- Technical Paper -

SEISMIC BEHAVIOR EVALUATION OF A RC RIGID-FRAME ARCH BRIDGE SUBJECTED TO DIFFERENT TYPES OF SEISMIC WAVE

Zhongqi SHI^{*1}, Kenji KOSA^{*2}, Jiandong ZHANG^{*3} and Hideki SHIMIZU^{*4}

ABSTRACT

Xiaoyudong Bridge received great damage in Wenchuan EQ, 2008. Being a RC rigid-frame arch bridge, its dynamic behavior has not been sufficiently studied. In this paper, it is found that the girder near the joints with arch leg, and the bottom of inclined legs are crucial points. Besides, the 1st mode mainly dominates the horizontal vibration, while the 2nd mode mainly controls the vertical. However, different components have different effects on the axial force variation and the flexural vibration. Wolong wave provides most severe damage due to the strongest wave component in U-D direction. Keywords: Wenchuan Earthquake, rigid-frame arch bridge, vibration behavior, seismic effect

1. INTRODUCTION

The Wenchuan EQ occurred in Sichuan Province, China, on May 12th, 2008. The magnitude was 8.0 by CEA and 7.9 by USGS^[1]. Reports have been published saying that nearly 1600 bridges suffered extensive damage. Authors conducted detailed field surveys on Xiaoyudong Bridge, as shown in Fig. 1, which crossed Baishui River in Xiaoyudong Town on Peng-Bai Road. This bridge is a 189m long, 13.6m wide, 4 spans, RC rigid-frame arch bridge that was built in 1998. Rigid-frame arch bridge is a composite structural type of arch bridge and inclined rigid-frame bridge, and a high-order hyperstatic structure with horizontal thrust. It has been abundantly constructed in China since 1980s. By a statistical investigation, the accumulative total length of this type of bridge exceeds 15,000 km.

In previous study, detailed observed damage was presented, and pushover analyses were done to judge its capacity, and to approach the failure mechanisms^[2,3].

But the vibration behavior still should be studied. Besides, the effect of dominant modes of cable-stayed bridge has been evaluated^[4], but still not for RC arch bridge. Thus, dynamic analyses by using 3 waves (2 by 2008 Wenchuan EQ and 1 by 1995 Hanshin EQ) are conducted to judge the seismic behavior of Xiaoyudong Bridge under different seismic wave effects.

2. INFORMATION OF OBJECT BRIDGE

Detailed information of Xiaoyudong Bridge has been summarized in Reference [2] and [3] for both bridge structure and damage condition. As shown in Fig. 1, totally 4 spans were arranged. Abutments, piers and spans are numbered from left. Span 1, 4 and Span 2, 3 has the length of 42.35m and 43.15m respectively. Details of Span 4, which is to be modeled in following analyses, are shown in Fig. 2. Arch leg (Point A) and inclined leg (B) has 21° and 40° slope respectively. Arch legs from each side and girder in mid-span form



Fig. 1 Elevation view of Xiaoyudong Bridge after the earthquake (view from upstream)



Fig. 2 Structure and analytical model of Span 4

- *1 Graduate Student, Dept. of Civil Engineering, Kyushu Institute of Technology, JCI Member
- *2 Ph.D., Prof., Dept. of Civil Engineering, Kyushu Institute of Technology, JCI Member
- *3 Senior Engineer, Jiangsu Transportation Research Institute, China, JCI Member
- *4 Senior Engineer, Structural Engineering Division, Nippon Engineering Consultants Co., Ltd., JCI Member

the arch frame. It composes one single rigid-frame with inclined legs, and girder at ends. A span consists of five rigid-frames connected by crossing beam, arch slab, and extending slab. Besides, Fig. 1 also introduced the damage condition generally. A1 suffered collision with the deck. Span 1, 2 survived to stand still, but arch legs of Span 1 collided with revetment and failed due to shear. Span 3, 4 fell into the river entirely. P3 tilted about 7.5° toward A2. A2 suffered notable shear failure, which was caused by the collision with deck before its collapse. Besides, greatest span length change occurred to Span 1 about -0.347m (-0.83%), while the shortening of Span 2 reached only -0.052m (-0.12%), and Span 3, 4 were slightly shortened by 0.252m (-0.29%) totally.

