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A comparative study of bridge damage due to the Wenchuan, Northridge, Loma Prieta and San Fernando earthquakes

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Abstract: A comparative study of selected bridge damage due to the Wenchuan, Northridge, Loma Prieta and San Fernando earthquakes is described in this paper. Typical ground motion effects considered include large ground fault displacement, liquefaction, landslide, and strong ground shaking. Issues related to falling spans, inadequate detailing for structural ductility and complex bridge configurations are discussed within the context of the recent seismic design codes of China and the US. A significant lesson learned from the Great Wenchuan earthquake, far beyond the opportunities to improve the seismic design provisions for bridges, is articulated.

Keywords: bridge damage; Wenchuan earthquake; Northridge earthquake; Loma Prieta earthquake; San Fernando earthquake; ground motion; liquefaction; landslide

1 Introduction

The Great Wenchuan Earthquake of May 12, 2008 (M_s =8.0) occurred at the middle segment of China's north-south seismic belt, with an NE direction along the Longmenshan Fault Zone in the Eastern Margin of the Tibetan Plateau (Yuan *et al*., 2008). The epicenter was located at latitude 31.021°N and longitude 103.367°E, in the Sichuan province. The focal depth was 14 km, and the epicentral intensity was up to XI. Land near the Fault Zone is typical for an alpine-gorge area with mountains and steep valleys. The fault had a length over 240 km, which resulted in large numbers of collapsed buildings and badly damaged lifeline systems. A large number of bridges collapsed or were damaged due to the large fault movement and strong ground shaking.

There are several striking characteristics of this destructive earthquake: the large magnitude, the large ground displacement, major landslides and geological hazards, and the high casualty and economic loss associated with a heavily populated region.

Due to the height and steepness of slopes and their loose geotechnical structure in the mountainous terrain, there were thousands of landslides in the fault zone during the earthquake, resulting in a large amount of geotechnical disasters that created quake lakes, and buried the roads, bridges and villages. Based on a survey reported by the State Key Laboratory for Geohazard Prevention, a total of 8,627 geological disasters occurred, among which there were 3,627 landslides, 2,383 slop collapses, 837 debris flows, 1,694 unstable slopes and 86 places with hidden danger of geological hazard (Yuan *et al.*, 2008).

The Wolong station in Wenchuan county, the nearest strong motion observation station, is 22.2 km from the epicenter. The peak acceleration there was 957.7 cm/s^2. Qingping station in Mianzhu city, the nearest strong observation station, is 0.74 km from the fault. The peak acceleration recorded was 824.1 cm/s^2 (Yuan *et al.*, 2008).

The San Fernando earthquake (magnitude M_w 6.6) occurred on February 9, 1971 (Jennings and Paul, 1971). Its epicenter was at the edge of the San Fernando Valley and its focal point was at a depth of about 13 km. Although this earthquake was of a relatively moderate magnitude, the intensity of surface ground shaking was considerably higher than expected for such an event at that time. At the sites of the five collapsed freeway structures, the horizontal PGA levels were estimated to be approximately 0.6 g (Seismic Safety Commission, 1994), which at that time was considered to be near the upper bound value.

The epicenter of the Loma Prieta earthquake (magnitude 7.1) of October 17, 1989 (Housner *et al*., 1990) was located in a sparsely populated, mountainous area. The fault rupture penetrated upward to within about 4 miles of the ground surface, but did not break the ground surface. The free-field, peak horizontal accelerations of ground motion exceeded 0.60 g close to

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the source and were as high as 0.26 g at a distance of 60 miles. Strong shaking lasted less than 10 s. Soil factors were the single most dominant issue in the Loma Prieta earthquake.

The Northridge earthquake, a magnitude (M_W) of 6.7 event, occurred on January 17, 1994. The epicenter was located at 34°12'N, 118°32'W (California Department of Transportation, 1994; Housner *et al*., 1994). The focal depth has been estimated at about 15–20 km. Strong shaking lasted about 15 s in the epicentral area. Damage was most extensive in the San Fernando Valley, the Simi Valley, and in the northern part of the Los Angeles Basin. The death toll from the quake was 57. The total losses caused by the earthquake are estimated to be at least $10 billion. The earthquake closed several major highways and freeways. In the epicentral area, peak horizontal ground accelerations approached or exceeded 1.0 g in several locations, and peak horizontal ground accelerations exceeding 0.25 g were recorded at 11 sites.

