Study on Seismic Constraint System of High-Pier Continuous Bridges

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SUMMARY:
Two kinds of constraint systems of the high-pier continuous bridges are analyzed: steel reinforced elastomeric rubber bearings plus sliding rubber bearings and pot rubber bearings. The analysis shows that in the first kind of constraint system, the displacements or deformations of bearings are hard to meet the seismic-resistant requirements, while in the second kind of constraint system, the seismic internal forces on the piers under the constraint bearings and the displacements of sliding bearings are excessive. Through the comprehensive consideration to the requirements of normal performance and seismic performance of bridges, the restrainers and the partial isolation bearings are used to optimize the restraint system. It is proposed that in the first kind of constraint system, the longitudinal and transverse restrainers should be used for the normal intensity seismic-resistant region, while in the second kind of constraint system, the isolation bearings should be used for the high intensity seismic-resistant region.

Keywords: Seismic constraint system, High-pier continuous bridges, Isolation bearing, Seismic restrainer

1. INTRODUCTION

In the past few years, with the rapid development of social economy, highway constructions have been built steadily up in the east of China, and have been gradually extended to the west of China. However, there are a lot of mountains and hills in the western region of China where the topographical and hydrological conditions are very complex. In order to cross the valleys, the high-pier continuous bridges (HPCB) have to be built inevitably. Moreover, the west of China is a region with high intensity earthquakes of high frequent occurrences. It is well known that bridges are the key element in the lifeline engineering. Once a bridge was damaged, it would bring the huge difficulties in the disaster salvage, even results in the sharp secondary disasters and greater economic loss. All of these have got a deep reflection in the earthquakes (EERI 1995; 2001, Fan and Li 2009). Therefore, the study on the seismic performance of HPCB is of great significance.
HPCB are widely used in Chinese highway because of the reasonable stress distribution, short construction period and low construction cost. Recently, some researches on seismic performance of HPCB have been carried out in China and it is found that the existing seismic design methods are not satisfactory for HPCB. Guidelines for Seismic Design of Highway Bridges-JTG/TB02-01-2008 (China) indicates that some special research on high-pier bridges (higher than 30m) should be made, however there is no specific or operable design method to solve this kind of problems. When the HPCB are designed, in general engineers blindly use the relative standards for the low-pier and medium-pier bridges. Based on the comprehensive analyses to two kinds of widely applied constraint systems in HPCB, two new constraint systems are proposed, which can effectively improve the seismic performance of HPCB and provide a reference for the seismic design of HPCB.

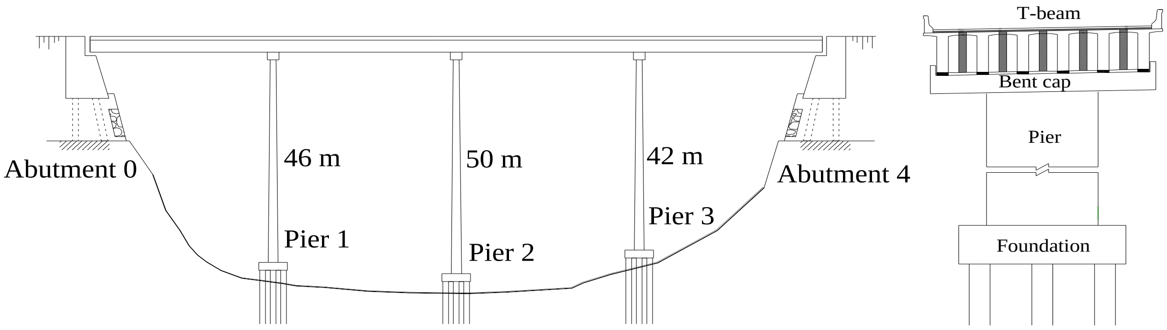
2. SEISMIC PERFORMANCE OF NORMAL CONSTRAINT SYSTEM FOR CONTINUOUS BRIDGES

Two limit cases should be considered in the constraint system for the simply supported-continuous bridges: the excessive temperature stress in girder and the requirement to the effective restraining on superstructure. Two schemes are usually applied in practice:

Scheme 1: steel reinforced elastomeric rubber bearings (SRERB) plus polytetrafluoroethene sliding rubber bearings (PTFE-SRB);

Scheme 2: fixed pot rubber bearings (fixed PRB) plus movable pot rubber bearings (movable PRB).

