement is common damage to the spherical bearing. The relative displacements of 37 mm, 104 mm, 136 mm, and 187 mm are taken as

TABLE 2: Details of 10 ground motions.

Number	Name	Event	Time (year)	Site	PGA (g)	Magnitude (M)
1	RSN-138	Tabas, Iran	1978	Boshrooyeh	0.24	7.35
2	RSN-164	Imperial Valley-06	1979	Cerro prieto	0.37	6.53
3	RSN-286	Irpinia, Italy-01	1980	Bisaccia	0.25	6.9
4	RSN-776	Loma prieta	1989	Hollister-south&pine	0.32	6.93
5	RSN-827	Cape mendocino	1992	Fortuna-fortuna blvd	0.22	7.01
6	RSN-880	Landers	1992	Mission creek fault	0.31	7.28
7	RSN-1008	Northridge-01	1994	LA-W 15th st	0.32	6.69
8	RSN-1100	Kobe, Japan	1995	Abeno	0.31	6.9
9	RSN-4840	Chuetsu-oki, Japan	2007	Joetsu kita	0.30	6.8
10	RSN-6886	Darfield, New Zealand	2010	Canterbury aero club	0.35	7

TABLE 3: Damage indexes of five piers.

Pier NO.	Height (m)	Curvature at the first yield	Effective yield curvature	Curvature at compressive strain in concrete is 0.004.	Ultimate curvature	μ_{cy1}	μ_{cy}	μ_{c4}	μ_{cmax}
1 [#] pier	9.68	0.00101	0.00121	0.00218	0.00511	1	1.199	1.482	4.482
2 [#] pier	36.44	0.00104	0.00125	0.00223	0.00516	1	1.203	1.276	4.276
3 [#] pier	35.07	0.00104	0.00125	0.00223	0.00515	1	1.203	1.259	4.259
4 [#] pier	9.17	0.00100	0.00120	0.00216	0.00509	1	1.198	1.485	4.485
5 [#] pier	16.55	0.00103	0.00124	0.00221	0.00516	1	1.200	1.456	4.456

TABLE 4: Probabilistic demand model of five piers.

Pier NO.	Considering bond-slip	Do not consider bond-slip
1 [#] pier	$\ln(\mu) = 1.330 \ln(SA) + 1.642$	$\ln(\mu) = 1.250 \ln(SA) + 1.409$
2 [#] pier	$\ln(\mu) = 1.014 \ln(SA) - 0.133$	$\ln(\mu) = 0.933 \ln(SA) - 0.335$
3 [#] pier	$\ln(\mu) = 1.013 \ln(SA) - 0.061$	$\ln(\mu) = 0.932 \ln(SA) - 0.262$
4 [#] pier	$\ln(\mu) = 1.348 \ln(SA) + 1.765$	$\ln(\mu) = 1.272 \ln(SA) + 1.533$
5 [#] pier	$\ln(\mu) = 1.124 \ln(SA) + 0.662$	$\ln(\mu) = 1.039 \ln(SA) + 0.446$

FIGURE 6: Fragility curves of piers. (a) 1[#] pier. (b) 2[#] pier. (c) 5[#] pier.

damage indices [40]. Consistent with the abovementioned fragility analysis method of piers, the fragility curves of 5 bearings under 4 damage states are obtained, the consistent

damage regularities of 1[#] bearing and 5[#] bearing are consistent, and the damage regularities of 2[#], 3[#], and 4[#] bearings are also consistent. Only the 1[#] bearing and 2[#] bearing

FIGURE 7: Fragility curves of bearings. (a) 1st bearing. (b) 2nd bearing.

fragility curves are listed here, as shown in Figure 7. (KL in the figure indicates that the bond-slip is considered, whereas KL/0 means that the bond-slip is not considered).

Figure 7 shows that with the increase of SA, the influence of the bond-slip on the exceeding probability of bearings increases gradually. Compared with 2nd, 3rd, and 4th bearings, the exceeding probability of 1st bearing under 4 damage states considering the bond-slip is larger, the maximum values are 1, 1, 0.999, and 0.986, respectively, and the rate of increase of the exceeding probability is also faster; 2nd, 3rd, and 4th bearings under minor damage have a large, exceeding probability, which is 0.999, but the exceeding probability under moderate, extensive, and complete damage is significantly lower than that of 1st bearing, and the values are 0.680, 0.420, and 0.159, respectively.

4.3. Analysis of Abutment Fragility. HAZUS takes an abutment displacement of over 50 mm as the basis for determining the moderate damage state [34]. Choi [41] regarded as embankment fill settlement had an influence on active deformation of the abutment and set thresholds of 4 damage states of abutment as maximum displacement of embankment fill; they are 4 mm, 8 mm, 25 mm, and 50 mm. In this study, the reference value of the abutment damage index is 50 mm, and the amplitude modulation coefficients corresponding to the four damage states are 0.5, 1.0, 2.0, and 3.0, respectively. The threshold of abutment displacement is obtained by conversion [42]. Consistent with the above-mentioned fragility analysis method of piers, the fragility curves of the four damage states of two abutments are obtained (Figure 8). (KL in the figure indicates that the

bond-slip is considered, whereas KL/0 means that the bond-slip is not considered).

