The study on the anti-pounding damper used in the high-piers bridge

MENG Qingli
Key Laboratory of Earthquake Engineering and Engineering Vibration，Institute of Engineering Mechanics, CEA,China Earthquake Administration, Harbin 150080, China

YE Jian
Key Laboratory of Earthquake Engineering and Engineering Vibration，Institute of Engineering Mechanics, CEA,China Earthquake Administration, Harbin 150080, China

SUMMARY:
In this paper, the anti-pounding dampers were installed in longitudinal and transverse directions of a high-pier bridge's pounding locations (expansion joints), under multi-excitation considering the travelling wave effect, doing comparative analysis of the seismic response of the bridge with high-pier between with and without the anti-pounding damper which is set on pounding location. anti-pounding damper can be effective in preventing pounding between deck beams and between deck beam and block, and the displacements of the bridge piers is limited, internal forces of piers' ends greatly reduced, reducing the damage to piers, which will help to improve the overall seismic performance; but setting pounding damper would also adversely affect the bridge with high-pier, increasing the possibility of damage to the deck beams, uncertainty of bearing reaction**.**

Keywords: high-pier Bridge; Anti-pounding damper; traveling wave effect

1. GENERAL INSTRUCTIONS

Earthquake may generate structural poundings of bridge and bridge plays a vital role in traffic net. Structural pounding mainly take place at expansion joint between bridge girders or between girder and abutment, which could results in cracks of the end of girder, concrete shedding from parapet of abutment, bearing's damage or even girder's falling (Meng Qingli 2003, Wang Junwen 2006). Structural damage from bridge's pounding does not only cause casualties and economic losses directly but also exerts severe adverse effects on rescuing work and reconstruction after the earthquake.

Because Midwest of China is a mountainous and hilly region, high-pier is widely used in bridge construction, which is enslaved to the terrain there. There are significant differences among the piers' heights of high-pier bridge, so dynamic characteristics and vibration periods of adjacent spans vary remarkably from each other. The differences of vibration periods of adjacent spans are considered to be main factor influencing seismic pounding of continuous beam bridge, and the larger the difference of vibration periods is, the more obvious the collisional effect is (Wang Junwen 2006, Reginald D 2002). Midwest of China is a high incidence area of earthquake. Chinese still suffer from Wenchuan earthquake on 12th May 2008. During the earthquake, the stoppers in Miao Ziping bridge is broken, which resulted from longitudinal and transverse movement and pounding and resulted into falling of one span approaching bridge. The main girder in Bai Shuixi bridge had about 50cm transverse shift, then pounding resulted into stoppers' damage (Meng Qingli 2010).

Two kinds of measures can be taken to avoid or reduce bridge's damage caused by seismic pounding, the first is to increase the distance between girders to avoid pounding, and the second is to install some kinds of seismic response damping devices to reduce damage from pounding. In this paper, the second measure is taken and an elementary study on anti-pounding measure is presented. Restrainer, damper and buffer are common devices for reducing earthquake response of bridges. Damping is some kind of friction and hinders force which leads to vibration attenuation, and damper is a sort of device which provides resistances and dissipates energy. Adding dampers to structure efficiently is very helpful to improve structure's performance under earthquake action. Now bridge engineers at home and abroad have got to a common view that it must be considered to use dampers in bridge engineering. In Occident, Japan, New Zealand etc countries and regions, dampers must be considered to use in aseismic and wind resistant design of bridge. Adding viscoelastic dampers between adjacent spans could improve bridge's dynamic performance, reduce the possibility of pounding and cut down longitudinal displacement of spans. Viscoelastic damper combined with rigid spring can form a buffer device, and it can reduce pounding efficiently. Bridge's seismic pounding could be controlled by using magnetorheological damper (Meng Qingli 2010, Meng Qingli 2011).

