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EVALUATION OF PERFORMANCE OF A SKEW BRIDGE IN WENCHUAN 2008 EARTHQUAKE

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ABSTRACT: The 2008 Wenchuan earthquake caused substantial damage with major consequences. Numerous buildings collapsed and thousands of lives were lost. Many bridges were damaged to different degrees, some collapsed and some were left with large residual displacements making them unserviceable after the earthquake. The focus of the study presented in this article was the seismic performance of a skew bridge based on field investigation and analytical studies. The field studies revealed that damage in many bridge substructures was reduced because of the isolation effect of the rubber bearings supporting the superstructure. However, since there was no connection between the bearings and bridge girders, large residual displacements developed once the bearing friction capacity was exceeded. The in-plane rotation effect enlarged the displacements on acute angle and reduced it on obtuse angle, an effect that was consistent with observations in previous strong earthquakes. Finite element method was used to model the response of bridges using the record of Wolong station from Wenchuan earthquake. The estimated residual displacements were reasonable at some locations but substantially lower in other parts.. Parametric studies were conducted by replacing the bearings with different bearing types to determine the effect of the bearings on the response and damage to the bridge. It was found that residual displacement would be lower under certain combination of fixed and free bearings. In addition, the effectiveness of cable restrainers in limiting displacement response of the bridge was studied. The results indicated that even nominal restrainers could reduce the longitudinal residual displacements.

Key Words: Wenchuan earthquake, skew bridge, residual displacement, nonlinear analysis, restrainer design,

INTRODUCTION

A great earthquake with magnitude of 8.0 struck Wenchuan, China on May 12th, 2008. The earthquake epicenter was located at latitude 31.021°N, longitude 103.367°E, with a focal depth of 14 km. Numerous buildings and structures collapsed and thousands of lives were lost. Highway bridges, as an important part of the transportation system with significant impact on the rescue operation after the earthquake, were extensively damaged to different degrees. Some bridges were buried by slope failures or hit by rolling rocks from the mountain (e.g. Shunhe Bridge and Chediguan Bridge). Others collapsed because of design deficiency (e.g. Baihua Bridge and Miaoziping Bridge). There were also many bridges that did not collapse but underwent large residual displacements, which required immediate rehabilitation to restore transportation service.

This paper presents the details of the investigation of a skew bridge that experienced large residual displacements. To understand the response of the bridge during the earthquake, nonlinear response history analyses were carried out subjected to acceleration records from the earthquake. In addition, parametric studies were conducted and measures to improve the seismic performance of the bridge were identified.

FIELD INVESTIGATION OF THE BRIDGE

The bridge that was investigated was the Duxiufeng Bridge on the national highway Route 213 spanning the Minjiang River, built on 2007, six months before the earthquake. This bridge is at about 15.3 km from the epicenter and 5.7 km from the fault. Fig. 1 shows the elevation and plan view of the bridge. The superstructure is supported on five drop cap bents each consisting of two 1.5 m diameter columns. The heights of the columns from the top of the footing to the bottom of the cap are 8.30 m, 11.49 m, 13.01 m, 14.29 m and 14.28 m for bent 1 to 5, respectively. Each column has a 1.8 m diameter pile as its footing and the top of the piles in each bent are linked by a transverse beam with cross section dimensions of 1.3 m by 1.5 m. The lengths of the piles are 43.25 m, 41.30 m, 38.00 m, 27.00 m and 27.30 m for bent 1 to 5, respectively. The height and width of the bent cap are 1.5 m and 1.8 m, respectively. The site class of the bridge is Type II, consisting mostly of gravel and sand. The definition of the site class used is from the Chinese Guidelines (JTG, 2008). The superstructure was constructed using four identical simply supported I-Shape girders in each span and continuous deck, with expansion joints at the middle bent and the abutments. The superstructure width is 8.5 m. The left three spans are slightly curved, but the remaining spans are straight. The skew angle at the abutments is 47°. The bearings at the abutments and bent 3 are Teflon plate rubber bearings with small friction to allow for thermal movement. The bearings on the other bents are rubber bearings supporting the girders directly without any other connections. All the bearings allow for sliding once the friction force between the girders and bearings are exceeded under lateral excitations such as earthquakes.