Because of the significant amount of bridges that were damaged and collapsed (over 400) during the Wenchuan earthquake, many studies are currently being pursued with respect to the lessons learned and the potential development of new knowledge to improve seismic design guidelines for bridges. This paper offers some observations on future research emphasis based on a comparatives study of the Wenchuan earthquake with three major earthquakes that occurred in the US.

2 Damage statistics and characteristics

Of the more than 400 bridges that were damaged and/or collapsed during the Wenchuan earthquake, many were simply supported girder bridges, arch and suspension bridges. The statistics of damaged bridges are given in Table 1 (Yuan *et al*., 2008). Most of the bridges located in the affected region were constructed using prestressed I girders supported with or without bearings on reinforced concrete piers. Design provisions, which presumably governed their design and construction, are set forth in the Specifications of Earthquake Resistant Design for Highway Engineering (MCPRC, 1990).

Most of the bridges damaged due to ground motions of the Wenchuan earthquake were associated with one or more of the following damage/failure modes: (1) unseating of the girder in the longitudinal or transverse direction, (2) shear or flexural failures of columns, (3) damage of abutments in the longitudinal or transverse direction, resulting from pounding between the superstructure and the abutments, and (4) damage due to large surface rupture displacement, landslides and loss of support of the bridge piers due to liquefaction.

Forty two bridge structures were significantly damaged during the San Fernando earthquake (counting twin freeway bridges as single structures), including five that collapsed. The most dramatic damage occurred to overpass structures at three major interchanges: the Golden State freeway (I-5) and the Antelope Valley freeway (State Highway 14 interchange); the Golden State freeway and the Foothill freeway (I-210); and the Golden State freeway and the San Diego freeway (Interstate 405) (Jennings and Paul, 1971). These three interchanges are all in the region of strong shaking. This earthquake was the primary reason for the US congress to establish the National Earthquake Hazard Reduction Program (NEHRP) in 1977.

There was only a small percentage of bridges that sustained damage in the Loma Prieta earthquake, and of these, most had been constructed before design standards were improved to reflect lessons learned from the 1971 San Fernando earthquake. Serious damage during the Lama Prieta earthquake occurred to older structures erected on soft soil sites. Damage to these bridges and viaducts was significantly greater than for those located at nearby rock and stiff soil sites (Housner *et al.*, 1990). The 1989 Loma Prieta earthquake was the major motivation for the development of seismic retrofit guidelines for highway bridges by the Multidisciplinary Center for Earthquake Engineering Research (MCEER) (FHWA, 2006).

About 1,200 bridges were located in the region that experienced ground accelerations greater than 0.25 g during the Northridge earthquake and most of these bridges performed well. Bridges constructed to Caltrans' seismic specifications survived the earthquake with very little damage. Seven older bridges (Housner *et al*., 1994), either designed for a smaller earthquake force or without following the ductility requirement, sustained severe damage during the earthquake. Post event studies show that bridges which have substructures with varying stiffness may be vulnerable to shear failures of the stiffer elements. Inadequate seat widths at superstructure hinges can contribute to superstructure collapse. Additionally, skew bridges without proper design are especially vulnerable to seismic events.

Table 1 Statistics of bridge damage in Wenchuan earthquake (Yuan *et al*., 2008)

%

Classification	Damage quantity	Normal	Minor repair	Medium repair	Top overhaul or reconstruct	Reconstruct or rebuild	Damage	Construction
Highways and key projects	576	5.08	34.18	45.68	2.97	11.45	0.53	0.11
Main roads of national highway	1081	7.08	33.90	31.27	11.26	11.55	3.10	1.84

3 Ground fault displacement induced damage to bridges caused by the Wenchuan earthquake

One important observation was the bridge failures due to large permanent ground deformations. One of these bridges was the Gaoshu Bridge, located in Yingxiu Town. The major fault trace was observed cross orthogonally oriented to the Gaoshu Bridge (see Fig.1). The vertical and horizontal displacements of the surface rupture were approximately 50 cm and 100 cm respectively. These large displacements directly lead to the collapse of the Gaoshu Bridge (see Fig. 2).

