

Damage of Bridges in 2008 Wenchuan, China, Earthquake

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This is a reconnaissance report on the damage to bridges during the 2008 Wenchuan, China, earthquake. Site investigation was conducted by the authors on August 10-14, 2008. Presented is a detailed discussion of the damage to twelve bridges as well as possible damage mechanisms. Characteristics of two near-field ground accelerations and Chinese seismic bridge design practices are also presented. An investigation of the damage finds insufficient intensity of seismic design force, inadequate structural detailing for enhancing the ductility capacity and an absence of unseating prevention devices.

Key Words : *Seismic damage, bridges, Wenchuan earthquake, failure mechanism, seismic effect, seismic design*

1. INTRODUCTION

The Wenchuan earthquake occurred in Sichuan Province, China, at 2:28 p.m. (local time) on May 12, 2008. Magnitude was 8.0 by CEA and 7.9 by USGS. The earthquake killed 69,227 people and 17,923 people were still missing by October 8, 2008. In Yinxu Town close to the hypocenter, almost all buildings were destroyed and about 7,700 people, nearly 80% of the residents, were dead or missing.

Transportation facilities, including nearly 1,600 bridges along main roads, suffered extensive damage, resulting in navigation obstacles for evacuation and rescue teams in regional and local transportation systems. Analyzing the damage caused by the earthquake provides important information about how bridges in the fault regions behaved under a M8 earthquake that had a very low probability of occurrence.

The authors conducted a field-damage investigation of bridges in the regions of Dujiangyan City, Wenchuan County, and Pengzhou City and Mianzhu City, Qinchuan County, on August 10-14, 2008. Several individual surveys were additionally conducted. Based on the field investigation, damage of twelve typical bridges and their damage mechanisms are presented here. It should be noted that since the field investigation

was conducted without prior information on design drawings and analysis, it is highly possible that the authors made incorrect interpretations of the failure mechanism. Moreover, because a number of regions could not be accessed due to suspension of transportation, there were bridges which were hardly investigated.

2. OUTLINE OF WENCHUAN EARTHQUAKE

The Wenchuan earthquake was induced by rupture of the Longmen Shan Fault as shown in Fig. 1. Large scale thrusting of the Tibetan Plateau towards the east of the rigid block of Sichuan Basin resulted in the NE-SE dip-slip reverse fault [Densmore et al, 2008; Lekkas, 2008]. The fault zone extended from Yingxiu Town, Wenchuan County, to Mianzhu City, Beichuan County, and to Qingchuan County. The epicenter was located at Yingxiu Town. Extensive damage occurred in nearly 300 km long and 50 km wide regions along the fault [Li and Zhao, 2008]. There were precipitous mountainous regions with a number of steep streams. Consequently, shallow and deep slope failures occurred extensively. The damaged regions had very harsh natural conditions.

Fig. 2 shows two typical strong motion accelerations recorded at Wenchuan Wolong and Shenfeng Bajiao stations during the earthquake [Li et al, 2008]. Locations of the recording stations are shown in Fig. 1. The Wolong station was located in Wenchuan, 19 km from the epicenter, while the Bajiao station was located in Shenfeng, 67 km from the epicenter and 20 km from the nearest fault. Fig. 3 shows the corresponding response acceleration spectra with 0.05 damping ratio.

The Wolong station recorded more than a 60 s duration. There were two major acceleration groups which resulted from the fault rupture process. PGA was 6.53 m/s^2 , 9.58 m/s^2 , and 9.48 m/s^2 in the NS, EW and UD components, respectively, which were the highest among the accelerations recorded during this earthquake. Response acceleration was nearly $30\text{--}35 \text{ m/s}^2$ at $0.2\text{--}0.4 \text{ s}$ in the NS and EW components and was nearly 30 m/s^2 at 0.2 s in the UD component. In spite of the high response accelerations at short periods, they sharply decrease as the period increases. For example, the response accelerations are less than 5 m/s^2 at 1 s in three components. Although precise soil conditions at recording stations are unknown, it is anticipated that they were recorded at rock sites. Peak velocity was 0.515 m/s , 0.417 m/s and 0.304 m/s in NS, EW and UD directions, respectively. The Bajiao record also had a long duration and was largest during short periods. However, it did not have two major acceleration groups.