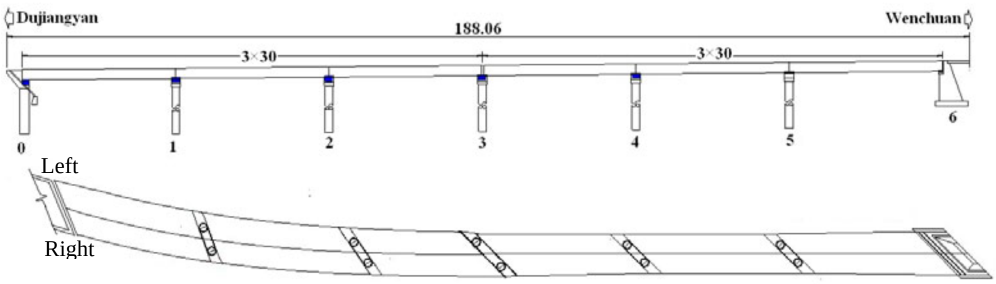


Fig. 1 Elevation and plan view of Duxiufeng Bridge (Dimensions in m)

Fig. 2 shows the overview of Duxiufeng Bridge after Wenchuan earthquake. No damage was noted in the bents and abutments. The main problem caused by the earthquake was large residual displacements at the left abutment (abutment 0 in Fig. 1) and between span 3 and 4 at the top of bent 3 (Fig. 3). The transverse relative displacement (TRD) at the left side of abutment 0 (Fig. 1) was 530 mm, as seen on Fig. 3a. The TRD between span 3 and 4 at the right side was 300 mm, as seen on Fig. 3b.

Fig. 2 Overview of Duxiufeng Bridge after the earthquake

a TRD at left side of abutment 0 *b*. TRD at right side at bent 3

Fig. 3 Residual displacements after the earthquake

These large residual displacements were mainly caused by the in-plane rotation of the skew bridge. By investigating the response of skew bridges in the 1971 San Fernando earthquake, Jennings (Jennings, 1971) pointed out that the skew bridge tend to rotate in a horizontal plane in the direction of increasing the skewness of the bridge (Fig. 4). Shear keys limiting superstructure movement in the transverse direction of the Duxiufeng Bridge had been installed. However, these shear keys failed during the earthquake (Fig. 5). After failure of the shear keys, the superstructure rotated in plane with little transverse restraint and experienced large residual displacements. Both spans 1 and 3 were unseated because of the large transverse displacements. Sliding also occurred on other bearings, leaving approximately 30 mm to 150 mm residual displacements. Due to the large residual displacements, the bridge was taken out of service after the earthquake. The bearings and shear keys were replaced, the superstructure was moved to the original position, and the bridge was opened to traffic approximately 100 days after the earthquake.

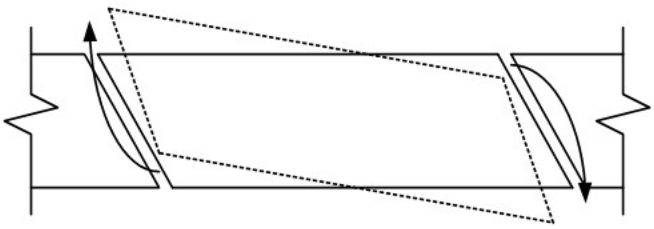


Fig. 4 In-plane rotation effect of skew bridge

a Abutment 0 *b.* Bent 1

Fig. 5 Failure of shear keys of Duxiufeng Bridge

MODELING OF THE BRIDGE

Modeling of the superstructure

To better understand the response of the bridge during the earthquake, a nonlinear finite element model was developed using computer program SAP 2000. From the results of comparative study, Meng (Meng, et al. 2000) concluded that the effect of superstructure flexibility is important and should not be ignored in dynamic analysis since the use of the rigid deck or stick model for the dynamic analysis of skew bridges with large skew angles leads to inaccurate axial forces in the columns during the earthquake. Therefore, the superstructure was modeled using a grid system representing the beam flexibility. In each span four elastic beams were used to model the I-Shape girders. The stiffness of the deck was included by enlarging the flange of the girders. Each elastic beam was divided into six elements. Transverse elements were assigned among the nodes in the same line and assumed to be rigid.