2. MODELING OF HIGH-PIER BRIDGE AND ITS FREE VIBRATION CHARACTERISTICS

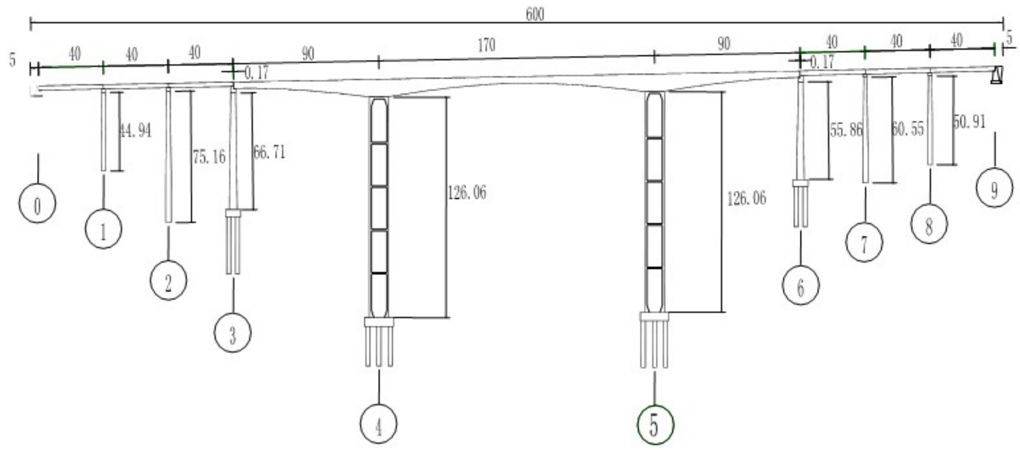

Figure 1. The high-pier bridge

The open source software OpenSees is adopted to create a finite element model of high-pier bridge in this paper. As shown in **Figure 1**, the high-pier bridge used in this paper is a concrete continuous rigid-frame bridge with overall length of 600m and width of 12m, and its main bridge consists of three spans, 90m+170m+90m. Each end of the bridge is a partial prestressed concrete continuous simple supported approach span of 3×40m. The pier of approach spans is double-column and adopts C40 concrete. The juncture pier is a double-column hollow thin-walled pier with a L-shaped capping beam, it also is C40 concrete, the transverse and longitudinal widths of its top are 2.5m and 3m, and the longitudinal width increases from top to base according to the slope gradient of 70:1. The main pier is a hollow thin-walled reinforced concrete pier, its thickness is 80cm, the transverse width of its top is the same as the bottom width of the box girder above, like the juncture pier, the slope gradient is 63:1, and C40 concrete is adopted.

There are 4 expansion joints all over the bridge, 2 of them are the steel expansion joints SSFB240, which locate on the top of juncture pier of main and approach span, and their width is 17cm. The other 2 are SSFB80, 10cm wide, and lie in the abutments. Three kinds of bearings are used in this bridge: laminated rubber bearing (which allowable deformation is 0.046m), PTEF sliding bearing (which allowable deformation is 0.09m in longitudinal direction and 0.04m in transverse direction) and pot rubber bearing (which allowable deformation is 0.2m).

The finite element model of the bridge is created with the open source software OpenSees. In analysis of seismic pounding, the pounding element of expansion joint is the 3D contact-friction pounding model which is a secondary development work based on OpenSees (Meng Qingli 2011). The dampers are added in the pounding location, their longitudinal stiffness and damping are $K=3.4\times10^{8}$N/m and $Cn=1.08\times10^{7}$Ns/m, and their transverse stiffness and damping are $K=3.4\times10^{8}$N/m and $Cn=1.62\times10^{7}$Ns/m.

The natural vibration characteristics of the high-pier bridge with dampers are shown below.

Table 2.1. The first three natural periods of the high-pier bridge (s)

No.	With dampers		Without dampers	
	Longitudinal	Transverse	Longitudinal	Transverse
1	2.40	3.90	2.72	4.09
2	1.78	2.65	2.21	2.71
3	1.02	1.78	1.42	2.37

It is adding anti-pounding dampers that makes the main and approach spans vibrating together instead of spans vibrating alone. As shown in **Table 2.1**, the periods of high-pier bridge are reduced.