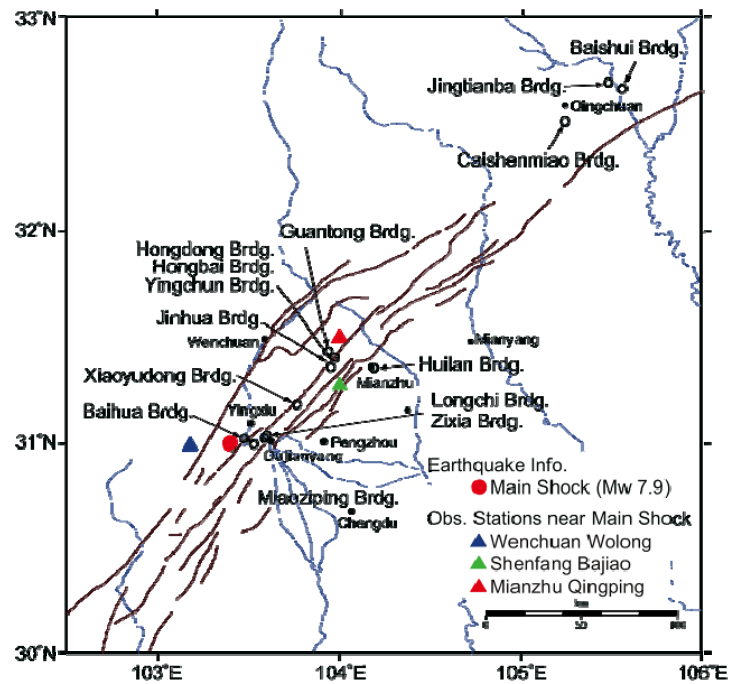


Fig. 1 Longmen Shan Fault and location of bridges shown in this paper

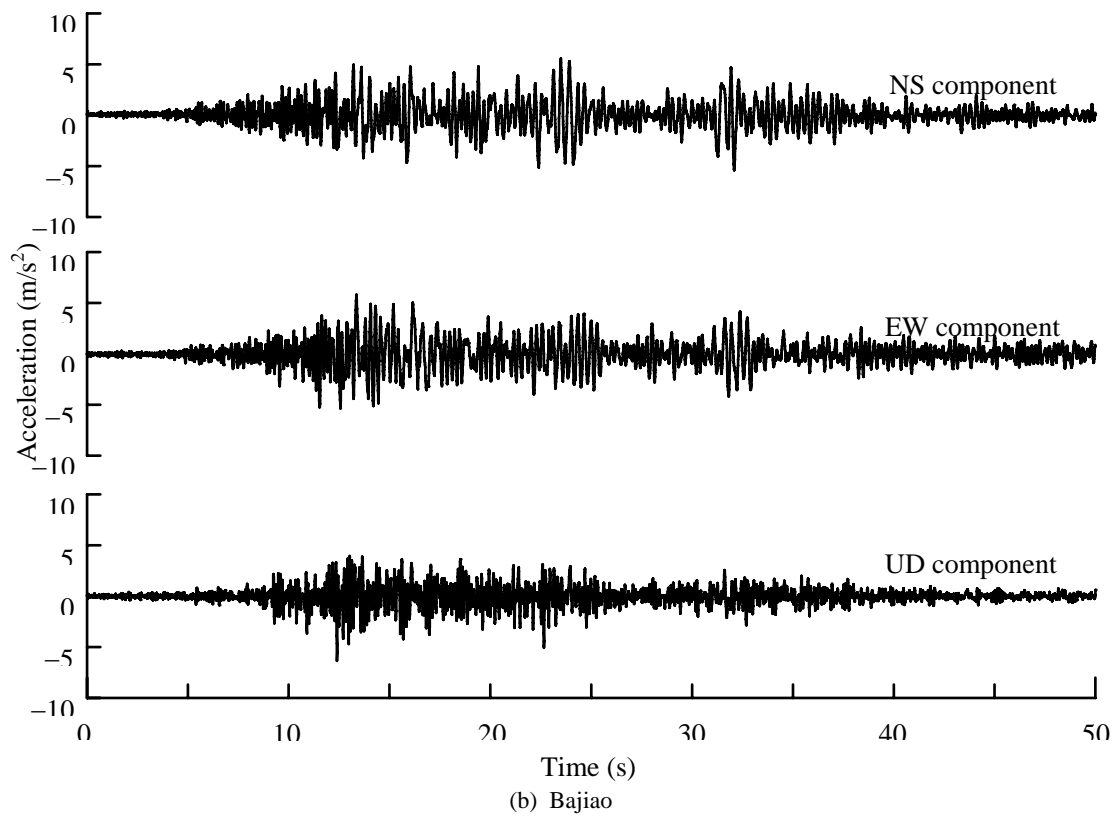
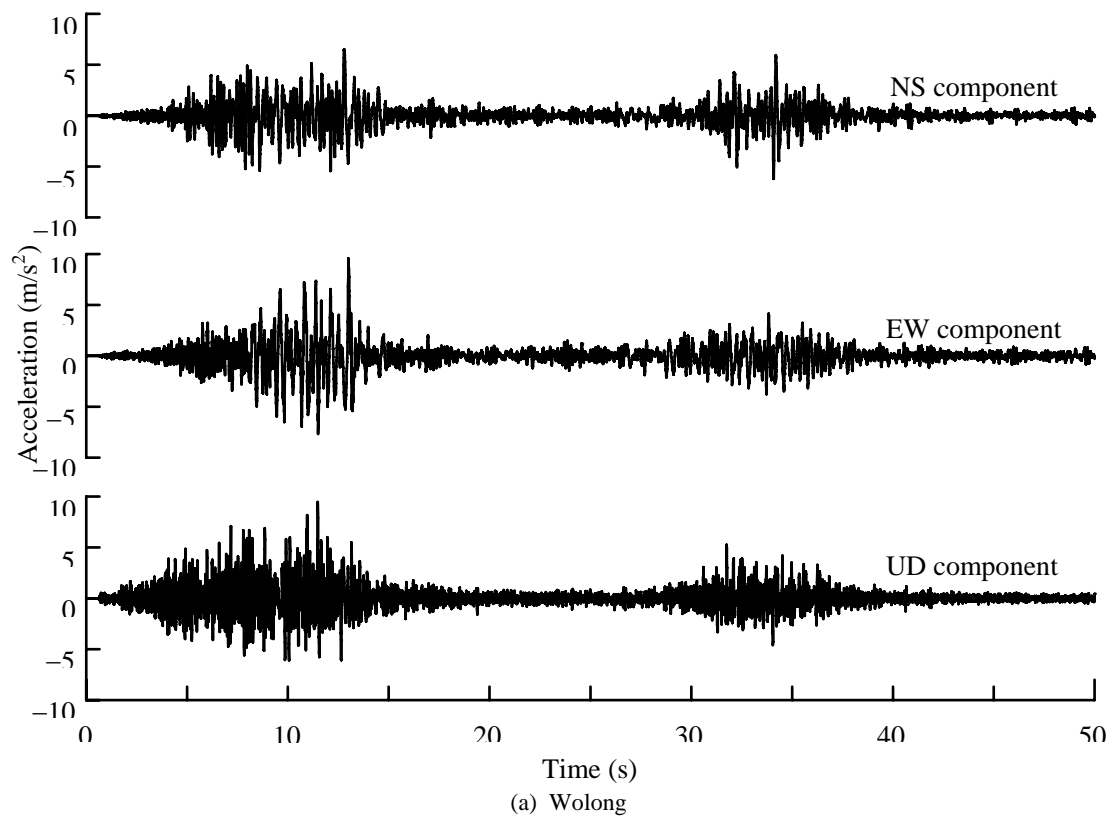