Modeling of the bents

Plastic hinges were assigned to the top and bottom of the columns. The plastic hinge sections consisted of three types of fibers: confined concrete, unconfined concrete, and longitudinal bars. The Mander (Mander, et al. 1988) model was used to model the unconfined and confined concrete. The hinge lengths were assumed to be (JTG, 2008):

$$L_p = \min\left(0.08H + 0.022 f_y d_s \geq 0.044 f_y d_s,\ \frac{2}{3}b\right) \quad (1)$$

where H is the distance from the top and bottom to the point where the moment is zero; b is the length of the smaller side of rectangular section or the diameter of circular section; f_y (MPa) is the specified steel bar yield strength and d_s is the bar diameter. The concrete grade used in the bents of Duxiufeng Bridge is C30 with a compressive strength of 20.1 MPa (JTG, 2004). The steel grades of longitudinal and transverse bars are HRB335 and R235, respectively. The yield strengths are 335MPa and 235 MPa, respectively (JTG, 2004). The locations of the hinges were defined at the middle of the hinge length. Bent caps were modeled as elastic elements.

Modeling of the rubber bearings

As stated before, the links between the girders and the bents were rubber bearings without any other connections. Therefore, the transfer of lateral forces takes place only through friction. No tension force can be transmitted through the bearings. Friction Isolator link in SAP2000 was used to simulate this mechanical behavior. Friction factors can be assigned in the lateral direction to calculate lateral forces based on the normal force on the bearing. The normal force is always nonlinear and is given by:

$$P = \begin{cases} kd & \text{if } d < 0 \\ 0 & \text{otherwise} \end{cases} \quad (2)$$

where k is the compressive stiffness, d is the normal deformation of the bearing. In order to generate nonlinear shear force in the element, the stiffness k must be positive, and hence normal force on the bearing P must be negative (compressive). The sliding forces of the bearings could be different because of the vertical input and the uneven distribution of the normal forces in the bearings. Friction factors were set the same in lateral direction for the rubber bearings. The radius of the sliding face was set zero because the contact face between the girders and the bearings is flat. The friction coefficient between rubber bearings and concrete girders was taken as 0.3 from the experimental results of Huang (Huang, et al. 2010). The friction coefficient between Teflon plate rubber bearings (at the abutments and bent 3) and concrete girders was taken as 0.02.

Modeling of the expansions joints

Gap elements were used to model the pounding effect between the superstructure segments at bent 3 and between the superstructure and the abutments. The initial gap was set 80 mm, which is the standard expansion joint gap thickness in bridges. The girder axial stiffness was used as the gap element stiffness in compression.

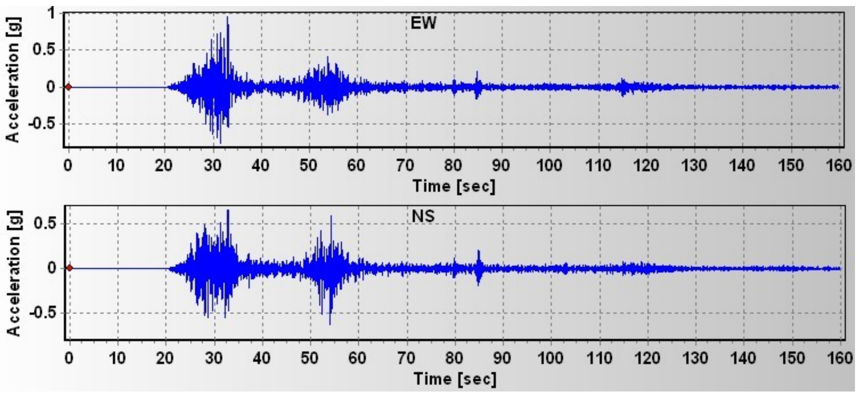
Modeling of the bents foundation and abutments

BCAD_PILE program developed at Tongji University was used to calculate the stiffness of the pile foundation of the bents. With the arrangement of piles and soil information from the bridge site, a 6×6 stiffness matrix was derived. The springs with derived stiffness matrixes were assigned to the bottom of the columns.