The seismic performance of two kinds of constraint systems for HPCB will be analyzed. A four-span bridge (4×40m) is shown in Fig.1. The simply supported-continuous prestressed T-beam is used as the superstructure which has the deck width 13m. The piers are made up of single-column concrete with rectangular variable cross-section. The cross-section of each pier at the top is 1.6m×6m and that at the bottom is 2m×6m. A square cross-section bent cap (2.2m×2.2m) is placed at the top of each pier. The foundation of the bridge is pile group.



(a) Geometric detail of the bridge (b) Cross-section at the pier

Figure 1. Skeleton diagram of HPCB

Considering the effective restraining to the superstructure, the links between high-piers and girder are generally designed into the rigid frame in HPCB. However, in practice, to let the rigid frame link come true is very difficult. Therefore, the constraint at each high-pier adopts the longitudinal constraint scheme using PRB. The arrangements of bearings in two kinds of normal constraint systems are shown in Table.1.

Table 1. Bearing arrangement in normal constraint system

Position	Scheme 1	Scheme 2
Abutment 0	6 SRB(350×450×86)(L)	1 PRB(L)+5 PRB(LT)
Pier 1	6 SRERB (500×650×110)	1 PRB(F)+5 PRB(T)
Pier 2	6 SRERB (500×650×110)	1 PRB(F)+5 PRB(T)
Pier 3	6 SRERB(500×650×110)	1 PRB(F)+5 PRB(T)
Abutment 4	6 SRB(350×450×86)(L)	1 PRB(L)+5 PRB(LT)

Note: (L) means the bearing movable in longitudinal direction, (T) means the bearing movable in transverse direction, (LT) means the bearing movable in both longitudinal and transverse directions, (F) means the fixed bearing. SRB means the sliding rubber bearings.

According to the process of bridge in China, SRERB are generally placed directly between the girder and the bearing padstone. There is no connection between the bearing and the bottom of the girder. When the earthquake-induced horizontal force is larger than the friction at the bottom of the girder, the relative sliding movement will occur at the interface. Therefore, the sliding friction element can be adopted to simulate the performance of SRERB. The relation between the lateral force and the lateral displacement of a bearing can be expressed by Eqn. 2.1 (Fan et al. 2003):

$$f_b = \begin{cases} k_b \cdot d_b & |f_b| \le F_{cr} \\ F_{cr} & |f_b| > F_{cr} \end{cases} \tag{2.1}$$

where f_b is the horizontal lateral force of the bearing; d_b is the horizontal lateral displacement of the bearing; k_b is the horizontal shear stiffness before the bearing slides; F_{cr} is the critical lateral sliding force of the bearing. $F_{cr}=N\cdot\mu$ in which, N is the bearing reaction during earthquake (including both dead load and earthquake excitation); μ is the sliding friction coefficient between the rubber bearing and the concrete, which is taken as 0.15 according to JTG/T B02-01-2008 (China).

In order to make the general analysis, three earthquake waves provided by Pacific Earthquake Engineering Research Center (PEER) are used in the analysis, based on the different ground motion parameters (such as magnitude, peak ground acceleration (PGA) and site characteristic etc.). Bridges in normal intensity seismic-resistant region and bridges in high intensity seismic-resistant region are taken into account, respectively. Besides, according to the design earthquake and the strong seismic action, The PGA of every earthquake wave has been adjusted appropriately. The bridge system is subjected to the combinational excitation: 100% longitudinal earthquake ground motion and 60% vertical earthquake ground motion. Unless there is special explanation, the seismic responses are taken as the average of the results calculated from the above three seismic waves. Dynamic responses of bridges from incremental dynamic analysis with respect to different intensity seismic actions are shown in Tables 3~6.

Table 2. Information of three earthquake waves

Serial	Earthquake wave	Component of earthquake	Magnitude	Site characteristic	PGA (g)
1	1940 El Centro	Imperial Valley C270	6.9	III	0.215
2	1995 Kobe	Takarazuka TAZ090	6.9	IV	0.694
3	1999 Chi-Chi	Taiwan CHY006-E	7.6	II	0.364

Table 3. Seismic response of bridge with constraint system of Scheme 1 (PGA is 0.1g)