3. CONTRAST ANALYSIS ON HIGH-PIER BRIDGES WITH AND WITHOUT ANTI-POUNDING DAMPERS

In the 3D numerical simulation analysis of seismic pounding and anti-pounding of the high-pier bridge, three types of input ground motions: El Centro wave, Parkfield wave, Northridge wave and Wolong wave.

3.1. Comparison of axial forces of beams with and without dampers

Table 3.1. Axial forces of deck beams ($\times 10^7$N)

Location		El Centro		Northridge		Parkfield		Wolong	
		Without damper	With damper	Without damper	With damper	Without damper	With damper	Without damper	With damper
Left approach bridge	Left abutment	0.33	0.36	1.02	1.27	1.75	2.18	2.02	2.4
	Span center of	0.29	0.32	0.68	0.69	1.12	1.81	1.92	2.2
	Left juncture pier	0.42	0.52	0.98	1.26	1.24	1.51	1.97	3.4
Main bridge	Left juncture pier	0.35	0.40	0.68	1.20	1.26	1.49	1.51	1.94
	Span center	0.35	0.56	0.60	1.32	1.06	1.86	1.23	2.7
	Right juncture	0.61	1.14	0.76	1.57	2.06	2.2	1.35	2.2
Right approach bridge	Right juncture	0.57	0.80	0.97	1.40	1.97	3.21	1.36	2.3
	Span center of	0.45	0.47	0.85	1.02	2.03	3.09	1.95	3.9
	Right abutment	0.68	0.73	0.93	1.50	2.38	2.81	1.36	2.1

After adding dampers, axial forces of beams under the 4 earthquake waves all increase compared with forces without dampers. The variation range of axial forces of beams under El Centro wave is 9%~87%, which includes: the variation range of the approach bridge is 9%~40% and the variation range of the main bridge is 14%~87%. The variation range of axial forces of beams under Northridge wave is 24%~120%, which includes: the variation range of the approach bridge is 24%~61% and the variation range of the main bridge is 76%~120%. The variation range of axial forces of beams under Parkfield wave is 10%~90%, which includes: the variation range of the approach bridge is 10%~63% and the variation range of the main bridge is 20%~90%. The variation range of axial forces of beams under Wolong wave is 19%~140%, which includes: the variation range of the approach bridge is 19%~72% and the variation range of the main bridge is 28%~120%.

The axial forces of beams increase sharply after adding dampers, and the leading cause is the different natural vibration periods of approach and main bridges. To add a damper is equivalent to add an element with stiffness and damping, which unites approach and main bridges. Approach and main bridges with different natural vibration periods influence each other, which results into deformations of dampers, so the axial forces of adjacent beams are changed. The forces may be beyond the forces without dampers and the variation range of the main bridge is larger than the approach bridge’s.

Table 3.2. Longitudinal accelerations of beams (m/s^2)

Location		El Centro		Northridge		Parkfield		Wolong	
		Without damper	With damper	Without damper	With damper	Without damper	With damper	Without damper	With damper
Left approach	Left abutment	2.60	2.00	8.0	0.87	8.7	3.92	7.0	3.99
	Span center of middle	1.30	1.10	2.33	1.51	3.10	1.3	4.28	2.43
	Left juncture pier	2.56	1.90	6.00	1.29	6.84	2.15	7.6	4.87
Main bridge	Left juncture pier	15.0	2.73	17.7	1.28	18.0	3.30	14.8	4.95
	Span center	6.53	3.06	8.7	1.24	9.0	1.78	7.69	6.27
	Right juncture pier	14.0	2.57	19.3	1.27	19.7	3.98	16.1	4.99
Right approach bridge	Right juncture pier	5.15	1.45	5.4	1.64	10.0	2.36	6.8	5.13
	Span center of middle	2.32	1.60	3.4	1.69	5.68	2.99	3.67	2.12
	Right abutment	4.11	1.75	7.7	0.94	11.9	3.28	9.6	4.00