According to the field investigation, the abutments did not experience any damage during the earthquake. The abutment stiffness is much larger than the bents. Therefore, in order to simplify the analysis, the abutments were modeled as rigid elements.

Input acceleration records

The measured acceleration records from Wolong station was used to carry out nonlinear response analysis of the bridge (Fig. 6). The site is approximately 30 km away from the bridge and has the largest PGA among all the records from Wenchuan earthquake (Li, et al. 2008). PGAs recorded in EW, NS and UD directions are 0.957g, 0.653g and 0.948g, respectively. 0.1HZ and 25HZ bandpass was applied to filter the original record. The part of the record used in the analysis was from 20 seconds into the record to 70 seconds. All three records were applied simultaneously. The corresponding elastic response spectra are shown on Fig. 7.



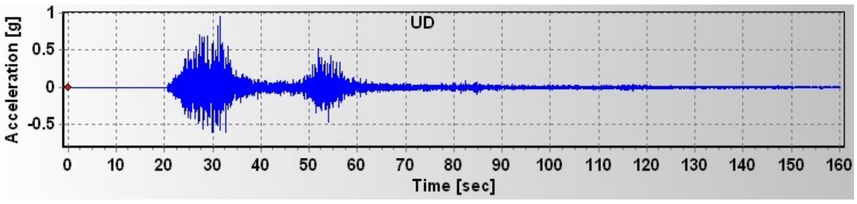


Fig. 6 Records of different direction from Wolong station

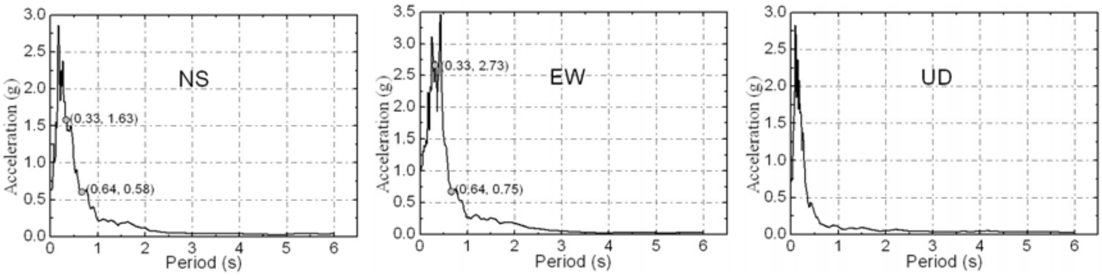


Fig. 7 Elastic response spectrum of Wolong record

RESPONSE ANALYSIS

Modal analysis

The dynamic response of a structure is generally based on its natural modes of vibration. Therefore, the nature vibration periods and mode shapes for the first seven modes of the bridge were calculated (Table 1). The results indicate that the fundamental mode of the bridge is in-plane rotation of the superstructure about the vertical axis, which is mainly due to the large skewness of the bridge and unsymmetrical column heights. Under excitations with predominant periods close to in-plane rotation periods, the rotation response could be significant and cause relatively large displacements compared to non skew bridges. Since the height of the columns of the right three bents is larger than that of the left two bents, the natural periods of the bridge segment to the right of bent 3 are longer than those of the left three spans. Out-of-plane vibration of bent 3 is similar to a cantilever beam since the Teflon plate rubber bearings at the top of the bent only have small friction force and provide little constraint to the bent.

Table 1 First seven natural modes of the bridge

Mode No.	Periods (s)	Frequency (Hz)	Mode Description
1	2.462	0.406	In-plane rotation (right three spans)
2	2.087	0.479	In-plane rotation (left three spans)
3	1.754	0.570	Longitudinal vibration (right three spans)
4	1.430	0.699	Longitudinal vibration (left three spans)
5	1.329	0.752	Transverse vibration (right three spans)
6	1.108	0.902	Transverse vibration (left three spans)
7	0.645	1.551	Out-of-plane vibration of bent 3

Results of nonlinear analysis

Nonlinear response history analysis was carried out to study the seismic response of the bridge with finite element method. The longitudinal direction of the bridge is NW 10° and the global coordinate system of the model is as follows: X axis is along the longitudinal direction of the bridge; Y axis is along the transverse direction of the bridge; Z axis is along the vertical direction of the bridge.