4 Landslide induced damage to bridges

Some highway and bridge damage during the Wenchuan earthquake was the result of landslides. Much of the damage was quite spectacular. Figure 3 shows some bridges of Duwen Roadway that were pushed over by a large landslide. Since landslides are not normally covered in design bridge codes, this issue may need to be addressed in future code revisions.

Some well-designed rock slope supports generally performed well, without noticeable signs of failure. Therefore, providing better retaining walls in mountainous regions would go a long way to keeping bridges open after earthquakes (although this is not always economically justified).

5 Unseating of near fault bridge spans and effects of complex bridge configuration (curve and higher pier bridges)

5.1 Minjiang Bridge in Yingxiu County

The Minjiang Bridge is located in Yingxiu County crossing the Minjiang River. It is a simply supported girder bridge with continuous slab. The superstructure is supported on rubber bearings. The Minjiang Bridge is located approximately 100 m from the fault and parallel to it. The girder moved transversely 1.8 m, and had a little torsion, but was not unseated. Because of the rubber bearings, the pier damage is less serious, as shown in Figs. 4 and 5.

5.2 Gaoyuan Bridge

The Gaoyuan Bridge is a four span simply supported girder bridge with continuous slab. The girder rested on rubber bearings and between transverse stopper. Surface fault rupture also occurred in this vicinity. The third span from the left had been pushed off the pier cap and collapsed. Backwall and transverse stopper of the abutment in the Gaoyuan Village side were also seriously damaged, as shown in Figs. 6 and 7.

Damage and collapse of the Minjiang and Gaoyuan bridges suggest that near fault design requirements need to be revisited to improve the code.

Fig. 1 Aerial view of Gaoshu and Minjiang bridges

Fig. 2 Collapsed Gaoshu Bridge (Yuan *et al*., 2008)

Fig. 3 Three damaged bridges in Duwen Roadway induced by landslide or rockfall (Yuan *et al*., 2008)

Fig. 4 Minjiang Bridge nearby fault trace (Yuan *et al*., 2008)

Fig. 5 Transverse displacement of girder (Yuan *et al*., 2008)

Fig. 6 Gaoyuan Bridge nearby fault trace (Chen, 2008)

Fig. 7 Unseating of superstructure (Yuan *et al*., 2008)

5.3 Shoujiang Bridge

The Shoujiang Bridge is located in Xuankou County. It is a multiple span simply supported and slab continuous bridge. The pier height is more than 40 m. The main girder moved longitudinally with the side span nearly unseated. The wingwall and the backwall of the abutment were damaged as seen in Figs. 8–10. The extremely large displacement response of the Shoujiang Bridge may have been induced by the very high pier.

Similar to the Shoujiang bridge, due to the very high pier, one approach span of the Miaoziping Bridge was unseated.

5.4 Baihua Bridge

During the Wenchuan earthquake, at least two curved bridges collapsed; one is Baihua Bridge, and the other is Yuzixi Bridge (Figs. 11 and 12).

The Baihua Bridge is 495.55 m long and is supported by twin-column piers. The maximum pier height is 30.87 m. The superstructure rested on rubber bearings and pot bearings. The Baihua Bridge consisted of six segments. The construction was completed in 2004. It is located about 2–3 km from the fault.

As can been see from Figs. 13 and 14, the 5-curve span segment from pier 13 to 18 totally collapsed. It is believed that the girder first unseated from the top of pier 18 that initiated the collapse.

6 Damaged bridge piers

During the Wenchuan earthquake, damage to bridge piers was less serious than in the San Fernando, Northridge and Loma Prieta earthquakes. It is believed that the sliding of rubber bearings helped to reduce the base shear. On the other hand, there were poor reinforcing details and damaged rigid frame bridges, as shown in Figs. 15 and 16.

Both types of bridge damage seen in Figs. 15 and 16 suggest the lack of transverse reinforcement and inadequate concrete confinement.