Fig. 2 Strong motion records

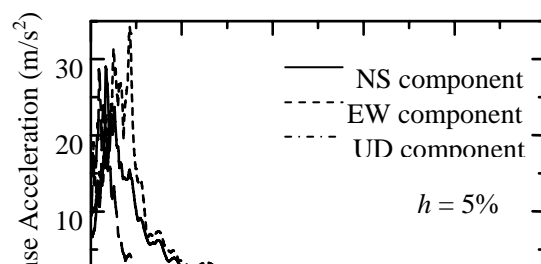


Fig. 3 Response acceleration spectra

3. SEISMIC DESIGN OF BRIDGES IN CHINA

Seismic design code (JTJ004-89) issued by the Chinese Ministry of Transportation in 1989 [Chinese Ministry of Transport, 1989] was used in the design of expressways and road bridges in China. This code was applied to reinforced/pre-stressed concrete bridges and stone masonry/reinforced concrete arch bridges with spans no longer than 150 m. The seismic force E_{ihp} is determined for girder bridges with limited contribution of piers to the seismic lateral force as

$$E_{ihp} = C_i C_z K_h \beta_1 \gamma_1 X_{1i} G_i \quad (1)$$

where, K_h : lateral force coefficient (refer to Table 1), C_i : importance factor (refer to Table 2), C_z : general influence factor (refer to Table 3), β_1 : dynamic amplification factor, γ_1 : mode participation factor for the first mode, X_{1i} : relative lateral displacement at node i , and G_i : weight at node i .

As shown in Table 1, the lateral force coefficient K_h depends on Chinese seismic intensity. It doubles by a unit increase of the seismic intensity. K_h was 0.1 in the damaged regions because the seismic intensity was 7. The dynamic amplification factor β_1 is determined depending on the fundamental natural period of a bridge T and soil condition as

$$\beta = \begin{cases} 0.3 \leq 2.25 \times (0.2/T) \leq 2.25 & \text{Group I} \\ 0.3 \leq 2.25 \times (0.3/T)^{0.98} \leq 2.25 & \text{Group II} \\ 0.3 \leq 2.25 \times (0.45/T)^{0.95} \leq 2.25 & \text{Group III} \\ 0.3 \leq 2.25 \times (0.7/T)^{0.9} \leq 2.25 & \text{Group IV} \end{cases} \quad (2)$$

Based on Eqs. (1) and (2), the elastic seismic coefficient $C_i K_h \beta_1$ is evaluated assuming two importance factors ($C_i = 1.0$ and 1.7), as shown in Fig. 4. The elastic seismic coefficient for a seismic intensity of 9 is also shown here for comparison. In regions with a seismic intensity of 7, the peak elastic seismic coefficient

$C_i K_h \beta_1$ becomes 0.23 and 0.39 for C_i equal to 1.0 and 1.7, respectively. Because the general influence factor C_z is in the range of 0.2-0.35, depending on the bridge type and pier height (refer to Table 3), the peak inelastic seismic coefficient $C_i C_z K_h \beta_1$ can be in the range of 0.045-0.079 and 0.077-0.14 for C_i equal to 1.0 and 1.7, respectively. Considering the extensive ground motions recorded during the earthquake, the above seismic coefficient was insufficient to prevent damage. If the seismic intensity was set at 9, the peak inelastic seismic coefficient $C_i C_z K_h \beta_1$ has a value in the range of 0.18-0.32 and 0.34-0.54 for C_i equal to 1.0 and 1.7, respectively. This level of inelastic seismic coefficient is likely the least required for design of bridges under the near-field ground motions developed during the earthquake.

Table 1 Lateral force coefficient K_h

Seismic intensity	7	8	9
K_h	0.1	0.2	0.4

Table 2 Importance factor C_i

Highway grade and structures	Importance factor
Aseismic key projects on expressways and first class roads	1.7
<ul style="list-style-type: none"> General projects on expressways and first class road Aseismic key projects on second class road Bearings at girder ends of bridges on second or third class road 	1.3
<ul style="list-style-type: none"> General projects on second class road Aseismic key projects on third class road Bearings at girder ends of bridges on fourth class road 	1.0
<ul style="list-style-type: none"> General projects on third class road Aseismic key projects on fourth class road 	0.6

Note: (1) Importance factor of 1.5 can be used in the expressways and aseismic key projects at the IX seismicity areas.

(2) Aseismic key projects include super large bridges, large bridges and tunnels as well as projects of subgrades, middle bridges and retaining walls which are difficult to be repair. General projects are non-key projects of subgrades, middle-small bridges and retaining walls.

Table 3 General influence factor C_z

Type of bridges, piers and abutments		Pier height H (m)		
		H<10	10≤H<20	20≤H<30
Girder bridge s	Flexible piers	0.3	0.33	0.35
	Stiff piers	0.20	0.25	0.30
	Pile foundations	0.25	0.30	0.35
	Abutments	0.35		
Arch bridges		0.35		

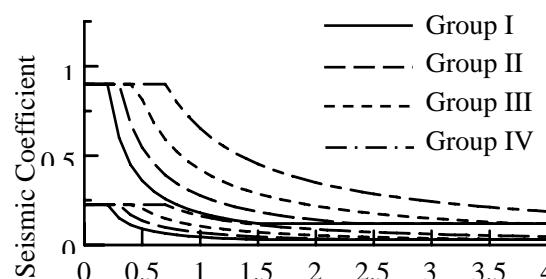


Fig. 4 Seismic coefficient (higher four lines show $C_i K_h \beta_1$ for seismic intensity 9 regions while lower four lines show $C_i K_h \beta_1$ for seismic intensity 7 regions)

4. TYPICAL DAMAGE OF BRIDGES

We investigated damage of bridges in Dujiangyan City, Wenchuan County, and Pengzhou City and Mianzhu City in Qingchuan County. Only typical damage is presented here.