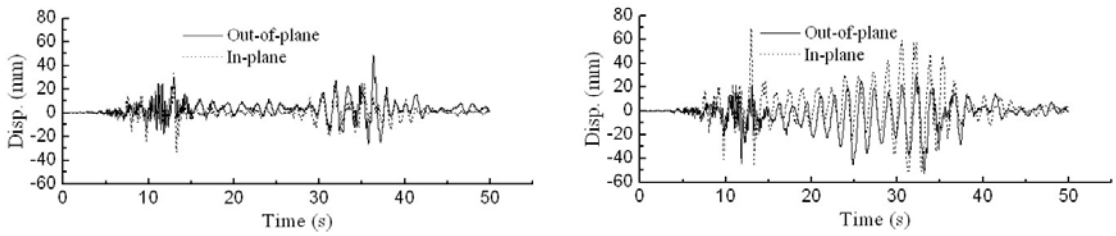
Therefore, NS record with an angle of 10° to X axis was taken as longitudinal input and EW record with an angle of 10° to Y axis was taken as transverse input. The vertical motion was also applied.

Nonlinear static pushover analysis was carried out first to determine the performance of the bents. The results are listed on Table 2. The results show that the yield displacements increased as the bent height increased, while the lateral load capacity dropped. The out-of-plane ductility and displacement capacities are larger than those of the in-plane direction. This could mean that the bents are more vulnerable for loading in the transverse direction of the bridges.

Table 2 Pushover results of the bents

Bent No.	Yield Displacement (mm)		Ductility Capacities		Max Shear Force (kN)	
	Out-of-plane	In-plane	Out-of-plane	In-plane	Out-of-plane	In-plane
1	40	10	6.90	2.67	1022	1025
2	60	15	7.00	3.05	765	767
3	80	20	7.48	3.17	653	654
4	100	25	5.88	2.79	619	619
5	100	25	6.40	2.88	608	608

The displacement histories of bent 2 and 4 for both directions are illustrated on Fig. 8. The responses indicate that the displacement of bent 2 (a relatively short bent) in the in-plane direction was smaller than the out-of-plane displacement. However, this trend was reversed in the taller bents (bent 4). Since the yield displacements in the out-of-plane and in-plane directions of bent 2 and 4 are 60 mm and 15 mm, 100 mm and 25 mm, respectively, it can be seen that in the out-of-plane direction the bents remained elastic, but they yielded in the in-plane direction with displacement ductility demands of 2.22 and 2.78 for bent 2 and 4, respectively. The ductility demands were smaller than the corresponding calculated capacity for both bent 2 and 4. Also note in Fig. 8 that there was no residual displacement in neither direction.



a Displacement histories for bent 2 *b* Displacement histories for bent 4

Fig. 8 Nonlinear displacement response of the bents

Fig. 9 compares the displacement ductility demands and calculated ductility capacities for all the bents, where μ_D is the ductility demand and μ_C is the calculated ductility capacity. The bents remain elastic if $\mu \leq 1$. It can be seen that all the bents remained elastic in the out-of-plane direction, while in the in-plane direction all the bents yielded but the ductility demands were all smaller than the corresponding calculated ductility capacities. It is interesting to note that even though the friction coefficient of the bearings on bent 3 was small and little friction force could be transferred from the superstructure to the bent through the bearing, the displacement ductility demands in both directions of bent 3 were almost the same as other bents. This result can be explained from the elastic response spectrum (Fig. 7). As stated above, there is almost no constraint at the top of bent 3 and it can vibrate as a cantilever system. The first two natural modes of bent 3 are out-of-plane and in-plane modes with the periods of 0.64s and 0.33s, respectively. It can be seen from the elastic response spectrum that the corresponding accelerations of these periods are relative large. The other bents are constrained to the superstructure through friction of the rubber bearings and the natural vibration periods are 2s or more (in-plane rotation period of the bridge, Table 1). The spectral accelerations are much smaller at these periods. Therefore, displacements generated with large accelerations and small mass (bent 3) were approximately the same as those generated with small accelerations and large masses (other bents).