Figure 8 illustrates that the fragility curves of the left and right abutments are almost the same. Compared with the right abutments, the maximum differences of the exceeding probability of the left abutments under four damage states are 3.12%, 2.08%, 1.08%, and 0.57%, respectively. The difference of fragility damage is small, and the exceeding probability of the abutment under four damage states is relatively large. The maximum values of the left abutments are 1, 1, 0.996, and 0.946, respectively, and the maximum values of the right abutments are 1, 1, 0.995, and 0.944, respectively. The rate of increase of exceeding probability is faster, indicating that the abutment structure is a fragile component, and the fragility increases with the increase of earthquake intensity.

4.4. Analysis of System Fragility. The bridge consists of the main beam, piers, bearings, and abutments, and any component may be damaged under earthquakes. Therefore, the fragility analysis of the bridge should study not only the fragility of a single component but also the fragility of the system. Most of the current studies still focus on components without considering the correlation between them. The fragility of each component is lower than that of the bridge system, and it will overestimate the seismic performance of the bridge. Therefore, on the basis of the correlation among piers, bearings, and abutments, this study explores the system fragility of bridges.

Ditlevsen [43] and Hunter [44] proposed an overall structure exceeding the probability method that considers

FIGURE 8: Fragility curves of abutments. (a) Left abutment. (b) Right abutment.

the correlation between different components, namely, the second-order reliability method, as follows.

$$\rho_{ij} = \frac{\text{Cov}(X_i, X_j)}{\sigma_{X_i} \sigma_{X_j}}, \quad (13)$$

$$P_{f1} + \sum_{i=2}^n \max \left(P_{fi} - \sum_{j=1}^{i-1} P_{fij}, 0 \right) \leq P_{sys} \leq \sum_{i=2}^n P_{fi} - \sum_{i=2}^n \max_{j < i} P_{fij},$$

where P_{ij} is the correlation coefficient of components i th and j th; X_i and X_j denote the seismic requirements of the i th and j th components, respectively; $\text{Cov}(X_i, X_j)$ represents the covariance; σ_{X_i} indicates the standard deviation of X_i ; P_{sys} is the exceeding probability of the system; P_{fi} and P_{fj} are the exceeding probabilities of the i th and j th components, respectively; n is the number of possible destruction components; P_{fij} is the exceeding probability of the i th and j th components simultaneously, as follows:

$$P_{fij} = P_r(F_i \cap F_j), \quad (14)$$

where F_i ($i = 1, 2, 3, \dots, n$) is the failure event of component i th in the structure.

The second-order reliability method is adopted to calculate the system fragility curves without considering the bond-slip and considering the bond-slip, as shown in Figure 9. (KL in the figure indicates that bond-slip is considered, whereas KL/0 means that bond-slip is not considered).

Figure 9 shows the gap between the upper and lower bounds of the fragility of the system under minor, moderate, extensive, and complete damage is larger when the bond-slip is considered than when the bond-slip is not considered. When the bond-slip is considered, pier top displacement increases, bearing relative displacement increases, pier, and bearing exceeding probability increases, and the system fragility increases accordingly. If the bond-slip is neglected,

FIGURE 9: Fragility curves of system. (a) Minor damage. (b) Moderate damage. (c) Extensive damage. (d) Complete damage.

then the seismic performance of the bridge will be overestimated; therefore, the bond-slip should be considered when analyzing the seismic fragility of bridges, especially for those with high piers.

5. Conclusions

In this study, OpenSees is used to establish the finite element model, taking a multispan continuous beam bridge with a high pier as the research object to explore the effect of the bond-slip on the seismic fragility of the bridge. The main conclusions are as follows:

- (1) The probability of the extensive or the complete damage of rigid frame piers under earthquake

actions is less than 1%, and the seismic performance is higher than that of piers with bearings. When the section of piers is the same, the higher pier is more flexible, the permissible displacement is larger, the ability to dissipate seismic energy is stronger, and the exceeding probability is lower. The bearing has a high exceeding probability and is prone to damage, which should be considered in the seismic design of the bridge.

- (2) When the bond-slip is considered, the exceeding probability of piers and bearings significantly increases, and when the reinforcement yields or reaches the ultimate state, the slip of reinforcement is large, and the displacement of pier top and bearings

increases, which is significantly different from that without considering the bond-slip.

- (3) The exceeding probability of piers and bearings increases with the increase of spectral acceleration when considering the bond-slip. The bridge enters the elastic state under the action of small earthquakes, producing tiny slips. The bridge enters the plastic state under the action of strong earthquakes, and the slip amount generated increases, which increases the displacement response of pier top and bearing, thus increases the impact of the seismic fragility of pier and bearing.
- (4) When the bond-slip is considered, the gap between the upper and lower bounds of the fragility of the system under four damage states is greater than when the bond-slip is not considered. If the bond-slip is not considered, then the seismic performance of the bridge will be overestimated, especially the high-pier bridge. It is suggested to improve the bond strength by appropriately reducing the anchorage length, enhance the strength of concrete, and increase the concrete cover thickness to reduce the impact of bond-slip on the seismic performance.

Data Availability

The data used to support the findings of this study are available from the corresponding author upon request.

Conflicts of Interest

The authors declare that there are no conflicts of interest regarding the publication of this paper.

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