Miaoziping Bridge

Miaoziping Bridge is located where Duwen Highway (connecting Chengdu and Wenchuan) crosses the Zipingpu dam reservoir and Min River. In total, the structure is 1,440 m long and consists of a three-span continuous prestressed box girder bridge with prestressed concrete girder approach bridges of two, four, and five spans as shown in Photo 1. The main structure is a 450 m long three-span continuous pre-stressed concrete frame bridge with a main span of 220 m. A pre-stressed concrete girder bridge was constructed by connecting 50 m long, simply supported, PC T-girder bridges with concrete slabs. PC girder bridges were supported by elastomeric bearings. All spans were supported by nearly 100 m tall reinforced concrete piers resting on pile foundations in the dam reservoir. Upper and lower bound bridges were separated.

When the earthquake occurred, the deck connections of T-girder bridges were almost completed, but expansion joints were not yet set. It was reported that the water level was about 85 m above the bottom of the pier when the earthquake occurred, but it was lowered to about 40 m after the earthquake in order to survey possible damage to the Zipingpu dam.

One of the two end-spans of a five-span continuous T-girder bridge collapsed as shown in Photo 2. It was reported that the distance between two adjacent piers which supported the fallen span was extended by about 0.69 m at the top based on measurements taken after the earthquake. Any damage in the piers above the water level was not identified; however, it was reported that piers suffered several small cracks at their base. Because the piers were 100 m tall, only 1% drift can result in 1 m lateral displacement at the deck level. It was likely that foundation flexibility further amplified the lateral displacement of the bridge. The fact that residual

extension was nearly 0.69 m between the two piers implies that peak relative displacement between the two piers was possibly over a meter.

Photo 3 shows the bearings at the side where the span started to fall. The remaining part of the lower bearings and extensive damage to the concrete supports clearly show that the fall of an end span of a five-span continuous T-girder bridge was initiated here. Although reinforced concrete side blocks were installed at both sides of the deck, they could not fully restrict the deck displacement in the transverse direction. Unseating prevention devices were not provided because they were not required in bridges located in regions of seismic intensity 7, according to the code. If unseating prevention devices were installed at this span, collapse could have possibly been prevented.

On the other hand, Photo 4 shows the concrete slab on the T-girder span next to the fallen span. When the adjacent span started to fall at the other side, the concrete slab was subjected to tension and was torn off.

There were many other pieces of evidence that show that deck displacement was extremely large during the earthquake. For example, Photo 5 shows a 300 mm offset between two adjacent T-girder bridges in the transverse direction. Obviously failure of side blocks at both sides of the bridge allowed residual drift to develop in the transverse direction. Photo 6 shows failure of one of the side blocks. Almost all side blocks suffered damage. Photo 7 shows that collisions occurred between adjacent girder bridges; the center curb crashed due to the collision of adjacent girder bridges. Vertical offset was also developed here because almost all elastomeric bearings moved out from their position due to the excessive relative bridge response displacement as shown in Photo 8. Photo 9 shows one of the damaged elastomeric bearings.

Photo 10 shows the repair of a movable bearing at one of the side spans of the main frame bridge. Steel bearings with rubber pads failed as shown in Photo 11; they were being replaced. Any other damage was not observed except at bearings in the main frame bridge.



Photo 1 Miaoziping Bridge and the location of falling girder



Photo 2 Span of falling girder

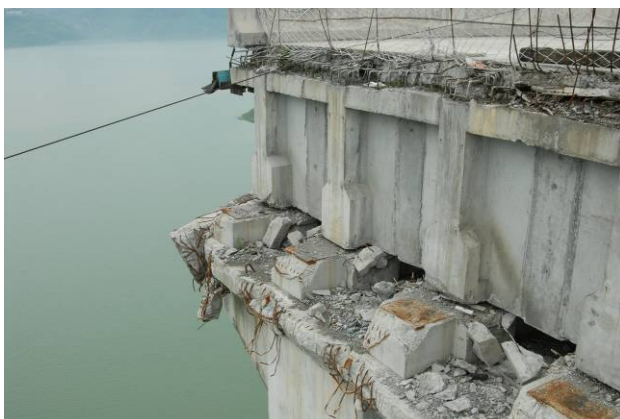


Photo 3 Bearings at the side where the girder bridge starting to fall



Photo 4 Concrete slab torn off due to fall of the girder bridge from the other side



Photo 5 Residual drift between two adjacent girder bridges in the transverse direction



Photo 6 Failure of a side block



Photo 7 Vertical offset between two adjacent girder bridges and damage of center curb



Photo 8 Elastomeric bearings moved out from their position



Photo 9 One of the damaged elastomeric bearings



Photo 10 Damage of an end bearing of main frame box girder bridge



Photo 11 A part of bearings replaced

Baihua Bridge

Baihua Bridge is nearly a 500 m long PC girder bridge on China Route 213 connecting Yingxiu and Dujianyang. It was completed in 2004. As shown in Photo 12, the four-span continuous curved section collapsed. It is located between a precipitous mountain and a river section. By numbering the piers from P2 to P6 as shown in Photo 13, spans between P2 and P6 were supported by about 30 m tall moment-resisting frame piers consisting of two circular reinforced concrete columns and lateral beams. The number of lateral beams depended on pier height. Both center and lower lateral beams were provided at higher piers. End piers had a cap beam in addition to lateral beams, while intermediate piers did not have a cap beam. The seating length was about 600 mm.

Since other sections did not collapse but suffered extensive damage interrupting refuge and rescue, the remaining sections were removed by explosion on May 28, 2008. Therefore, it was difficult to identify the section which collapsed during the earthquake; however, the collapsed section was preserved in its original condition after the earthquake.

Photo 14 shows the damage of P6 and the adjacent pier P7 immediately following the earthquake. The center lateral beam of P6 fell off and the girder tilted toward the mountain side. Furthermore, large cracks occurred at the column and center/lower lateral beam joints of P7. Thus P7 also tilted and drifted in the mountain side.