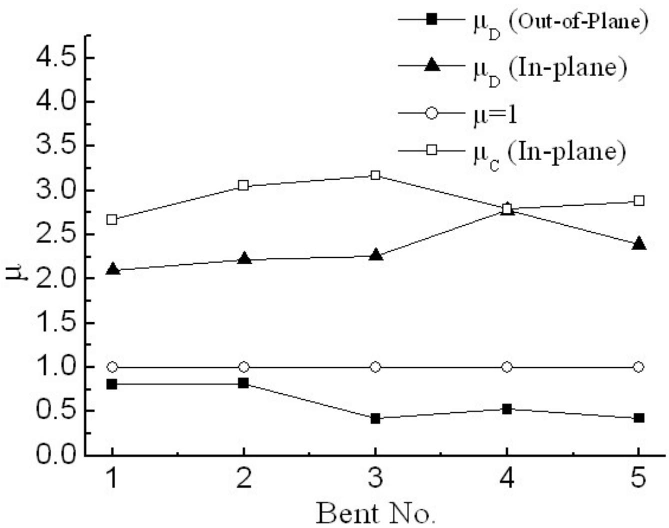


Fig. 9 Calculated ductility demands

The axial displacements of most Friction Isolator links had positive values, indicating that opening between the girders and the bearings occurred during the earthquake. Strong vertical input and uneven distribution of axial force among the bearings caused by dynamic vibration led to unloading of the bearings and opening at the interfaces. There was no friction force between the bearings and the girders when opening occurred, which means the girders lost lateral constraint leading to to large sliding displacements. All the bearings had residual displacements in both directions. The maximum residual displacements of the bearings in the longitudinal and transverse directions were 121 mm and 229 mm, respectively. The calculated residual relative displacements of the superstructure are listed in Table 3. There are some differences between the calculated and field results. According to field investigation, the transverse residual relative displacement at abutment 0 on left side was 530 mm. However, it was only 45.5 mm from the nonlinear analysis. The difference at bent 3 was relatively small. These differences may be caused by the site effect. Although acceleration records from Wolong station are the nearest records to the bridge, local site effect may change the excitations of the bridge. The riverbed can alter the amplitude and frequency content of the seismic waves. For a more accurate estimate of the response, measured records in the actual site are necessary. Another possible explanation for discrepancies between the calculated and measured residual displacements is the fact that calculated residual displacements are generally very sensitive to small variations in modeling assumptions and the material properties.

Table 3 Calculated residual relative displacement of superstructure

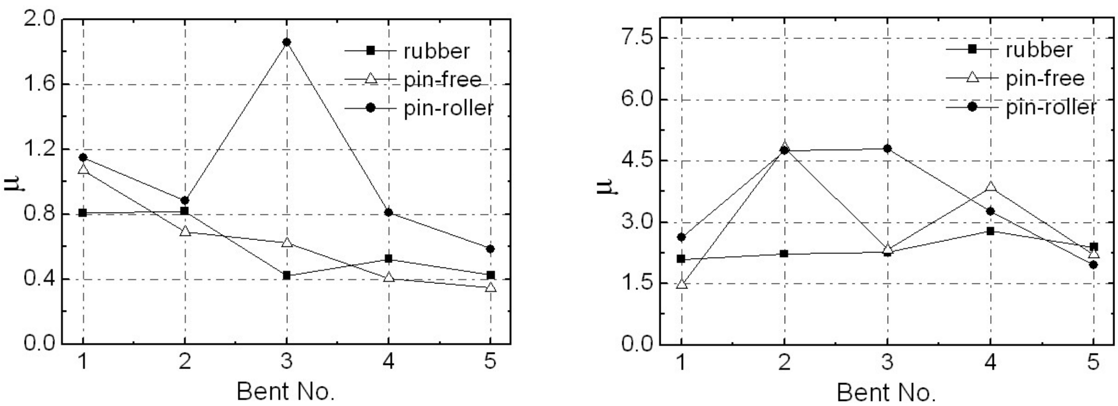
Position		Longitudinal (mm)	Transverse (mm)
Abutment 0	Right side	90.3	19.4
	Left side	116.1	45.5
Bent 3	Right side	58.0	274.0
	Left side	82.7	250.2
Abutment 6	Right side	31.6	62.2
	Left side	29.4	60.2