Serial	Seismic action in longitudinal+vertical directions			Seismic action in transverse+vertical directions		
	Shear (kN)	Moment (kN m)	Relative displacement between pier and girder (m)	Shear (kN)	Moment (kN m)	Relative displacement between pier and girder (m)
0	/	/	0.10	/	/	0.08
1	731	14251	0.03	1364	41733	0.08
2	863	14785	0.03	1311	41473	0.08
3	811	15683	0.05	1448	43454	0.07
4	/	/	0.10	/	/	0.07

Table 4. Seismic response of bridge with constraint system of Scheme 1 (PGA is 0.2g)

Serial	Seismic action in longitudinal+vertical directions			Seismic action in transverse+vertical directions		
	Shear (kN)	Moment (kN m)	Relative displacement between pier and girder (m)	Shear (kN)	Moment (kN m)	Relative displacement between pier and girder (m)
0	/	/	0.16	/	/	0.17
1	1503	22787	0.13	2824	95104	0.18
2	1768	25513	0.13	2535	84631	0.16
3	1326	26591	0.15	2971	91111	0.14
4	/	/	0.16	/	/	0.12

Table 5. Seismic response of bridge with constraint system of Scheme 2 (PGA is 0.2g)

Serial	Seismic action in longitudinal+vertical directions			Seismic action in transverse+vertical directions		
	Shear (kN)	Moment (kN m)	Relative displacement between pier and girder (m)	Shear (kN)	Moment (kN m)	Relative displacement between pier and girder (m)
0	/	/	0.21	/	/	/
1	2350	43276	/	3462	119376	/
2	2042	34578	/	4972	157087	/
3	2157	43881	/	3602	129634	/
4	/	/	0.21	/	/	/

Table 6. The maximum relative displacements between abutment and girder or between pier and girder (PGA is 0.4g）

Serial	Scheme 1		Scheme 2	
	Longitudinal	Transverse	Longitudinal	Transverse
0	0.43	0.32	0.42	/
1	0.43	0.36	/	/
2	0.44	0.33	/	/
3	0.46	0.33	/	/
4	0.43	0.28	0.42	/

It is shown from Table 3 that under the design seismic action in normal intensity seismic-resistant region (PGA is 0.1g), the forces on piers under longitudinal-vertical seismic actions and transverse-vertical seismic actions are not large. Moreover, the deformations of SRERB (Maximum deformation capacity is 0.08m) and the displacements of SRB (Maximum displacement capacity is 0.1m) meet the seismic-resistant requirements. However, it can be seen from Table 4 that under the rare seismic action (PGA is 0.3g) in normal intensity seismic-resistant region, the maximum relative displacements between girder and substructure have exceeded the deformation capacities of the bearings. Namely, SRERB slide occurs during the earthquake excitation. Once the SRERB slide happens, the bearings cannot effectively restrain the displacement of the girder. The excessive displacements could cause the bearing damage and the collision damage at the ends of girder and abutments. Table 5 shows that in high intensity seismic-resistant region (PGA is 0.2g), under the longitudinal-vertical and transverse-vertical seismic actions the piers used Scheme 2 can satisfy the demands on carrying capacity. Moreover, the displacements in all PRB (Maximum displacement capacity is 0.25m) can meet the seismic-resistant requirements. Comparing the data in Table 4 and Table 5, it can be found that the under seismic action (PGA is 0.2g) the pier forces in Scheme 1 are obviously smaller than those in Scheme 2. The main reason is that the SRERB slide can effectively reduce the seismic response under the high intensity seismic action. As shown in Table 6, under the rare seismic action (PGA is 0.4g) the maximum relative displacement between girders and abutments in Scheme 2 reaches 0.42m in the high intensity seismic-resistant region, which is far beyond the displacement capacity of PRB. This may cause the bearing damage and the collision damage. Also, because of the severe slide of SRERB, the superstructure cannot be restrained effectively. Therefore, SRERB are not suitable for use in the high intensity seismic-resistant region.