After adding dampers, accelerations of beams under the 4 earthquake waves all reduce compared with forces without dampers. The variation range of accelerations of beams under El Centro wave is -23%~-81%, which includes: the variation range of pounding location of the approach bridge is -23%~-72%, the range of no pounding location is -20%~-31% and the variation range of pounding location of the main bridge is about -81%, the range of no pounding location is about -53%. The variation range of accelerations of beams under Northridge wave is -35%~-93%, which includes: the variation range of pounding location of the approach bridge is -70%~-80%, the range of no pounding location is -35%~-50% and the variation range of pounding location of the main bridge is about -93%, the range of no pounding location is about -86%. The variation range of accelerations of beams under Parkfield wave is -47%~-81%, which includes: the variation range of pounding location of the approach bridge is -55%~-72%, the range of no pounding location is -47%~-58% and the variation range of pounding location of the main bridge is about -81%, the range of no pounding location is about -75%. The variation range of accelerations of beams under Wolong wave is -20%~-69%, which includes: the variation range of pounding location of the approach bridge is -43%~-58%, the range of no pounding location is about -20% and the variation range of pounding location of the main bridge is about -69%, the range of no pounding location is about -43%.

Peak accelerations decrease after adding dampers. Earthquake energy is expended by damping in dampers, so peak accelerations at each position are reduced, and the increases of accelerations caused by pounding are reduced too, so pounding is restrained. The variation range of accelerations of beams of main bridge is larger than approach bridge’s, and girder end’s is larger than center’s.

3.2. Comparison of bearing’s peak distortion with and without dampers

Table 3.3. The peak distortion of the bearing (m)

Direction	Bearing type	location	El Centro		Northridge		Parkfield		Wolong	
			Without damper	With damper	Without damper	With damper	Without damper	With damper	Without damper	With damper
longitudinal	PTEF sliding bearing	Left abutment	0.16	0.08	0.13	0.05	0.16	0.23	0.32	0.14
		Left juncture	0.25	0.38	0.27	0.14	0.35	0.33	0.22	0.19
		Right juncture	0.22	0.51	0.26	0.11	0.21	0.51	0.24	0.22
		Right	0.22	0.11	0.16	0.05	0.47	0.24	0.32	0.14
	Pad elastomeric bearing	Pier No 1	0.08	0.06	0.19	0.18	0.17	0.27	0.15	0.13
		Pier No 2	0.05	0.04	0.19	0.12	0.12	0.25	0.22	0.11
		Pier No 7	0.05	0.05	0.15	0.12	0.17	0.26	0.16	0.11
		Pier No 8	0.09	0.06	0.19	0.22	0.24	0.33	0.14	0.13
	Pot bearing	main bridge left	0.16	0.37	0.19	0.13	0.15	0.32	0.16	0.19

		main bridge right	0.16	0.45	0.19	0.12	0.16	0.52	0.18	0.23
transverse	PTEF sliding bearing	Left abutment	0.04	0.03	0.024	0.006	0.012	0.001	0.027	0.012
		Left juncture	0.39	0.027	0.19	0.001	0.04	0.000	0.22	0.003
		Right juncture	0.26	0.020	0.24	0.001	0.07	0.000	0.27	0.004
		Right	0.05	0.016	0.03	0.007	0.027	0.001	0.04	0.012
	Pad elastomeric bearing	Pier No 1	0.07	0.055	0.09	0.077	0.028	0.074	0.08	0.092
		Pier No 2	0.07	0.087	0.08	0.108	0.035	0.066	0.059	0.124
		Pier No 7	0.07	0.1	0.13	0.151	0.05	0.098	0.059	0.149
		Pier No 8	0.08	0.078	0.12	0.057	0.04	0.059	0.05	0.093