The maximum axial deformation of the gap elements was 84 mm and the corresponding force was 1446 kN. This pounding occurred at left side of bent 3, due to the in-plane rotation

EFFECTS OF DIFFERENT BEARING TYPES

Neither the field investigation nor the nonlinear response history results indicated strength loss in the bridge under the Wenchuan earthquake. However, the large residual displacements led to the closure of the bridge. An analytical study was undertaken to determine if, by changing the bearings, residual displacements could be reduced without compromising the load capacity of the bridge. Two different types of bearings were studied. The first had pin bearings at bent 2 and 4, and roller bearings at other bents. The roller bearings allowed for unrestrained movement of the superstructure in the longitudinal direction but prevented movement in the transverse direction. The second type also had pin bearings at bent 2 and 4, but allowed for unrestrained longitudinal and transverse movement of the superstructure at other bents. To facilitate discussions in this article, the bridge model with the first bearing combination is labeled “pin-roller bearing” and the one with the second type is labeled “pin-free bearing.” The model with the actual bearing used in the Duxiufeng Bridge is referred to as “rubber bearing.”

Fig. 10 shows the displacement ductility demands of the bents for different bearing types. The pin-roller bearing experienced the largest bent displacement ductility demands in the out-of-plane direction and was the only model in which the columns in bent 1 and 3 yielded. None of the piers in the other two models yielded in the longitudinal direction. The in-plane rotation is restrained because the transverse movement is restrained. It appears that the transverse restraint led to large bent forces in both directions, which increased the displacement ductility demands for the bents. The differences of displacement ductility demands in the out-of-plane direction of pin-free and rubber bearing were relatively small. Pin-roller bearing also had the largest displacement ductility demands in the in-plane direction of all the bents, except for bent 4 and 5. In contrast, the results of pin-free bearing were slightly larger than that of rubber bearing at pinned bents (bent 2 and 4). Therefore, the pin-free and rubber bearing models are preferred in controlling damage in the bents under the Wolong station earthquake records.



a Out-of-plane displacement ductility demands *b* In-plane displacement ductility demands

Fig. 10 Displacement ductility demands of bridge models with different bearing types

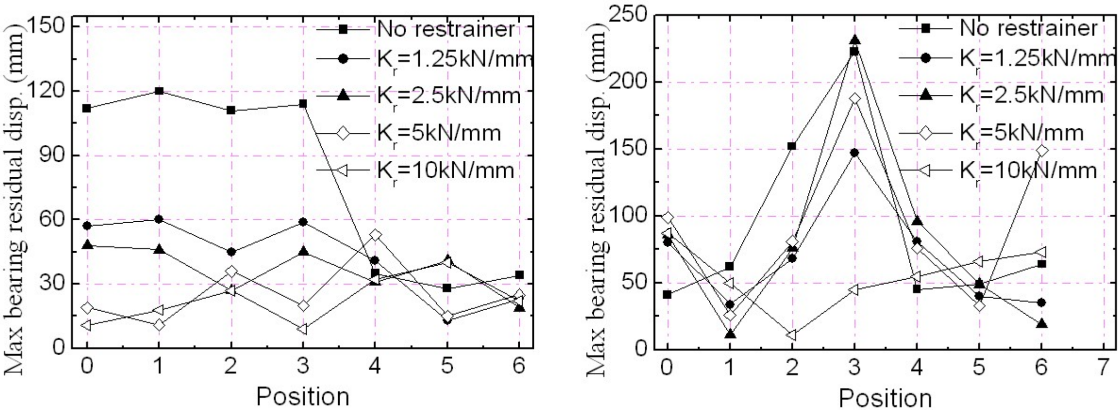
The maximum relative displacement between girders and bents of different bearing types are listed in Table 4. The results indicate that the bridge with rubber bearing had the largest relative displacement in both the longitudinal and transverse directions. The results for pin-free bearing model were approximately one half of the results for rubber bearing. The longitudinal residual displacement in the pin-roller bearing model was slightly smaller than that with rubber bearing, and because of constraint against transverse movement, the transverse relative displacement was zero. The results indicate significant reduction in both the longitudinal and transverse residual displacements in the pin-free model compared to bearing types of the actual bridge labeled as “Rubber” in the table. Therefore, pin-free bearing is recommended for the Duxiufeng Bridge to control permanent displacements.