3.THE SEISMIC PERFORMANCE OF OPTIMIZED CONSTRAINT SYSTEM FOR HPCB

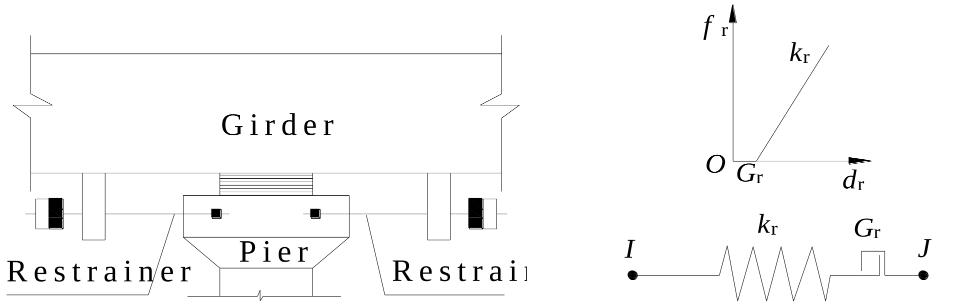
In order to improve the seismic performance of the above two kinds of constraint systems for HPCB, it is proposed to optimize these two constraint systems.

3.1 Anti-drop-beam device

In order to limit the excessive relative displacement caused by the slide of SRERB under the high intensity seismic action, a constraint system composed of SRERB and anti-drop-beam device is applied. Namely, the seismic restrainers in longitudinal and transverse directions are set in the bridge and the cable restrainers are set at both ends of the abutments for connecting girders and abutments and at both sides of each pier for connecting piers and girder, as shown in Fig. 2(a). Assuming the cable restrainer is a tension element and in the elastic state during the earthquake excitation. The spring-hook element is adopted to simulate the restrainer, as shown in Fig. 2(b). The nonlinear relation between the tension and the restrainer displacement is expressed in Eqn. 3.1 (Huang et al. 2009):

$$f_r = \begin{cases} k_r\left(d_r - G_r\right) & d_r - G_r > 0 \\ 0 & d_r - G_r \le 0 \end{cases} \tag{3.1}$$

where f_r is the tension of the restrainer; d_r is the relative displacement between point I and point J; G_r is the relaxation length of the restrainer under the consideration of the effect of the temperature deformation at the bearings, which is taken as 0.06m; k_r is the stiffness of the restrainer. There are few studies on the restrainer stiffness (i.e. the stiffness of the pier-girder connection). The stiffnesses of the restrainers at abutments and piers are, respectively, taken as 2×10^5kN/m and 1×10^5kN/m in our analysis.

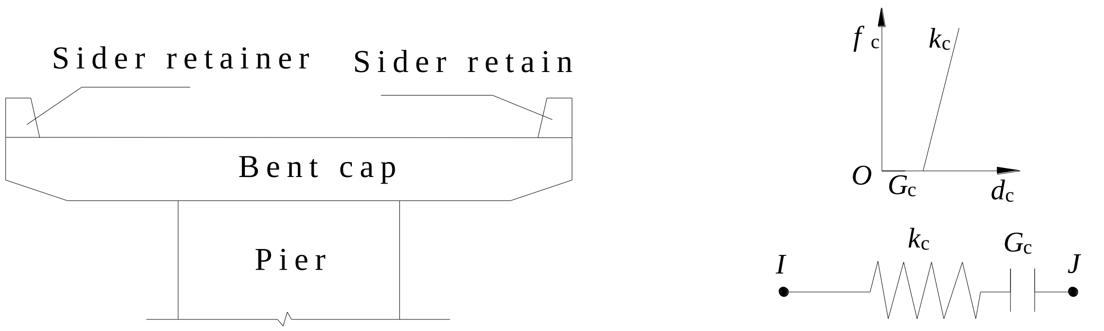


(a)Constitution of the restrainer (b) Contact element of the restrainer
Figure 2. The restrainer

In order to limit the transverse displacement of the girder, the concrete retainers are set at the both sides of bent caps and abutments, as shown in Fig. 3(a). The transverse retainer is simulated by the contact element shown in Fig. 3(b). The nonlinear relation between pressure and displacement is expressed in Eqn. 3.2 (Huang et al. 2009):

$$f_c = \begin{cases} k_c\left(d_c + G_c\right) & d_c + G_c \le 0 \\ 0 & d_c + G_c > 0 \end{cases} \tag{3.2}$$

where f_c is the impact force of the contact element; d_c is the relative displacement between point I and point J; G_c is the initial clearance between the transverse retainer and the girder under the consideration of the effect of the restrainer on the deformation of the bearing, which is taken as 0.06m; k_c is the stiffness of the contact element, which is taken as 3×10^5kN/m in the analysis.