The peak distortion of every bearing is changed after adding dampers. When the ground motion is El Centro wave, the variation ranges of longitudinal and transverse peak distortions of PTEF sliding bearing are -50%~134% and -25%~-93%, the variation ranges of longitudinal and transverse peak distortions of pad elastomeric bearing are 0%~-30% and -3%~43% and the variation range of peak distortion of pot bearing is 130%~178%. When the ground motion is Northridge wave, the variation ranges of longitudinal and transverse peak distortions of PTEF sliding bearing are -50%~-70% and -75%~-99%, the variation ranges of longitudinal and transverse peak distortions of pad elastomeric bearing are -16%~35% and 53%~35% and the variation range of peak distortion of pot bearing is -31%~-36%. When the ground motion is Parkfield wave, the variation ranges of longitudinal and transverse peak distortions of PTEF sliding bearing are -50%~140% and -92%~-100%, the variation ranges of longitudinal and transverse peak distortions of pad elastomeric bearing are 39%~109% and -48%~-64% and the variation range of peak distortion of pot bearing is 111%~221%. When the ground motion is Wolong wave, the variation ranges of longitudinal and transverse peak distortions of PTEF sliding bearing are -7%~-57% and -55%~-99%, the variation ranges of longitudinal and transverse peak distortions of pad elastomeric bearing are -8%~-50% and 15%~50% and the variation range of peak distortion of pot bearing is 20%~27%.

There is no distinct regularity in bearing’s longitudinal distortions of main and approach bridges after adding dampers. Dampers connect approach bridge, main bridge and abutment, spans with different natural vibration periods influence each other, so there is no regularity in bearing’s distortions. The transverse distortion of PTEF sliding bearing of approach bridge is reduced sharply, for the transverse dampers are located between approach bridge and juncture pier or abutment, which restrains the relative displacement between approach bridge and juncture pier or abutment and controls poundings. There is no distinct regularity in the transverse distortion of pad elastomeric bearing.

3.3. Comparison of the drifts of pier top with and without dampers

Table 3.4. The drifts of pier top (m)

Direction	Pier No.	El Centro		Northridge		Parkfield		Wolong	
		Without damper	With damper	Without damper	With damper	Without damper	With damper	Without damper	With damper
longitudinal	1	0.3	0.13	0.35	0.26	0.46	0.28	0.28	0.11
	2	0.41	0.15	0.48	0.25	0.58	0.46	0.33	0.14
	3	0.29	0.24	0.36	0.17	0.47	0.37	0.30	0.21
	4	0.23	0.07	0.25	0.04	0.34	0.10	0.23	0.04
	5	0.22	0.08	0.24	0.03	0.35	0.07	0.21	0.10
	6	0.27	0.25	0.39	0.17	0.46	0.37	0.27	0.23
	7	0.38	0.17	0.42	0.17	0.52	0.23	0.32	0.17
	8	0.33	0.20	0.36	0.31	0.45	0.37	0.25	0.14
transverse	1	0.21	0.17	0.18	0.12	0.16	0.07	0.12	0.11
	2	0.3	0.20	0.23	0.19	0.19	0.16	0.22	0.12
	3	0.17	0.14	0.16	0.07	0.11	0.06	0.27	0.19

	4	0.18	0.16	0.16	0.14	0.14	0.12	0.28	0.29
	5	0.21	0.19	0.18	0.16	0.16	0.15	0.29	0.28
	6	0.22	0.20	0.18	0.10	0.12	0.06	0.27	0.16
	7	0.27	0.17	0.24	0.17	0.2	0.14	0.27	0.17
	8	0.20	0.16	0.17	0.12	0.15	0.07	0.12	0.09