Fig. 8 Shoujiang Bridge (Li *et al*., 2008)

Fig. 9 Dislocated of superstructure (Yuan *et al*., 2008)

Fig. 10 Abutment damage (Yuan *et al*., 2008)

Fig. 11 Collapsed Baihua Bridge (Li *et al*., 2008)

Fig. 12 Collapsed Yuzixi Bridge (Yuan *et al*., 2008)

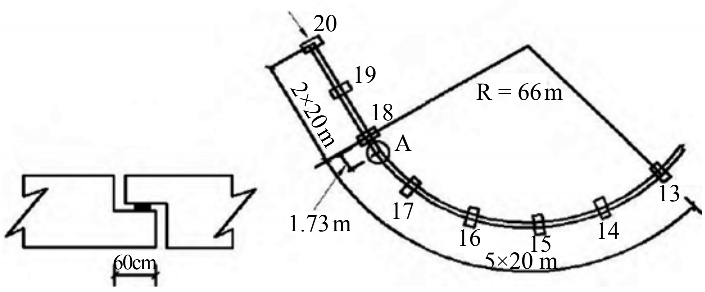

Fig. 13 Details of pier top of (Li *et al*., 2008)

Fig. 14 Collapse of five curve span

Fig. 15 Shear failure of column of Mianyang Airport viaduct (Chen *et al.*, 2008)

Fig. 16 Failure of column top of ramp of Huilan overpass (Yuan *et al.*, 2008)

7 Damage characteristics of bridges in US earthquakes

The collapse of five concrete bridges during the San Fernando earthquake initiated revolutionary changes in seismic design criteria and performance standards in the US. Decks that were pulled off their supports at expansion joints were the weak link for many bridges that existed in 1971. Figure 17 shows the overcrossing at the Golden State Freeway (I-5) and Foothill Freeway (I-210) interchange that carried southbound traffic from the Foothill Freeway onto the Golden State Freeway. The 770-ft. long box girder deck of this overcrossing was supported on six single-column piers with an abutment at each end. It had one expansion joint near the mid-span and two each at the two abutments. Separation at the centrally located expansion joint allowed one end of an adjacent span to fall off its support, which initiated the collapse of the entire structure (Seismic Safety Commission, 1994).

This earthquake also revealed certain deficiencies in RC columns, such as inadequate ties, both in size and spacing, as shown in Fig. 18.

In the Loma Prieta earthquake, most of the bridges damaged were constructed prior to 1971. The most serious damage occurred to older structures located on soft ground. The most tragic impact of the earthquake was the life loss caused by the collapse of the Cypress viaduct (see Figs. 19 and 20). The Cypress viaduct and the San Francisco·Oakland Bay Bridge had been retrofitted with cable restrainers to limit the relative motions between adjacent decks at expansion joints (Housner *et al.*, 1990).

The installation of cable restraints through a seismic retrofit program appears to have improved the seismic behavior, possibly preventing some spans from collapse by limiting the relative displacements of the decks at the hinges. However, focusing on bridge components, in hindsight, may have inhibited the likelihood of identifying deficiencies in overall seismic behavior such as those uncovered in the collapse of the Cypress Viaduct and the Bay Bridge. Therefore, the Loma Prieta earthquake provided lessons on the relative importance of considering the response of the entire structural system including soil-structure interactions.

The Northridge earthquake caused the collapse or partial collapse of seven concrete freeway bridges and damaged many more. The two bridges of these bridges that collapsed at the SR-14/I-5 interchange, the North Connector Overcrossing and the South Connector and Overcrossing, were different from the structures at this same interchange that collapsed during the 1971 San Fernando earthquake. They were designed prior to 1971 but their construction was not completed until 1974 (Housner *et al.*, 1994), when it was believed that hinge restrainers would be adequate to prevent the type of collapse that occurred during the San Fernando earthquake. Therefore, pre-1971 design detailing was followed in constructing these bridges but hinge restrainers were added to prevent the decks from failing off their narrow support seats. During the Northridge earthquake, brittle shear failures in short stiffer columns next to the abutments initiated collapse as shown in the Fig. 21.

I-5 crossing Gavin Canyon on two skewed five-span bridges (Fig. 22) was constructed in 1967. Cable restrainers had been installed across the hinge seats in a 1974 retrofit (Housner *et al*., 1994). Displacements were very large to cause the loss of bearing support at the hinge seats. The unseating caused flexural failures of the box girder. An apparent cause was the large torsional response associated with skewed geometries.