(a)Constitution of the transverse retainer (b)Contact element of the transverse retainer
Figure 3. The transverse retainer

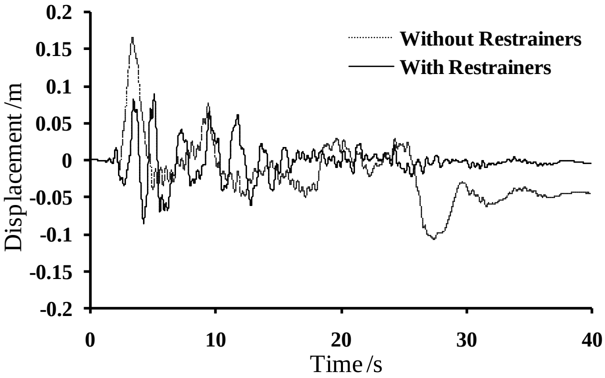


Figure 4. Time-history of the longitudinal relative displacement of pier-girder at Pier 2 under El Centro wave (PGA is 0.2g)

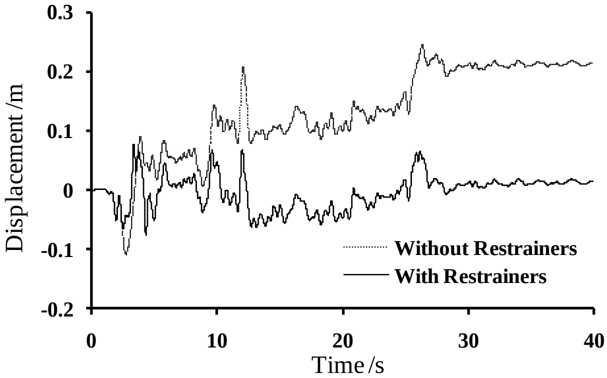


Figure 5. Time-history of the transverse relative displacement of pier-girder at Pier 2 under El Centro wave (PGA is 0.2g)

The time-history comparisons of longitudinal relative displacements and transverse relative displacements of pier-girder at Pier 2 with/without restrainers under El Centro wave are shown in Fig. 4 and Fig. 5. It is seen from Fig. 4 and Fig. 5 that under the high intensity seismic action, the longitudinal and transverse deformations of SRERB with anti-drop-beam device is larger than that without anti-drop-beam device (composed of the longitudinal displacement restrainer and the transverse retainer). However, the relative displacements between the piers and the girder decrease obviously and the sliding displacements of SRERB have been controlled effectively when the anti-drop-beam device is applied. Moreover, the relative displacements between the girder and the substructure in other positions could also be controlled, same as those through the anti-drop-beam device. Fig. 7 shows the maximum relative displacement ratio at pier-girder or at abutment-girder, with/without anti-drop-beam device, when PGA is 0.2g. It can be seen from Fig. 7 that the relative displacements between the girder and the substructure with anti-drop-beam device decrease obviously (the minimum decrease is near 20% and the maximum decrease reaches 50%), compared to the case without anti-drop-beam device. Therefore, the proposed constraint system could effectively prevent beam from dropping and improve the seismic performance of the bridge.

Table 7. The maximum relative displacement ratio at pier-girder or at abutment-girder, with/without anti-drop-beam device (PGA is 0.2g)

Seismic direction	Abutment 0	Pier 1	Pier 12	Pier 3	Abutment 4
Longitudinal-vertical	0.82	0.51	0.60	0.37	0.80
Transverse-vertical	0.43	0.42	0.46	0.53	0.63

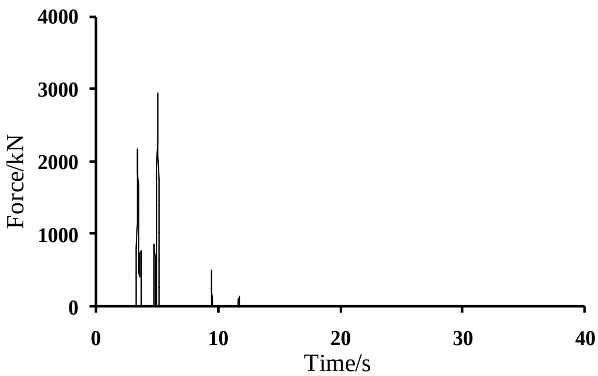


Figure 6. The time-history of the tension of the left restrainer at Pier 2 under the longitudinal-vertical El Centro wave (PGA is 0.2g)