Table 4 Max relative displacement between Girders and Bents

Bearing type	Longitudinal (mm)	Transverse (mm)
Rubber	135	254
Pin-free	68	118
Pin-roller	121	0

EFFECTS OF RESTRAINERS

The effect of cable restrainers on reducing residual displacements of the Duxiufeng Bridge was also studied. Cable restrainers were assumed to be installed at the expansion joints at the abutments and bent 3. Two restrainers were symmetrically arranged at each abutment expansion joints attaching the girders to the abutments. Another four restrainers were symmetrically arranged at the middle expansion joint attaching span 3 and 4 to bent 3. The restrainer stiffness was assumed to be the same at different locations. The minimum restrainer stiffness was calculated 2.5 kN/mm according to the minimum restrainer stiffness requirement recommended by Saiidi et al. (Saiidi, 2001). Different stiffness values of 1.25kN/mm, 2.5 kN/mm, 5 kN/mm and 10 kN/mm were assumed to carry out a parametric study. The initial slack of the restrainers was taken as zero to account for the extreme low temperature and the most critical condition for restrainer loading.



a Longitudinal *b* Transverse

Fig. 11 Comparison of max bearing residual displacement

The comparison of the maximum bearing residual displacement with different restrainers is shown in Fig. 11, where positions 0 and 6 represent abutment 0 and 6 and positions 1 to 5 represent bent 1 to 5. It can be seen that when there was no restrainer, the max bearing residual displacements in the longitudinal direction of position 0 to 3 were significant. Even a relatively small restrainer stiffness can reduce these residual displacements by approximately 50%. Increasing restrainer stiffness can further reduce the residual displacements, but the rate of reduction becomes small when the restrainer stiffness exceeds 5 kN/mm. Fig. 11b shows that restrainer had little effect on limiting the max bearing residual displacement in the transverse direction when the stiffness was smaller than10 kN/mm. This was expected because the restrainers acted mainly in the longitudinal direction. Meanwhile, lateral biaxial input and rotation of the superstructure led to coupled longitudinal and transverse displacements of the superstructure. Therefore, restrainers partly affected the transverse displacement.

If the stiffness is sufficiently large, the effect on limiting the transverse displacement becomes significant. The maximum elongation of the restrainers were 94 mm, 72 mm, 60 mm and 52 mm for the stiffness of 1.25 kN/mm, 2.5 kN/mm, 5 kN/mm and 10 kN/mm, respectively. These are all smaller than the yield displacement of the most commonly used restrainers (Saiidi, et al. 2001), which is 107 mm, indicating the restrainers can remain elastic during the earthquake.

CONCLUSIONS

Based on the field investigation and nonlinear analysis of the Duxiufeng Bridge after Wenchuan Earthquake, following conclusions are drawn:

1. The Wenchuan Earthquake did not cause severe damage to Duxiufeng Bridge. The main issue that led to the closure of the bridge was the large residual displacement of the superstructure and the unseating of the bearings.
2. Large sliding displacements occurred between the rubber bearings and the girders because there were no stoppers.
3. Nonlinear dynamic analysis of the bridge based on a record that was obtained 30 km away from the bridge site provided an approximate estimate of residual displacements.
4. Parametric analytical studies revealed that by allowing unrestrained movement at most of the bearing, the residual superstructure displacements can be reduced without imposing additional demand on the bents.
5. Parametric studies on the effect of restrainers showed that even nominal restrainers are effective in reducing the residual displacements in the bearings. Both the longitudinal residual displacements can be controlled with appropriate size of restrainers.

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