3. ANALYTICAL CONDITIONS

3.1 Analytical Model

Frame model is made for Span 4, in Fig. 2. On the transversal direction, noticing five arch frames were arranged, we select one single frame to establish the 2-D model. Due to greater sectional area and rebar amount, rigid element is set to footings, beam on the top of pier and joints between legs and girder. Besides, horizontal and rotational springs under P3 and A2, supporting and friction spring upon P3, supporting, friction and collision springs upon A2 are assumed based on former studies^[5]. Additionally, tri-linear M- Φ relationships are calculated by the axial forces under dead load based on the Japanese specifications^[6]. Damping coefficient of 2% and 20% is respectively used to superstructure and base springs.

3.2 Characteristics of Natural Vibration

To discuss the dynamic behavior, the eigenvalue analysis has been conducted firstly. The characteristics of natural vibration are listed in **Table 1**. Judged by the effective mass ratio, the 1st, 3rd, 4th and 6th modes are the dominant modes in horizontal direction, totally occupying 92.5% of the effective mass, while the 2nd and 3rd modes contribute about 64.9% in vertical direction. Besides, the 4th, 10th and 11th modes also have certainly effective mass. Thus they may affect on the vibration as well.

Generally, the 1st, 4th modes and the 2nd, 3rd modes provide greatest mid-span displacement in horizontal and vertical direction respectively. Thus, their vibration modes are shown in Fig. 3. The 1st mode (0.625s) vibrates as 1st-order anti-symmetric. The vibration of the 2nd mode (0.397s) is controlled by the up-down movement at mid-span. The 3rd and 4th mode deforms in the 2nd order symmetric and anti-symmetric shape respectively.

3.3 Faults, Waves and Analytical Cases

Two main surface faults have been defined after Wenchuan EQ, Fault-1 and Fault-2 in Fig. 4. These two surface faults roughly divided the zones of intensities geographically. Also important is another fault named Xiaoyudong Fault (Fault-3), which roughly connected the main two. Our object bridge located between Fault-1 and Fault-2, and very close to Fault-3. Based on

Table 1	Characteristics	of	f natura	vibration
---------	-----------------	----	----------	-----------

Mode	Period (s) -	Effective Mass Ratio		
		Х	Z	
1	0.625	22.61%	0.17%	
2	0.397	2.17%	52.91%	
3	0.188	12.96%	12.01%	
4	0.122	29.75%	<u>7.50%</u>	
6	0.102	27.17%	2.07%	
10	0.060	0.09%	<u>5.89%</u>	
11	0.055	0.68%	<u>6.99%</u>	
	Σ	95.43%	87.55%	

(Shadowed: dominant modes;

Underlined: other important modes)



Fig. 3 Dominant modes of natural vibration (×25)



Fig. 4 Faults and nearby seismic observing stations



Fig. 5 Acceleration spectra of input waves



the investigation^[1] on nearby surface faults, Fault-3 probably extended along the left dyke of Baishui River and crossed the road at 10m and 50m behind A1, which is considered to be the main reason for the collisions occurred between A1 and the deck, and between the revetment and arch legs. However, on the other side, there was no obvious trace of surface fault having been found at the right dyke (A2 side).

Further, the locations of two available seismic observing stations, Bajiao and Wolong, are illustrated in Fig. 4. Bajiao wave is used as standard case for its shortest distance from the bridge for Case-1, while Wolong wave and Takatori wave (in 1995 Hanshin EQ) are used for Case-2 and Case-3 respectively for parameter studies. Being a special RC arch bridge, the bridge is significantly affected by U-D component of wave. Thus, both the U-D and E-W (since the bridge is roughly along E-W direction) components are used. Response acceleration spectra are shown in Fig. 5.