The drift of pier top is changed after adding dampers. When the ground motion is El Centro wave, the variation ranges of longitudinal and transverse drift of approach bridge's pier top are -28%~-54% and -19%~-37%, the variation ranges of longitudinal and transverse drift of main pier top are -66%~-70% and -9%~-11%, and the variation range of longitudinal and transverse drift of juncture pier is -30%~-57% and -10%~-18%. When the ground motion is Northridge wave, the variation ranges of longitudinal and transverse drift of approach bridge's pier top are -15%~-59% and -17%~-33%, the variation ranges of longitudinal and transverse drift of main pier top are about -86% and -11%~-12%, and the variation range of longitudinal and transverse drift of juncture pier is -53%~-57% and -14%~-19%. When the ground motion is Parkfield wave, the variation ranges of longitudinal and transverse drift of approach bridge's pier top are -18%~-56% and -18%~-56%, the variation ranges of longitudinal and transverse drift of main pier top are -72%~-81% and -11%~-25%, and the variation range of longitudinal and transverse drift of juncture pier is -30%~-41% and -14%~-27%. When the ground motion is Wolong wave, the variation ranges of longitudinal and transverse drift of approach bridge's pier top are -34%~-61% and -18%~-45%, the variation ranges of longitudinal and transverse drift of main pier top are -52%~-83% and -4%~3%. and the variation range of longitudinal and transverse drift of juncture pier is -44%~-70% and -16%~-41%.

The longitudinal drift of each pier top decreases after adding dampers, and the reduced scope of main pier's drift is the largest, juncture pier's takes the second place, and approach bridge pier's is the lowest. The result is concordant with the first order mode and the mode shows that the longitudinal displacement of bridge is restrained so that the possibility of pounding declines greatly. The transverse drift of each pier top reduces after adding dampers. The reduced scope of different pier's drift is in the order of approach bridge pier's, juncture pier's and main bridge pier's, and in some case the drift of main bridge pier top increases a bit. The transverse damper for juncture pier is installed between juncture pier and approach deck beam, so main bridge's displacement is not limited well.

3.4. Comparison of the bending moment of piers with and without dampers

Table 3.5. The maximal moment at the bottom of the piers in longitudinal direction ($\times 10^7$Nm)

location	Pier No	El Centro		Northridge		Parkfield		Wolong	
		Without damper	With damper	Without damper	With damper	Without damper	With damper	Without damper	With damper
The pier bottom	1	3.04	2.33	3.45	2.45	3.6	2.8	2.94	2.14
	2	3.35	2.53	3.54	2.67	3.8	3.1	4.45	3.24
	3	9.9	7.9	12.8	8.32	13.6	9.1	12.3	10.4
	4	101	44.5	105	46.8	108	52	105.7	47.3
	5	95.6	38.3	101	40	105	53	97.2	49.2
	6	9.5	6.8	12.7	8.66	14.5	10.2	13.7	10.6
	7	2.97	2.16	3.35	2.5	3.56	2.8	5.01	3.54
	8	2.77	1.87	2.9	1.9	3.34	2.46	3.94	3.04

Table 3.6. The maximal moment at the bottom of the piers in transverse direction ($\times 10^7$Nm)

location	Pier No	El Centro		Northridge		Parkfield		Wolong	
		Without damper	With damper	Without damper	With damper	Without damper	With damper	Without damper	With damper
The pier	1	1.9	1.5	1.56	1.1	1.43	0.9	1.48	1.08

	2	2.56	1.85	2.34	1.33	2.11	1.1	1.78	1.37
	3	3.63	2.64	3.55	2.54	2.6	1.94	9.0	7.6
	4	76.2	61	72	58	63	52	89.5	77.6
	5	72.9	60	70.9	55	62.1	50	85.7	73
	6	3.64	3.2	3.24	2.44	2.57	2	9.7	7.3
	7	2.41	1.5	2.31	1.51	1.97	1.2	3.63	1.58
	8	1.96	1.42	1.58	1.1	1.45	0.9	1.38	1.08