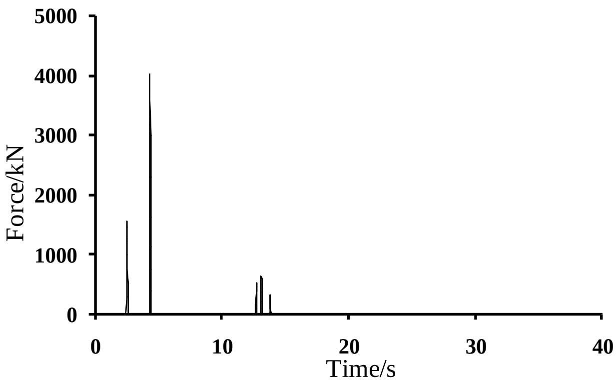


Figure 7. The time-history of pressure of the left retainer at Pier 2 under the transverse-vertical El Centro wave (PGA 0.2g)

Fig. 6 and Fig. 7 show the time-histories of the forces on restrainers and retainers at Pier 2 under the El

Centro wave. It can be seen from Fig. 6 and Fig. 7 that both the restrainer and the retainer suffer the discontinuous impulsive forces during the earthquake, when the relative displacement exceeds the initial clearance of the restrainer or retainer. Moreover, the maximum seismic forces of the restrainers and the retainers when PGA is 0.2g are presented, respectively, in Table 8 and Table 9. Under the high intensity seismic action, the restrainers and transverse retainers in every position are subjected to large seismic forces, especially in the abutments. It has been confirmed that the larger relative displacements could occur at the abutments, compared to those at the piers during earthquakes (Wang et al. 2005). In order to limit the relative displacements, the corresponding devices have to undertake the larger forces.

Table 8. The maximum tension of restrainers under the longitudinal-vertical seismic action (PGA is 0.2g)

Maximum tension of restrainers (kN)							
Abutment 0	Left of Pier 1	Right of Pier 1	Left of Pier 2	Right of Pier 2	Left of Pier 3	Right of Pier 3	Abutment 4
12321	599	1400	1877	2015	385	0	13215

Table 9. The maximum forces of retainers under the transverse-vertical seismic action (PGA is 0.2g)

Maximum force of retainers (kN)				
Abutment 0	Pier 1	Pier 2	Pier 3	Abutment 4
3313	2784	3382	2875	3782

The maximum shear ratio and the maximum moment ratio of every pier with/without anti-drop-beam device when PGA is 0.2g are shown in Table 10. It is seen from the table that the effectiveness of the isolation from the SRERB slide is weakened after the anti-drop-beam device is applied. Furthermore, the seismic response of the substructure increases obviously because the larger inertia force of the girder has been transmitted to the substructure through the anti-drop-beam device. The maximum values of shear and moment at the bottom of piers increase 1.89 times and about 2 times, respectively. Thus, the seismic damage form of the bridge can be controlled, and the ineffective exertion of seismic-resistant ductile performance of the pier can be prevented due to the use of the drop-beam. Moreover, the amplification effection from the anti-drop-beam device on seismic responses of the bridge should be considered sufficiently in the seismic design of the piers.

Table 10. The maximum shear ratios and the moment ratios of the piers with/without anti-drop-beam device (PGA is 0.2g)

Pier	Longitudinal-vertical seismic action		Transverse-vertical seismic action	
	Shear ratio	Moment ratio	Shear ratio	Moment ratio
1	1.51	1.64	1.89	1.14
2	1.71	2.03	2.05	1.36
3	1.28	1.30	1.61	0.98

3.2 Partial isolation system

Based on the results of the above analysis, it can be demonstrated that the natural period of HPCB is usually relative large due to the high piers which may lead to the smaller anti-push rigidity of the substructure and the bigger flexibility of the structure. Moreover, the displacement responses of HPCB are much larger than those of the normal-pier continuous bridges under earthquake. This may result in the method of extending the structure period to reduce the seismic response unsuitable for the seismic design of HPCB. The design specification of Japan high-way bridge (explanation) (Institute of Japan Highway 2001) stipulates that if the fundamental period of a bridge (when the piers and girders are fixed) is more than 1 second, the existing seismic mitigation and isolation system will be no longer suitable for this type of bridges. The fundamental period of the bridge shown in Fig. 1 is 1.15 second when the piers and girders are fixed, which is already beyond the scope of 1 second. In Scheme 2 (i.e. the constraint scheme of PRB), in order to reduce the seismic response forces at the fixed piers and the relative displacements between the girder and the substructure at abutments, a partial isolation constraint system is proposed which can effectively enhance the seismic performance of HPCB. In the proposed constraint system, the seismic performance of the structure can be greatly increased by consuming the earthquake energy and fully exerting the seismic capacities of piers and abutments. The detail for the proposed constraint system is based on the Scheme 2: six PRB in Abutments 0-4 and Piers 1 and 3 are replaced by LRB with the diameter 600mm. The core diameter of the lead is 12cm in the LRB while the constraint at Pier 2 is the same as that used in Scheme 2.