Photo 15 shows the collapse of P5 (refer to Photo 13). P5 had no cap beam and had only a 1 m x 1 m center lateral beam because P7 was an intermediate pier. Subjected to strong ground motion, the lateral beam was detached and the pier was broken into two pieces. Photo 16 shows a lateral beam-column joint where the lateral beam was detached. The surface of the detached lateral beam was very smooth which apparently showed that plastic hinges were not formed at the beam-column joint. Photo 17 shows another beam-column joint after a lateral beam was detached. The lateral beam was connected to a column by eighteen-29 mm diameter beam rebars that were lap-welded with the column rebars. The bars did not rupture at the welded connection but eccentricity could result in deterioration of their capacity under strong tension. Only 10 mm diameter hoops were provided at 300 mm intervals, resulting in poor lateral confinement of the joints. Thus, the evidence shows that insufficient amounts of reinforcement resulted in a brittle failure at the joints.

Photos 18 and 19 show a rubber pad and its support on the top of a pier. They were located not at the section collapsed by the earthquake, but at the section removed by explosion; however, they provided useful information for identifying the collapse mechanism. Girders were supported by rubber pads without any special connections, which enabled the girder to laterally displace or freely uplift from the piers.



Photo 12 Baihua Bridge right after the earthquake (from Xinhua News)



Photo 13 P2-P6 section collapsed (modified from a photo by Xinhua News)



Photo 14 Damage of P6 and P7 (modified from a photo by Xinhua News)



Photo 15 Collapse of P5



Photo 16 Detached beam-column connection



Photo 17 Column where a lateral beam was detached and rebars at joint



Photo 18 Rubber pad



Photo 19 Shoe seat on a pier

Based on the investigation, failure mechanisms may be considered as follows. Subjected to a significant ground motion, the bridges were extensively moved in the transverse direction. Lateral beams were detached from the columns at the joints, which resulted in deterioration of the continuity of the piers. Piers consequently drifted and tilted. This could develop further responses of the bridge. Collapse of piers due to capacity deterioration at the lateral beam and column connections could directly result in collapse of the bridge. Since girders and piers were not fully connected, the bridge moved and dislodged from support due to excessive lateral displacement and uplift as shown in Fig. 5. It is noted that damage occurred at the curved section which likely developed complicated responses. As a result, the girders possibly fell off the piers and collapsed them. Dislodgement of the girders from their seat in the longitudinal direction due to insufficient seat length could be another possible failure mechanism.

Finally, it was noted that several surface cracks occurred along a line crossing the collapsed section [Lee, 2008], as shown in Photo 13. No further information was available because they were already invisible upon our investigation. Further precise measurement is required to discuss the effect of surface cracks on the damage of this bridge.

Xiaoyudong Bridge

Xiaoyudong Bridge crossed Baishui River in Xiaoyudong Town on Peng-Bai Road, which connects Pengzhou and Longmenshanzhen. It is a 187 m long, 12 m wide, four-span, simply supported, slant-leg, frame bridge that was built in 1998. A pier consists of a single story reinforced concrete moment-resisting frame with two columns and a cap beam on which two decks were simply supported. Two columns of a pier and 12 slant-legs of two adjacent bridges were connected to a pile cap supported by two reinforced concrete piles without a footing. It is likely in such a structure that piles become unstable if lateral force equilibrium from two adjacent decks is lost.

Photo 20 shows the damage of Xiaoyudong Bridge. By numbering abutments, piers, and decks from the left bank, decks 1, 3 and 4 collapsed. Two slant-legs failed at A1 as shown in Photo 21. In the columns, 29 mm diameter main rebars and 10 mm diameter ties were provided; however, the amount of reinforcement was insufficient to form the plastic hinge.

Photo 22 shows the connection of a pile cap, two piles and slant-legs at P2 which supported Deck 2. Flexural and shear failure developed at the slant-legs. These were a typical failure with limited plastic deformation capacity due to insufficient reinforcement. Local buckling of main bars was observed in the slant-legs. If this failure progressed further, the failure shown in Photo 21 would develop.

Photo 23 shows Deck 2, which was about to fall off from the cap beam of Pier 1. This obviously shows that response displacement of Deck 2 was quite large. Deck 2 was displaced not only in the longitudinal direction but also in the transverse direction.

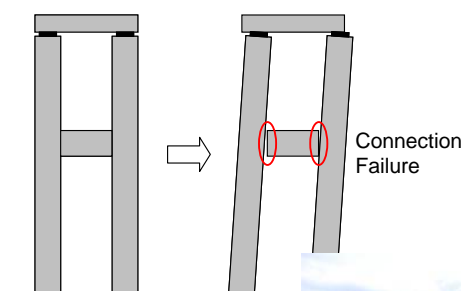


Fig. 5 A possible failure mechanism



Photo 20 Collapse of Xiaoyudong Bridge (Deck 4, Deck 3 and Deck 2 from the left)



Photo 22 Connection of pile cap, slant-legs and vertical piers at P2

Photo 21 Failure of a slant-leg at A1



Photo 23 Deck 2 about to fall from pier cap at P1

At right abutment A2, pavement permanently displaced about 0.2 m in the backsoil side due to the collision of deck 4 and A2, as shown in Photo 24. Extensive shear cracks were developed in the side walls resulting from the collision. Most likely, deck 4 dislodged from A2 support due to excessive response displacement in the longitudinal direction, and then fell down, which resulted in the columns breaking. Insufficient seat length of about 0.3 m at A2 contributed to the failure.

Reinforced concrete pile P2 suffered extensive cracks as shown in Photo 25. This resulted from the extensive drift of P3 in the A2 side as shown in Photo 26. This suggests that Deck 4 collapsed earlier than Deck 3. Once Deck 4 collapsed, the equilibrium of lateral forces between Decks 3 and 4 was lost, and P3 tilted extensively toward A2, which, in turn, resulted in Deck 3 being dislodged from its reliance on P3 and P4. As will be described later, this evaluation is consistent with a farmer's account that Deck 3 collapsed after Deck 4.