The maximal moment at the bottom of the piers is changed after adding dampers. When the ground motion is El Centro wave, the variation ranges of longitudinal and transverse moment of approach bridge pier's bottom are -23%~-32% and -28%~-37%, the variation ranges of longitudinal and transverse moment of main pier's bottom are -56%~-60% and -18%~-20%, and the variation ranges of longitudinal and transverse moment of juncture pier's bottom are -20%~-28% and -12%~-27%.
When the ground motion is Northridge wave, the variation ranges of longitudinal and transverse moment of approach bridge pier's bottom are -26%~-34% and -30%~-43%, the variation ranges of longitudinal and transverse moment of main pier's bottom are -55%~-60% and about -19%~-22%, and the variation ranges of longitudinal and transverse moment of juncture pier's bottom are -32%~-35% and -25%~-28%.

When the ground motion is Parkfield wave, the variation ranges of longitudinal and transverse moment of approach bridge pier's bottom are -18%~-26% and -37%~-47%, the variation ranges of longitudinal and transverse moment of main pier's bottom are -50%~-52% and about -17%~-19%, and the variation ranges of longitudinal and transverse moment of juncture pier's bottom are -30%~-33% and -22%~-25%.

When the ground motion is Wolong wave, the variation ranges of longitudinal and transverse moment of approach bridge pier's bottom are -22%~-29% and -27%~-39%, the variation ranges of longitudinal and transverse moment of main pier's bottom are-49%~-55% and about -13%~-15%, and the variation ranges of longitudinal and transverse moment of juncture pier's bottom are -15%~-23% and -15%~-25%.

The maximal moments at the bottom of the piers in longitudinal direction reduce after adding dampers, and the reduced scope of main pier's moment is the largest, juncture pier's takes the second place, and approach bridge pier's is the lowest. The maximal moments at the bottom of the piers in transverse direction reduce after adding dampers, and the reduced scope of approach bridge pier's moment is the largest, juncture pier's takes the second place, and main pier's is the lowest.

3.5. Comparison of the strains of concrete and reinforcement at pier's bottom with and without dampers

The yield strain of reinforcement is 1.59×10^{-3}, the peak compressive strain of concrete is 1.79×10^{-3} and the maximal tension strain of concrete is 1.0×10^{-4}.

Table 3.7. Maximal tension strains of concrete cover ($\times10^{-4}$)

Pier No	El Centro		Northridge		Parkfield		wolong	
	Without damper	With damper	Without damper	With damper	Without damper	With damper	Without damper	With damper
Shorter pier of approach	7.00	4.1	7.20	6.5	6.37	5.50	10.9	11.4
Longer pier of approach	6.90	7.0	5.09	4.77	3.92	3.54	9.7	8.2
Juncture pier	9.40	8.90	9.15	7.08	3.92	3.69	10.5	10.2
Main pier's bottom	9.10	8.0	7.29	6.82	10.2	8.20	10.7	9.2

Table 3.8. Maximal tension strains of core concrete ($\times10^{-4}$)

Pier No	El Centro		Northridge		Parkfield		wolong	
	Without damper	With damper	Without damper	With damper	Without damper	With damper	Without damper	With damper
Shorter pier of approach bridge	3.88	3.04	10.10	5.51	5.21	4.21	9.5	8.7
Longer pier of approach bridge	4.61	4.30	8.13	3.16	3.56	2.54	10.4	9.1
Juncture pier	13.86	12.4	14.80	12.2	6.62	5.42	13.2	11.5
Main pier's bottom	9.57	8.5	12.50	12.4	9.78	8.69	10.3	9.4

Table 3.9. Maximal compressive strains of core concrete ($\times 10^{-4}$)

Pier No	El Centro		Northridge		Parkfield		Wolong	
	Without damper	With damper	Without damper	With damper	Without damper	With damper	Without damper	With damper
Shorter pier of approach bridge	1.77	1.14	2.65	1.39	3.6	1.89	3.17	2.56
Longer pier of approach bridge	2.10	1.39	2.73	1.15	3.26	1.67	3.26	2.63
Juncture pier	3.41	3.60	5.01	3.0	5.29	2.56	5.65	4.3
Main pier's bottom	4.41	3.39	6.46	2.82	6.84	3.23	6.29	5.48