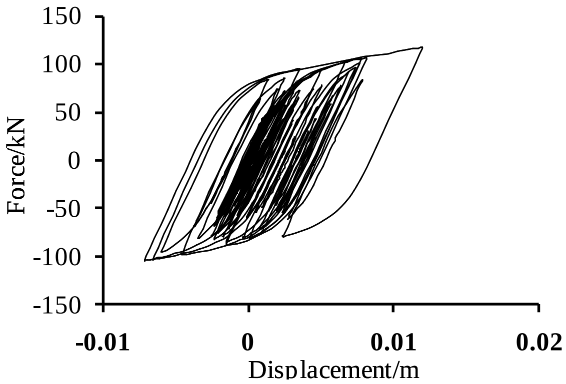


Figure 8. Hysteretic curve of LRB at Pier 1 under longitudinal-vertical El Centro wave (PGA is 0.2g)

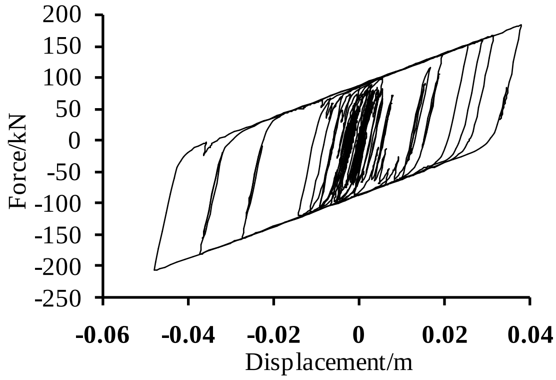


Figure 9. Hysteretic curve of LRB at Pier 1 under transverse-vertical El Centro wave (PGA is 0.2g)

Fig. 8 and Fig. 9 show the longitudinal and transverse hysteretic curves of LRB at Pier 1 subjected to the El Centro wave. It is clear from the figures that the LRB has already yielded under the longitudinal-vertical or transverse-vertical seismic actions when PGA is 0.2g. The area of the hysteretic curve represents the magnitude of the energy dissipation, therefore LRB has dissipated energy under the seismic action (PGA is 0.2g). The maximum shear ratios and maximum moment ratios of piers between the partial constraint system and the PRB constraint system in Scheme 2 are shown in Table 11. It can be seen from the table that the maximum shears and maximum moments of piers with partial constraint system are decreased because of the energy dissipation from LRB (the minimum decrease of both shear and moment is near 30%).

Table 11. The maximum shear ratios and moment ratios of piers between the partial constraint system and the Scheme 2 (PGA is 0.2g)

Pier	Longitudinal-vertical seismic action		Transverse-vertical seismic action	
	Maximum shear ratio	Maximum moment ratio	Maximum shear ratio	Maximum moment ratio
1	0.71	0.70	0.77	0.75
2	0.99	0.86	0.75	0.83
3	0.77	0.72	0.80	0.67

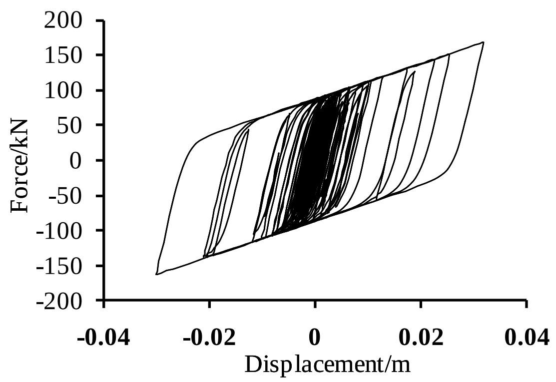


Figure 10. Hysteretic curve of LRB at Pier 1 under longitudinal-vertical El Centro wave (PGA is 0.4g)