4. ANALYTICAL RESULT OF CASE-1

4.1 General Result

The max plastic ratio distributions for both sides of flexure are calculated according to Eq. 1, and shown in Fig. 6, to judge the general damage condition.

$$\mu_{\rm max} = \Phi_{\rm max} / \Phi_{\rm y} \tag{1}$$

where, μ_{max} : max plastic ratio; Φ_{max} : max response curvature; Φ_y : yield curvature. As shown in Fig. 6 (a) for the girder, Point A and C receive most serious failure (as their max plastic ratios reached at 26.781 and 19.468 respectively). The points around Point B in mid-span also damage notably, with μ_{max} ranges from 1.391 to 4.121. Besides, the bottoms of leg (Point D and E for the inclined legs, Point F and G for the arch legs, from (b) to (e)) are found to have relatively severe response (μ_{max} from 0.718 to 0.836) among legs. Thus, Point A ~ G are checked as crucial points in following evaluation.

The curvature history is shown in Fig. 7 for Point A and C. As the vibration becoming strong, Point C reaches the yield curvature at 36.55s firstly, followed by Point A at 36.74s. Their non-linear response can be confirmed in Fig. 8. Then, Point E, the right bottom of inclined leg, yields due to extensive decrease of axial



Fig. 7 Curvature histories of Point A & C (36s~38s)



Fig. 8 M-Φ histories of Point A & C



Fig. 9 M-N histories at bottoms of inclined legs

force at 37.42s (in Fig. 9). Besides, as shown in Fig. 9 as well, the axial force in the inclined legs varies considerably from -140% to +149%. This is a phenomenon with considerable importance that affects the failure of inclined legs. However, as being a horizontal member, the girder is not influenced by the

axial force variation noticeably. Besides, no obvious event is found for both flexural failure and axial force variation of the arch legs. Therefore, Point A, C, D and E are considered as the crucial points for this bridge.

The development of deformation shape for these first three yields is illustrated in Fig. 10. Compared with the modes shown in Fig. 3, the dominant modes can be roughly judged. The deformation shape in (a) and (c) are able to be confirmed by the 2nd and 1st mode respectively, while (b) may be considered as that, based on the shape of the 2nd mode, rotation of both sides bring in the effect of the 1st mode. Thus, the 1st and 2nd modes mainly control the vibration.

4.2 Seismic Behavior Judgment by Modes

Mentioned in 4.1 in Fig. 10, the vibration is mainly dominated by the 1st and 2nd modes. To judge the effect of dominant modes on vibration numerically, FFT method is applied for response acceleration and displacement (Point B is taken as the representation), shown in Fig. 11 and Fig. 12 respectively.

It can be obviously discovered that near the natural periods of the 2nd (0.397s) and 3rd (0.188s) modes, peak amplitudes occurred to the vertical direction for both the response acceleration (0.405s and 0.175s) and the response displacement (0.405s and 0.189s). It indicates that the vertical vibration is mainly controlled by the influence of the 2nd and 3rd modes. On the other hand, for the horizontal direction, although the Fourier amplitude of response acceleration can not show noticeable peak point according to the period, peak point (0.554s) near the natural period of the 1st mode (0.625s) can be found for the displacement. This suggests that the vibration in the horizontal direction is mainly controlled by the 1st mode. But other modes may influence on it as well.

This result is confirmed by the comparisons shown in Fig. 13. The 1st mode and the deformation, when girder reaches maximum horizontal displacement, are shown in Fig. 13 (a), from which we can see clockwise rotations at both sides made the deformation similar to the 1st mode. (b) illustrates the 2nd mode and the deformation when Point B has maximum vertical displacement. Although the failure of mid-span caused the notable drop at middle, both 2 comparisons show similar deformation shape, and good coincidence between the dominant modes and the actual vibrations.

5. RESULTS OF CASE-2 AND CASE-3

In this chapter, by the dynamic analyses under different waves (Case-2: Wolong wave, Case-3: Takatori wave), the effect of E-W and U-D components is discussed by evaluating the max plastic ratio for girder and legs, and the variation range of the axial force for legs. Then, the dominant modes for vibration, explained in 4.2, are confirmed by these two cases.