Fig. 19 Cypress viaduct (Housner *et al*., 1990)

Fig. 17 Golden State Freeway (I-5) and Foothill Freeway (I-210) interchange (http://www.smate.wwu.edu)

Fig. 20 San Francisco·Oakland Bay Bridge (Penzien *et al.*, 2003)

Fig. 18 Foothills Freeway overpass (http://ntl.bts.gov)

Fig. 21 SR-14/I-5 interchange (http://www.smate.wwu.edu)

Fig. 22 I-5 Gavin Canyon undercrossing (http://ntl.bts.gov)

These failures clearly demonstrated that retrofitting such structures with hinge restrainers alone is inadequate. Large skews in geometry result in excessive torsional response during earthquakes; thus, they are detrimental to overall good seismic performance. Therefore, methods are needed for improved design of skewed bridges, possibly including elimination of the skew where feasible, elimination of in-span hinges, and lengthening of seats.

8 AASHTO (US) and MCPRC JTJ004-89 (China) seismic design specification

The following are comparison of certain seismic design provisions between AASHTO LRFD and MCPRC JTJ004-89 with respect to design earthquake force, detailing to prevent unseating prevention and requirements of transverse reinforcement.

8.1 Design earthquake force

The Specifications of Earthquake Resistant Design for Highway Engineering (MCPRC JTJ 004-89, China) (MCPRC, 1990) specifies that bridge structures shall be designed to resist a seismic force (F) in accordance with the following formula:

$$EQ = C_i C_z K_h \beta W \tag{1}$$

Where, C_i is the importance factor, C_z is the combined effects of the coefficient, K_h is the acceleration coefficient, $\beta = 2.25(0.3/T)^{0.98}$ (for soil type II), where, $T \geq 0.3\text{s}$ is the elastic seismic response coefficient, T is the period of vibration, and W is the structural weight.

The AASHTO Guide Specifications for LRFD Seismic Bridge Design (AASHTO, 2008) requires that bridge structures shall be designed to resist a seismic force (F) in accordance with the following formula:

$$V_{col} = \frac{C_{sm} W}{R} \tag{2}$$

where,

$$C_{sm} = A_S + (S_{DS} - A_S)(T_m/T_0), \text{ for } \quad T_m < T_0 \tag{3}$$

or,

$$C_{sm} = S_{DS}, \text{ for } \quad T_0 < T_m < T_s \tag{4}$$

or,

$$C_{sm} = \frac{S_{D1}}{T_m}, \text{ for } \quad T_s < T_m \tag{5}$$

where, $A_s = F_{pga}\, a_{PG}$, $S_{DS} = F_a S_S$, $S_{D1} = F_v S_1$, a_{PG} is the peak ground acceleration coefficient on site class B, S_S is the horizontal response spectral acceleration coefficient at 0.2 s period on site class B, T_m is the period of vibration of the mth mode, T_0 is the reference period used to define spectral shape = $0.2T_S$, T_S is the corner period at which spectrum changes from being independent of period to being inversely proportional to period = S_{D1}/S_{DS}, S_1 is the horizontal response spectral acceleration coefficient at 1.0 s period on site class B, R is the response modification factor, and W is the structural weight.

To compare the design earthquake force of the two design specifications, a standard bridge supported by single-column bents was chosen. The pier height is 10 m. The bridge site type is assumed to be type II (or site class B). The bridge is placed in Wenchuan and in California and is assumed to be an essential bridge. Its fundamental vibration period is 0.6 s. The comparison is given in Table 2.

8.2 Detailing to prevent unseating

In order to prevent unseating of bridge spans, both seismic specifications provide requirements for unseating prevention detailing. Support lengths of intermediate joints (hinges) are considered the first method of preventing unseating of a bridge span.