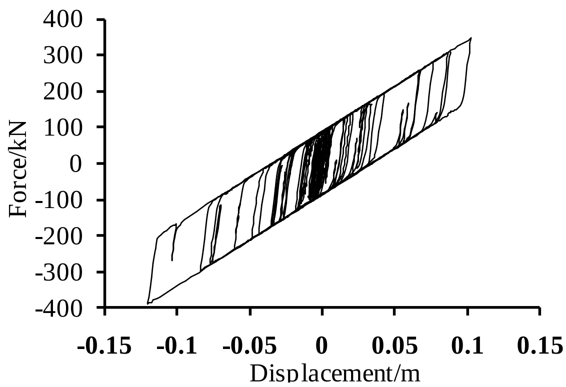


Figure 11. Hysteretic curve of LRB at Pier 1 under transverse-vertical El Centro wave (PGA is 0.4g)

Fig. 10 and Fig. 11 show the longitudinal and transverse hysteretic curves of LRB at Pier 1 under El Centro wave when PGA is 0.4g. It can be seen from the figures that with the rising of seismic action, the maximum deformation of LRB increases and the area of the bearing hysteretic curve fill out. This means the energy dissipation is larger. Besides, it can be clearly seen from Fig. 8 to Fig. 11 that the area of the hysteretic curve of LRB under the transverse-vertical earthquake is larger than that under the corresponding longitudinal-vertical earthquake. The main reason is that the longitudinal fundamental period of the bridge is obviously less than that in the transverse direction. In general, the effect of the isolation is obviously better in the short-period structure. The maximum relative displacements between the girder and the corresponding substructure in partial constraint system (PGA is 0.4g) are shown in Table.12. The maximum longitudinal relative displacement in abutments decreased near 30% (from 0.42m to 0.30m), compared with the maximum relative displacement of the bridge with the constraint system of Scheme 2 in Table 6. Moreover, the relative displacements between the girder and the corresponding substructure are small. However, it should be considered that the sufficient space in abutments should be previously leaved to allow the full deformation of the bearings under the longitudinal-vertical seismic action when the bridge is designed.

Table 11. The maximum relative displacements in partial constraint system in pier-girder or abutment-girder (PGA is 0.4g)

Seismic direction	The maximum relative displacements between piers and girders or between abutments and girder				
	Abutment 0	Pier 2	Pier 2	Pier 3	Abutment 4
Longitudinal-vertical	0.30	0.03	/	0.04	0.30
Transverse-vertical	0.20	0.14	/	0.16	0.18

4. CONCLUSION

The seismic performance of HPCB has been studied. The result shows that the normal constraint system hardly satisfies the seismic-resistant requirements of HPCB. Based on the comprehensive consideration on normal use and the seismic performance requirements of bridges, the restrainers or the partial isolation bearings are applied to enhance the seismic performance of HPCB. The following conclusions are made:

(1) In the normal intensity seismic-resistant region, using the SRERB plus PTFE-SRB constraint system for HPCB can satisfy the seismic performance requirements under the design earthquake, however the slides of bearings could occur under the strong earthquake. Therefore, the seismic displacement of the superstructure cannot be effectively restrained, and the large displacement will cause the collision damage.

(2) The SRERB constraint system for HPCB with anti-drop-beam device can effectively decrease the excessive relative displacement caused by the SRERB slide between the girder and the substructure. It prevents the collided girder from the falling damage and satisfies the seismic performance of the bridge in normal intensity seismic-resistant region. However, the seismic responses of the piers increase and the device suffers the larger earthquake-induced force at abutments after the anti-drop-beam device is applied. This amplificative effect should be considered in the bridge design.

(3) The PRB constraint system for HPCB in high intensity seismic-resistant region can satisfy the seismic performance requirements under the design earthquake. However, both the seismic internal force and the displacements of piers with SRB constraint bearings are large, which can hardly satisfy the seismic-resistant requirements under the high intensity seismic action. Moreover, the seismic performance of the SRERB constraint system for HPCB becomes worse in the high intensity seismic-resistant region, which is not suitable for use in the case.

(4) According to the characteristics of HPCB system, combined with the existing seismic mitigation and isolation method, the partial isolation constraint system is proposed, which is a combination of the fixed constraint system and the seismic isolation bearings. The analytical results show that the partial isolation design applied in HPCB not only can significantly reduce the force of piers but also can

reduce the displacement response of the structure, which is suitable for HPCB in the high intensity seismic-resistant region.

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