5.1 Evaluation of Max Plastic Ratio and Axial Force Variation

By Eq. 1, max plastic ratio is calculated and compared in Fig. 14. We can see that the max plastic ratio distributions of mid-span are with similar shapes:



(c) 37.42s: Yield of Point E by tension





(a) 42.08s: Girder Max Horizontal Disp (×25) & 1st mode (×50)



Fig. 13 Deformation of analytical result and mode

 μ_{max} is relatively great near Point A, B and C. It indicates that the general damage conditions of girder did not obviously distinguish from case to case. Thus, conclusion can be drawn that Point A and C are the weakest points of this bridge, while Point B may suffer notable damage too. Moreover, we can find that Point A, B and C suffered most noticeable failure in Case-2, as the comparison summarized in Fig. 15. The max plastic ratio at the bottoms of legs (where were also found to be weak points, mentioned in 4.1) are shown in Fig. 15 as well. Phenomena are found that for the girder, Case-2 (Wolong wave) shows the most destructive influence, followed by Case-3 (Takatori wave). But for the bottoms of legs, Case-3 affected the bridge most severely. Besides, Case-1 was weakest to destroy the bridge for both the girder and the legs judged by μ_{max} .

As discussed for Case-1, the variation of axial force is the most important failure type of legs. Thus, its variation ranges are calculated. It is found that the axial force changes most severely in Case-2 (+221% to -166% for inclined legs, +107% to -113% for arch legs). In the other two cases, the axial forces vary in much smaller ranges (+149% to -140% in Case-1 and +137% to -115% in Case-3 for inclined legs, less than 100% in both cases for arch legs). Thus, although Case-3 caused strongest flexure failure to the legs (by μ_{max} in Fig. 15), the axial variation is relatively limited. Thus, the effect by Case-3 is not as strong as that by the others.

5.2 Evaluation of Dominant Modes

Similar to the discussion in 4.2, the evaluation of dominant modes by FFT is conducted for Case-2 and Case-3 as well and shown in Fig. 16 and Fig. 17.

As illustrated in Fig. 16 (a) for Case-2, the peaks near the 2nd and 3rd modes can be found for the acceleration in both directions (0.429s and 0.190s for the vertical, 0.418s and 0.209s for the horizontal). For the displacement (shown in (b)), the 2nd (0.429s) and 1st (0.566s) modes control the vertical and horizontal vibration respectively. Thus, the result for displacement can coincide with Case-1 well, while the result in horizontal direction for acceleration distinguishes obviously. This is considered to be caused by extensively strong U-D wave in Case-2. On the other hand, the result of Case-3 (shown in Fig. 17) shows very clear trend that the 2nd and 1st modes dominate the vertical and horizontal vibrations separately (0.394s and 0.585s for the acceleration, 0.394s and 0.589s for the displacement). As a result, although the 3rd mode affects the vertical acceleration and the 2nd and 3rd modes influence the horizontal acceleration in Case-2, for the general vibration, the 2nd and 1st mode mainly dominates vertical and horizontal direction respectively.

5.3 Summary

To summarize the influence from seismic waves, the properties of wave are summarized for 2 directions separately (in Fig. 18) according to Fig. 5. Based on the previous explanation of modes effect, the accelerations of the 1st and 2nd modes are used as representations for the horizontal and vertical vibration respectively. From Fig. 18, we can see that Case-1



Fig. 14 Max plastic ratio distributions of mid-span



(Bajiao wave) has similar strengths for both directions (654gal and 539gal). Case-2 (Wolong wave) gives extremely strong U-D wave (1416gal), while it is also not weak in E-W direction (884gal). However, Case-3 (Takatori wave) is only strong in E-W direction (1133gal), but weak in U-D (261gal).

Thus properties of wave, and relative damage are ranked in Table 2. The plastic ratio is considered as the representation for the flexural vibration. Judged based on the same ranks, the U-D component controls the axial force variation of legs, while the E-W component dominates the flexural vibration of legs. However, the flexural vibration of girder was comprehensively affected by both components.