Minimum support length in the MCPRC JTJ004-89 Specifications is calculated using the following empirical formula considering the bridge span length L (unit, m):

$$a \geq 50 + L \text{ (cm)} \tag{6}$$

The support length shall be calculated by equation 6 but not less than 60 cm. Moreover, for simply supported bridges, shear key, stopper and other falling-off prevention devices shall be provided to prevent longitudinal or transverse unseating. For curved bridges, anchorage bolt connections between side pier and

Table 2 The comparison of design earthquake force

Parameter	AASHTO LRFD	MCPRC JTJ004-89
Peak ground acceleration (PGA) coefficient, a_{PG}	0.6 g	0.1 g
Response modification factors R or C_Z	2.0	0.3
Important factors	---	1.3
Soil factor of site class B or II	1.0	1.0
S_S and S_1	1.432 g, 0527 g	---
C_{sm} or β	0.878 g	1.141 g
Column force	$0.732 \times a_{PG} \times$ W	$0.45 \times a_{PG} \times$ W

superstructure shall be provided to prevent unseating from side pier.

For AASHTO LRFD Specifications, the minimum support lengths is calculated using the following empirical formula considering the structure length L, column height H, and skew S. Support length is increased depending on the seismic performance zone:

$$N\,(\text{inches}) = (8 + 0.02 \times L + 0.08 \times H)(1 + 0.000125 \times S^2) \tag{7}$$

For a straight bridge, the span length is 30 m, pier height is 10 m, support lengths N of AASHTO LRFD is 160.3 cm using Eq. (7), and the support lengths a of MCPRC JTJ 004-89 is 80 cm using Eq. (6).

The AASHTO LRFD specification also takes into account the effects of pier height and skew on minimum support, while MCPRC JTJ004-89 does not. From unseating damage of the Shoujiang bridge, New Nanba bridge and Baihua bridge during the Wenchuan earthquake, higher pier and skew and curved bridges should require more support length.

8.3 Transverse reinforcement

In the MCPRC JTJ004-89 specification, the maximum spacing for lateral reinforcement at the pier top and bottom in the length of the cross-section dimension of piers shall not exceed 10 cm, stirrup diameter is not less than 8 mm, and the length of pier top and bottom should also not be less than 50 cm. The spirals shall be welded and the hoops shall be anchored by 135º-hooks surrounding a longitudinal bar plus adequate extension into the core concrete.

In the AASHTO LRFD specification, explicit requirements of transverse reinforcement inside and outside the plastic hinge region are provided.

Table 3 gives a comparison of the transverse reinforcement ratios betwfeen AASHTO LRFD and MCPRC JTJ 004-89.

Table 3 The comparison of transverse reinforcing ratio

	Spiral	Hoop
AASHTO LRFD	$\rho_s = 0.45\frac{f_c'}{f_{yh}}\left[(\frac{A_g}{A_{he}}) - 1\right]$ or $\rho_s = 0.12\frac{f_c'}{f_{yh}}$	$A_{sh} = 0.12sh_c\frac{f_c'}{f_{yh}}$ or $A_{sh} = 0.3sh_c\frac{f_c'}{f_{yh}}\left[(\frac{A_g}{A_{he}}) - 1\right]$
MCPRC JTJ004-89	Longitudinal and transverse $\rho_s = 0.3\%$	Longitudinal and transverse $\rho_s = 0.3\%$

Where, A_g, A_{he} Gross cross section area of pier and confined concrete area of pier; f_c Compressive strength of unconfined concrete; f_{yh} Nominal yield stress of transverse column reinforcement (hoops/spirals); S vertical space of hoop; H_c core dimension of tied column in the direction under consideration; ρ_s, A_{sh} Ratio of volume of spiral or hoop reinforcement to the core volume confined by the spiral or hoop reinforcement.

9 Summary and discussion

This study examined several bridge damage/collapse modes due to the Wenchuan earthquake and compared them with similar bridge damage/collapse modes that occurred in three major earthquakes in the United States in recent decades within the context of lessons learned and improvement of bridge design guidelines.

It is well understood that lessons learned due to destructive earthquakes in the past were very instrumental in revising and improving design codes. This fact is well demonstrated in bridge engineering practices in China, Japan, US and other countries over the past few decades. In this paper, several key improvements in the US based on lessons learned from the San Fernando, Loma Prieta and Northridge earthquakes are briefly noted.