Several surface fault displacements occurred around the bridge as shown in Fig. 6. At the left dyke nearly 70 m upstream of the bridge (refer to Point A in Fig. 6), about 1.5 m vertical offset occurred, with lateral drift being very small, as shown in Photo 27. This fault displacement extended downstream along the left dyke and crossed the approaching road at 10 m and 50 m behind A2 (In fig. 6, points B and C, respectively). The fault displacement caused the failure of backsoil and pavement of A2 at Point B and several large cracks on concrete pavements at Point C as shown in Photos 28 and 29, respectively. A farmer named Shunxue Zhu witnessed the collapse of the bridge. He observed that Decks 1 and 4 collapsed at almost the same time, followed shortly thereafter by the collapse of Deck 3.

To determine whether the collapse of Decks 3 and 4 was affected by fault displacements, the authors analyzed them at the right dyke near the bridge. The authors found about 0.3 m settlement at the right dyke, about 50 m upstream of the bridge (Point D, Fig. 6). However as shown in Photo 30, the settlement of the dyke at Point D was much less than the offset of the left dyke, and there was no trace of lateral drift near the dyke. No matter how this offset resulted from the general effect of fault displacement, it was hard to understand that the bridge was seriously affected by such a small ground displacement. No further evidence of a fault

displacement was seen around Point D.

It should be noted here that Hao, et al, reported a 2.8 m lateral and 1.5 m vertical main fault displacement about 190 m upstream near the right dyke [personal communication with Hao, 2008], and reported that this bridge collapsed due to fault displacement [Hao et al, 2008]. The authors can neither agree with Hao, et al, that this bridge collapsed due to fault displacement nor deny it based on the information available from the field investigation. Further precise measurements of ground deformation are required to discuss this issue. Nevertheless, it seems most likely that the bridge collapsed due to the failure mechanisms described above.



Photo 24 Shear failure of side walls and setback of pavement at A2



Photo 25 Failure of a pile at P3



Photo 26 Tilt of P3 and collapse of Decks 3 and 4



Photo 27 1.5 m vertical offset at 70m upstream of Xiaoyudong Bridge

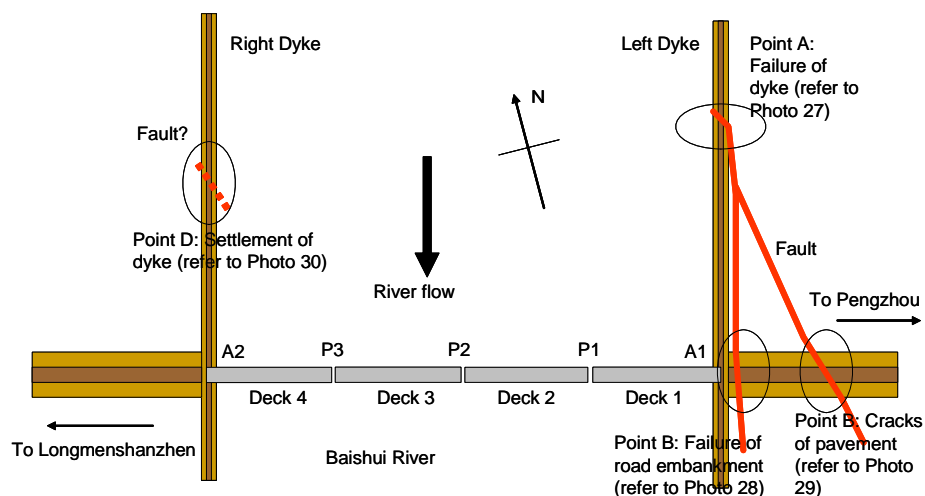


Fig. 6 Fault displacements around Xiaoyudong Bridge



Photo 28 Failure of backsoil and pavement of A1 due to a fault displacement



Photo 29 Cracks of concrete pavement due to a fault displacement



Photo 30 Settlement of right dyke at 50 m upstream of Xiaoyudong Bridge

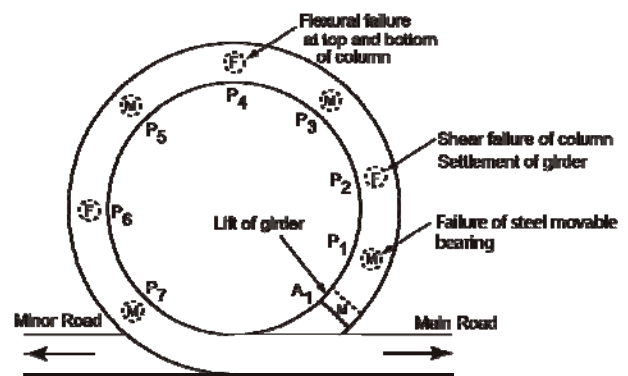


Fig.7 Damage of a lamp bridge, Huilan Bridge

Huilan Bridge

Huilan Bridge is an interchange bridge at the crossing of Demao Road and Huilan Street in Mianyang. This bridge consists of a main bridge and four curved ramp bridges for light vehicles. A ramp bridge suffered damage at its piers as shown in Fig. 7. Other ramp bridges suffered similar damage. They were 170-200 m long ($11\text{m}+(9\sim 11)\times 16\text{m}+15\text{m}$) and 4.5 m wide hollow deck girders with a plane curvature of 25.25m, and they were supported by 0.8 m diameter circular reinforced concrete piers with pre-tensioned PC steel cables inside. Girders were alternatively supported by rubber pad bearings and rigid connections. It was supposed that spans were too short to fully support the girders by rigid connections, and full support by rubber pad bearings would induce a large span displacement in the transverse direction because of the small radius of curvature. Photos 31 and 32 show flexure and shear failure of piers which were rigidly connected to the girders. The failures resulted in settlement in the girders, and a number of extensive transverse cracks occurred at the bottom of deck panels near the support.