The effect of dominant modes is summarized in Table 3. For the displacement, all 3 cases show close result that the 1st and 2nd modes control the horizontal and vertical vibrations respectively. Considering the eigenvalue result mentioned in 3.2, it can be inferred that this effect is mainly from the natural vibration characteristics of the bridge itself. However, for the acceleration, the properties of seismic waves affect the vibration noticeably. Probably due to the strong U-D component in Case-2, the 2nd and 3rd modes, two typical vertical modes, dominate the horizontal acceleration in this case. But for the other two cases, the controlling modes for the acceleration roughly coincide with the result for the displacement.

In brief, Wolong wave influenced the bridge most severely due to the strongest U-D component and not weak E-W component. Although Takatori wave showed strongest E-W component, its influence was limited because of its weak U-D component. Thus, the waves with strong vertical component, Wolong and Bajiao waves by Wenchuan EQ, noticeably affected this rigid-frame arch bridge. Therefore, Xiaoyudong Bridge was probably damaged since the waves in Wenchuan EQ were relatively strong in U-D direction.

6. CONCLUSIONS

By dynamic analyses, discussion on damage, and effect by dominant modes, conclusions are drawn as:

- (1) The crucial points of this bridge were the joints between girder and arch legs at mid-span (failure mainly by flexure), and the bottoms of inclined legs (failure mainly by axial force variation).
- (2) The vibration in horizontal and vertical directions was mainly controlled by the 1st mode and the 2nd mode respectively, having limited relationship with different types of seismic wave.
- (3) Wave components in different directions had different effects on the damage of the bridge. U-D component totally controlled the axial force variation of legs, and caused their tensile failures. E-W component dominated the flexural vibration of legs. However, the plastic ratio of girder was influenced by both directions comprehensively.
- (4) Furthermore, thanks to strongest in U-D direction and not weak in E-W direction, Wolong wave (Case-3) gave most severe effect to both girder and leg. The other two waves gave different level



Fig. 18 Strength of input seismic wave

Table 2 Property wave & relative damage rank

	Property		Relative Damage Rank			
Rank	Wave Strength		Plastic Ratio		Axial Force	
	E-W	U-D	Girder	Legs	Legs	
Case-1	С	В	С	С	В	
Case-2	В	А	А	В	А	
Case-3	А	С	В	А	С	
Note: A: Strong/Severe: B: Medium: C: Weak/Slight						

Table 3 Evaluation of dominant modes

Modes	Displacement		Acceleration		
	Horizontal	Vertical	Horizontal	Vertical	
Case-1	1st	2nd&3rd	1st	2nd&3rd	
Case-2	1st	2nd	2nd&3rd	2nd&3rd	
Case-3	1st	2nd	1st	2nd	

of failure to different members, since they were strong in only one direction (Bajiao wave in U-D, Takatori wave in E-W). Therefore, strong U-D component in Wenchuan EQ was confirmed as a main reason of the failure of this bridge.

REFERENCES

- Kawashima, K., Takahashi, Y., Ge, H., Wu, Z., & Zhang, J., "Reconnaissance Report on Damage of Bridges in 2008 Wenchuan, China, Earthquake", J. of EQ Eng. Vol.13, JSCE, 2009, pp. 965-996
- [2] Kosa, K., Shi, Z., Zhang, J., & Shimizu, H., "Damage analysis of Xiaoyudong Bridge affected by Wenchuan Earthquake, China", J. Struct. Eng., Vol.57A, JSCE, 2011, pp. 431-441
- [3] Shi, Z., Kosa, K., Zhang, J., & Shimizu, H. "Seismic Damage Approach of a RC Rigid-Frame Arch Bridge Affected by Wenchuan Earthquake, 2008", Proc. of JCI Vol.33, No.2, 2011, pp. 829-834
- [4] Tasaki, K., Kosa, K., Ikeda, T., & Ogo, M., "Detailed Investigation of PC Cable-stayed Bridge (Ji-Ji-Da Bridge) Damaged to the Taiwan Chi-Chi Earthquake", J. Struct. Eng., Vol.50A, JSCE, 2004, pp. 487-494
- [5] Nippon Engineering Consultants Co., Ltd., "Manual of Dynamic Analysis", 2002
- [6] Japan Road Association, "Specifications for Highway Bridges Part V Seismic Design", 2002