Table 3.10. Maximal tension strains of reinforcing steel ($\times 10^{-3}$)

Pier No	El Centro		Northridge		Parkfield		Wolong	
	Without damper	With damper	Without damper	With damper	Without damper	With damper	Without damper	With damper
Shorter pier of approach bridge	2.00	1.77	2.35	1.58	2.91	1.95	2.2	1.77
Longer pier of approach bridge	2.65	1.94	2.84	1.05	3.15	2.01	2.28	1.80
Juncture pier	2.80	1.62	3.91	1.81	4.59	2.56	2.59	1.93
Main pier's bottom	3.48	1.42	4.56	1.77	5.01	2.87	3.04	2.17

The strains of concrete and reinforcing steel reduce sharply after adding dampers. Under El Centro wave, the variation range of strains of concrete and reinforcing steel at pier's bottom is -20%~-50%. Under Northridge wave, the variation range of strains of concrete and reinforcing steel at pier's bottom is -40%~-60%. Under Parkfield wave, the variation range of strains of concrete and reinforcing steel at pier's bottom is -40%~-50%. Under Wolong wave, the variation range of strains of concrete and reinforcing steel at pier's bottom is 20%~-30%. Some reinforcing steel at pier's bottom of main and approach bridge do not yield.

4. CONCLUSION

A preliminary analysis of effects of anti-pounding measures on high-pier bridge's seismic response is given in this paper. The seismic response of high-pier bridge with longitudinal and transverse dampers at the pounding location is compared with the seismic response of high-pier bridge without dampers and some conclusions are found.

The peak axial forces of beams increase sharply after adding dampers. The chief reason is that dampers restrain the relative movement between main and approach bridge, it influences the axial forces of adjacent beams, and the axial forces may be beyond the axial forces caused by direct pounding. The variation range of beam's peak axial forces of main bridge's is larger than approach bridge's.

1) The pounding is controlled by adding dampers, and mutation of acceleration caused by pounding is avoided. The peak accelerations of deck beams reduce sharply because dampers consume a lot of seismic pounding energy. The acceleration variation range of main bridge's beams is larger than approach bridge's, and girder end's is larger than their center's.

2) Dampers are very helpful to control transverse distortion of PTEF sliding bearings and to stop pounding between beam and stopper. Because longitudinal dampers are installed between approach bridge and main bridge, the vibrations of their influence each other, then there is no distinct regularity in bearing's longitudinal distortions of main and approach bridges.
3) The longitudinal and transverse drifts and moments of piers' top will be reduced after adding dampers, and the chief reason is that dampers consume some seismic pounding energy and unite the main and approach bridges to make their influence each other. The displacement of the whole bridge is controlled. The displacements and moments are to be reduced in transverse direction, the scope of decrease of approach bridge pier is the largest, juncture piers take the second place, and main bridge piers are the lowest. The transverse damper for juncture pier is installed between juncture pier and approach deck beam, so main bridge's displacement is not limited well. The strains of concrete and reinforcing steel are to be reduced sharply after adding dampers, and some reinforcing steel at pier's bottom of main and approach bridge do not yield.

Adding dampers in the pounding locations of high-pier bridge is useful to control structural displacement well, to avoid the mutation of acceleration caused by seismic pounding, to prevent pounding between deck beam and stopper, to reduce strains of concrete and reinforcing steel, to protect pier from serious damage, and to improve the whole bridge's aseismic performance. But, adding dampers between approach bridge and main bridge, which makes the vibrations of approach bridge and main bridge influence each other and restrain each other, may bring several bad or uncertain influences to some structural members, and there is no distinct regularity in bearing's longitudinal distortion and some bearing's transverse distortion.

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