Arch bridges

There were many reinforced concrete arch bridges and stone masonry arch bridges in the damaged regions. Most of them survived with limited damage; however, several arch bridges suffered extensive damage. For example, Jingtianba Bridge (China Route 212 and Sicuan Provincial Route 105) connecting Qinchuan of Guangyuan to the large area of Gansu Province, collapsed as shown in Photos 33 and 34. The bridge is about a 270 m long, 7 m wide, and 50 m high, two-span reinforced concrete arch bridge. It was completed in 1997. A part of the mid pier collapse and the right foundation are seen in Photos 33 and 34, respectively. A resident in the area witnessed that the mid piers first toppled in the longitudinal direction, followed by the collapse of the two main arches. However, the account was hardly trusted because the toppling of the mid pier could not occur

alone unless the arch ribs failed. It was likely that excessive response displacement of the arches in the longitudinal direction resulted in failure of the mid pier as well as failure of the arch ribs.

Hongdong Bridge is about an 80 m long, reinforced concrete arch bridge in the district of Mianzhu. It also completely collapsed as shown in Photo 35. Photo 36 shows one of the supports. It was difficult to estimate the failure mechanism because most of the collapsed pieces were removed. It was probably a two-hinge arch bridge based on the reinforcement at the base shown in Photo 36.

Jinhua Bridge is a 228 m long reinforced concrete arch bridge with the main span of 150m as shown in Photo 37. It was completed in 1997. The main arch consisted of two box section ribs and the rise vs. span ratio was 1/6. Main girders were supported by vertical members with rubber bearings. Photo 38 shows cracks of a lateral beam near a beam-vertical member joint. The cracks showed typical damage for low reinforced concrete structural members with insufficient ductility capacity. Similar damage occurred at many other lateral beams. Extensive cracks occurred on the side walls of the abutments as shown in Photo 39, which were induced by collision between girders and the abutment in the longitudinal direction. Shear cracks occurred at the support of the deck at the arch crown as shown in Photo 40. This was also induced by collision between the girders and the support.

Baishui Bridge is located on national road 212 connecting Qinchuan and Guangyuan City. As shown in Photo 41, it is a three span reinforced concrete arch bridge completed in 1988. It did not suffer serious damage but moderate failure occurred at various locations. For example, Photo 42 shows an emergency repair by steel truss for cracks of arch girders in the transverse direction. Also, a large number of cracks occurred at the abutments. Temporary steel trusses (bailey truss) were set on the damaged decks to ensure traffic safety.

There were a number of short span reinforced concrete arch bridges which suffered damage in the fault regions. They were important for daily life inside regional communities. As an example, shown in Photo 43 is Caishenmiao Bridge in Qingchuan. It is a 10 m long reinforced concrete arch bridge. The construction era is unknown but it is perhaps a considerably old bridge. Because cracks ran along the body of the arch, an emergency temporary steel truss (bailey truss) was set up for traffic safety.

There were also a large number of stone masonry arch bridges. For example, Guantong Bridge is a 30 m long unreinforced sandstone masonry arch bridge between skewbacks as shown in Photo 44. It was built nearly 10 years ago. Because it was built on stable bedrock, damage was not extensive. Only top stones suffered compression failure near the arch crown and the 1/4 point as shown in Photo 45 are due probably to large response displacement of the bridge in the longitudinal direction.

Yingchun Bridge in the district of Mianzhu was a 30 m long unreinforced stone masonry arch bridge along a steep slope. It completely collapsed as shown in Photo 46. It failed because the arch supports lost stability and the retaining walls at the supports toppled outward. Though it was a small bridge, it caused a disruption in the lives of the local population.



Photo 31 Damaged P2 (left) and undamaged P3 (right)



Photo 32 Shear failure



Photo 33 Collapse of Jintianba Bridge (Prof. Li Yadong from Xi'nan Jiaotong University)



Photo 34 Collapsed arch rib around the base at left foundation



Photo 35 Collapsed Hongdong Bridge



Photo 36 Support of the main arch



Photo 37 Jinhua Bridge



Photo 38 Cracks of a lateral beam close to beam-column joint



Photo 39 Cracks of side walls at an abutment

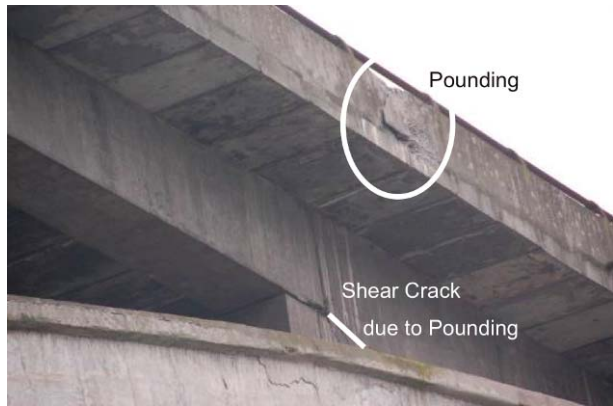


Photo 40 Shear cracks at the support of deck at arch crown



Photo 41 Baishui Bridge



Photo 42 Angle truss for emergency repair of damaged deck



Photo 43 Caishenmiao Bridge



Photo 44 Guantong Bridge



Photo 45 Damage of top stones near arch crown and the 1/4 point



Photo 46 Collapse of Yingchun Bridge

Longchi Bridge

There were many bridges which suffered damage due to slope failure. For example, Longchi Bridge on Duwen Highway between Dujiangyan and Yinxiu is a four-span, simply supported, slab bridge near Longxi Tunnel, as shown in Photo 47. It suffered damage due to slope failure as shown in Photo 48. Lateral drift as well as tilting occurred in the foundations, and this resulted in a large offset of the deck relative to the abutment in the transverse direction.