9.1 Summary of lessons learned

(1) Many bridges in the areas affected by the Wenchuan earthquake were subjected to ground shaking that was much higher than the code specified design earthquake intensity. Geological hazard induced damage in this destructive earthquake were more severe than those in the US major earthquakes.

(2) Premature failure of some bridge bearings appeared to have reduced seismic damage in their supporting substructures by uncoupling them with the superstructures. This fuse-like action with stopper or shear key may have saved a number of spans from collapse and columns from shear and flexural failures. Design of shear keys must consider the entire structural system to prevent falling spans and limit the earthquake force transmitted to the substructure.

(3) Studies on damage characteristics of many multi-span girder bridges supported on rubber bearings during the Wenchuan earthquake may provide valuable information to the Central and Eastern United States, where bridge bearings are typically used.

(4) Skewed or curved bridges, particularly those with high piers, should be avoided in high seismic zones. If possible, the skew angle of bridges should be limited

to a small value. Otherwise, special analyses should be required to substantiate the design.

(5) The Wenchuan earthquake clearly showed that bridges located near faults or across faults sustained serious damage or collapse. Similar observations were made in the 1999 Taiwan earthquake and the 1999 Turkey earthquake (Anastasopoulos *et al*., 2008). Design of fault crossing bridges by considering both the near fault ground motions and the permanent displacements is a challenge. Recently, Anastasopoulos *et al*. (2008) present a methodology for the design of bridges against tectonic deformation. Billings (Fédération internationale du béton Task Group 7.4, 2007) reviewed some bridge design projects for active fault crossings, where unique approaches were taken to meet the challenging demand for individual fault crossing bridges. The Wenchuan earthquake offered more emphasis on the importance to formulate seismic design guidelines and strategies for bridges located near faults or even across faults against large displacement demands.

(6) Landslides have a widespread and critical impact on bridge and lifeline infrastructure, but current codes do not address them. Post-event investigations show some well-designed rock slopes supports generally performed well. This challenging issue should be considered in future bridge code modifications.

9.2 Further thoughts

While each major earthquake will have its special characteristics and the lessons learned may be different, the Wenchuan earthquake is an unusual event which needs to be viewed from a broader perspective to minimize casualties and property damage in densely populated regions or urban centers. While there have been other large magnitude earthquakes in the past, the Wenchuan event was unparalleled in the number of large aftershocks, massive landsides, debris flows and other geological hazards, and widespread damage/collapse of buildings and lifeline infrastructure systems in heavily populated regions (over 70,000 fatalities, 370,000 injured and 5,000,000 people left homeless).

Because of the special characteristics of the Wenchuan earthquake as noted above, particularly for highly populated regions, greater and deeper thoughts must be given to regional resilience of urban centers to destructive earthquakes with rare occurrence but high consequence. This will require an integrated effort between planning, mitigation, preparedness and emergency response professionals on a regional basis. Within this context, destructive earthquakes are measured more in level of destruction and consequences rather than magnitude alone. In the US, metropolitan areas such as Boston and New York City, for which serious destruction may occur under moderate earthquakes, need to be prepared.

Seismic performance and requirements of bridges and other components of the highway system serving a region must be planned and designed to satisfy a targeted level of regional resilience to destructive earthquake. This broader perspective should be one of the important objectives in developing the next generation seismic design provision for bridges. It will be a significant challenge to integrate interdisciplinary team efforts within the current social and organizational structure of our societies.

Within the regional resilience perspective, important and essential bridges must consider landslides and other geological disasters, as well as develop creative and unique mitigation strategies and measures for near-fault and cross-fault location. Ample examples are available (e.g. refs 17 and 18). With respect to regional resilience against low probability and high-consequence earthquakes in dense population centers, plans should be made for important buildings (schools, hospitals, command centers etc.) and essential lifeline systems (utilities and transportation networks) to meet a certain targeted level of performance such as minimum damage. Therefore, the relative importance of bridges will have additional meaning, because of the importance of the highway system for first responders and for evacuations. Beyond mandatory requirements for essential buildings and lifeline systems, a regional resilience plan may have to allow for limited fatalities due to cost or owners choice in the design of the built environment. Furthermore, because emergency responses are more or less the same for all natural and manmade disasters, a strong argument can be made for considering multiple hazard resilient regions as an agenda for future international cooperative research.

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