Zixia Bridge

It should be noted that a number of bridges located in the fault regions remained undamaged. For example, Zixia Bridge located 1 km downstream of Zipinpu dam and 10 km away from Miaozipin Bridge and Baihua Bridge suffered almost no damage. As shown in Photo 49, it is a two-span, simply supported pipe arch bridge with about 80 m spans. The arch is composed of two steel pipes. Transverse girders are hung by the arch through vertical members and deck panels are set on the transverse girders. Damage was very minor because of light steel pipe arches, light weight girders and the steady bedrock ground condition at both shores. Photo 50 shows minor damage, such as partial flaking to the cover concrete of curbs around a pipe arch due to its oscillation.



Photo 47 Longchi Bridge



Photo 48 Offset of the girders in the transverse direction



Photo 49 Zixia Bridge

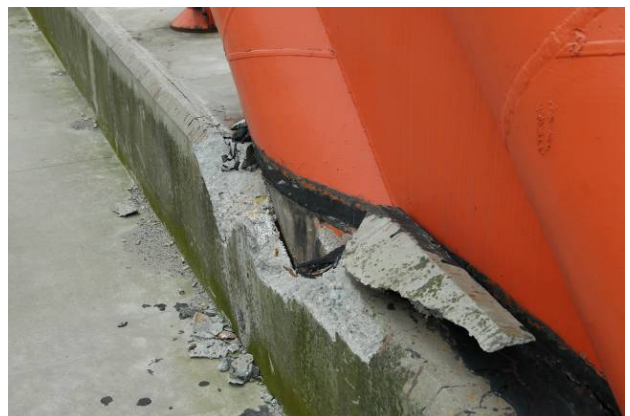


Photo 50 Curb damage at the position of pipe arch

5. FEATURE OF DAMAGE

The damage to bridges may be summarized as follows.

Insufficient seismic design force and lack of ductility capacity

Collapse of most bridges resulted from the failure of reinforced concrete structural members. Important structural members and joints such as lateral beam-column connections in Baihua Bridge and pile caps and slant-leg joints in Xiaoyudong Bridge were insufficiently reinforced to form plastic hinges. Insufficient ductility capacity of the members and joints further exacerbated the damage leading to collapse.

As shown in part 3, design inelastic seismic coefficient $C_i C_z K_h \beta_1$ was in the range of 0.045 and 0.079 if the importance factor C_i was 1.0. This range of inelastic seismic coefficient was insufficient to prevent extensive damage of bridges under significant near-field ground motions during the Wenchuan earthquake. In fact, it seemed that the main structural components were sized and reinforced based on the dead load, and not on the seismic effect.

Absence of unseating prevention devices

There were several bridges in which collapse could have possibly been prevented if unseating prevention devices were provided. Providing enough seat length, connecting a deck and a substructure, connecting adjacent decks, and strengthening anchor bolts of bearings are some prevention practices. Unseating prevention devices were first developed in Japan after the 1964 Niigata earthquake, and it has been effectively implemented worldwide. A couple of devices and practices are generally combined in the implementation.

For example, collapse of one of the five spans of Miaoziping Bridge could have possibly been prevented if unseating prevention devices were installed. Likewise, collapse of the Xiaoyudong Bridge could have also possibly been prevented if seat length was sufficiently long and if the decks, piers, and abutments were properly connected.

In many bridges, side blocks were provided on both sides of the decks so that excessive lateral deck translation can be prevented. However, the side blocks failed and could not limit the deck displacement in the transverse direction as expected.

Fault displacements

An approaching embankment and the backsoils of abutment A1 suffered damage due to fault displacements in Xiaoyudong Bridge. A fault displacement, or at least cracks, occurred on the ground surface crossing the damaged section at Baihua Bridge. Contribution of fault displacements to the failure of bridges is not yet known, but its effect has to be carefully investigated based on measured information.

Slope failure and rock falls

Because mountainous regions were hit by a significant earthquake, there were a number of bridges which suffered damage due to slope failure. For example, decks displaced in the transverse direction because foundations of piers and abutments tilted; they suffered damage due to slope failure in Longchi Bridge.

Unreinforced stone masonry bridges

There were a number of unreinforced stone masonry bridges in the fault regions. They were built with stone masonries available nearby. They were cheap, stable for intense river flow, and matched the landscape of the regions. However, once damage occurred in the main structural components, damage tended to progress to failure such as the collapse of Yingchun Bridge.

6. SUMMARY

The Wenchuan earthquake provided important lessons about how small seismic force designed bridges behaved under a M8 high consequence earthquake. Significant damage of bridges developed within the 300 km long and 50 km wide regions along the fault. Because the fault zone was in mountainous regions, there were several features inherent to the natural environment which were important in interpreting the damage of bridges; 1) short period ground motions were predominant in near-field ground accelerations and response accelerations were not significantly high at 0.5-2 seconds because they sharply decrease as period increases, 2)

stable bedrocks were widely spread and many bridges rested on the bedrocks, 3) extensive landslides and rock falls occurred, 4) liquefaction and lateral spreading did not occur extensively because thick alluvium sandy sedimentation did not exist in the extensive damage region, and 5) surface ground displacements occurred due to the Longmen Shan fault.

Since the seismic design force in the seismic intensity 7 regions was small, it did not contribute, as expected, to enhancing the seismic performance of bridges. This is because the bridges were likely sized and reinforced based on the dead load and not necessarily on seismic effects. Reinforced concrete members with low reinforcement did not form plastic hinges, and sometimes lateral beams were detached from columns at beam-column joints.

However, there were many occasions wherein a strong contrast of damage degree was observed. Consequently, there were many bridges which did not suffer damage or at least suffered only minor damage around a bridge that collapsed. Even old, unreinforced, and insufficiently reinforced bridges survived without significant damage.

It is noted that new bridges on expressways behaved satisfactorily to some extent. One of the five spans of Miaoziping Bridge collapsed. Elastomeric bearings and side blocks suffered extensive damage. However, other structural members survived under strong excitation. Many other bridges which were under construction on expressways suffered limited damage.

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