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Shake Table Test Of A Cable-Stayed Bridge Using Yielding Steel Dampers

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ABSTRACT
To investigate the seismic performance of the cable-stayed bridge with different seismic structural systems, a 1/20-scale full bridge model from a typical medium span concrete cable-stayed bridge was designed, constructed and tested on the shake tables at Tongji University, Shanghai, China. Viscous Fluid Dampers (VFD) and Yielding Steel Dampers (YSD) were used as passive energy dissipation devices in the longitudinal and transverse directions, respectively. The seismic responses of the bridge model with and without passive devices were compared. The test results show that the passive energy dissipation devices applied in the cable-stayed bridge model can significantly reduce the strains at the bottom of tower legs and the displacement at the tower top, achieving a better seismic performance.

KEYWORDS: *Cable-stayed bridge, Shake table test, Energy dissipation system, Viscous fluid damper, Yielding steel damper*

1. INTRODUCTION

Cable-stayed bridges represent key points of the transport networks and, consequently, they are designed to remain nearly elastic under the design level of earthquake intensity, typically including the dampers to improve the seismic performance when located in seismic-prone areas. There are wide varieties of passive energy dissipation devices that can be used to improve the seismic performance of the structures, such as a metallic yield damper, friction damper, viscous fluid damper and viscoelastic damper etc[1]. Analytical investigations conducted on cable-stayed bridges with passive energy dissipation devices in the longitudinal and transverse direction demonstrated that the passive energy dissipation devices can achieve a significant reduction in seismic-induced displacement and forces as compared to the cases without dampers [2-7]. Shake tables are used extensively in seismic research because they provide the means to excite structures in such a way that they are subjected to conditions representing true earthquake ground motions. Godden [8] tested the dynamic characteristics of 1/200-scale Ruck-A-Chucky Bridge, including static live-load response, system damping, natural frequencies, and mode shapes. Shake table tests of the 1/150-scale Jindo Bridge was conducted at the University of Bristol to study the dynamic characteristics [9]. Although there were several numerical studies conducted on the application of passive energy dissipation devices for cable-stayed bridges, there are few test verifications for the effectiveness of these applications.

Therefore, to investigate the seismic performance of the cable-stayed bridge with different seismic structural systems, a 1/20-scale full bridge model from a typical medium span concrete cable-stayed bridge was designed, constructed and tested on the shake tables at Tongji University, Shanghai, China. Viscous Fluid dampers (VFD) and Yielding Steel Dampers (YSD) were used as passive energy dissipation devices in the longitudinal and transverse directions, respectively. The objective of this study is to investigate the effectiveness of passive energy dissipation devices in the seismic mitigation of concrete cable-stayed bridges.

2. TEST MODEL DESIGN AND CONSTRUCTION

The shake-table test model was designed to investigate the seismic performance of the cable-stayed bridges with and without passive energy dissipation devices. The test model, illustrated in Figure 1, modeled a typical medium-span cable-stayed bridge with two H-shaped concrete towers at 1/20 scale to maximize the size of the specimen, while remaining within the capacity limits of the shake tables testing systems. According to the scale factor, the total height of the model from the base of the footings to the top of the tower is 4.85 m, and the total length is 32 m. The testing system is composed of four shake tables, which were installed into a linear trench and worked as a large shake table array.

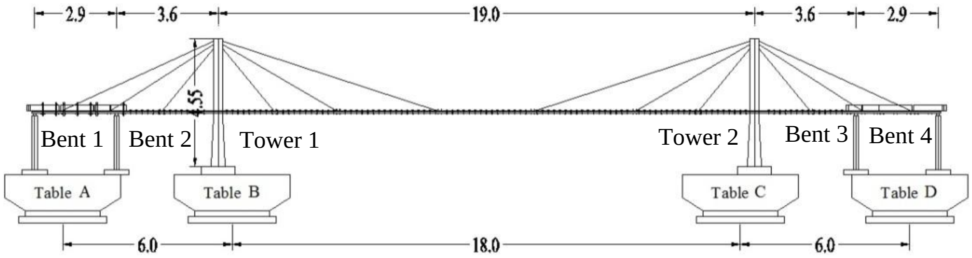


Figure 1. Bridge model and arrangement of four shake tables

To simulate the prototype response accurately under the earthquake input, the acceleration scaling factor was set to be 1. Due to the scaling effect, additional masses were attached to the bents, towers and bridge deck accordingly. Table 1 lists the self-weight and the additional masses of the model. The total mass of the model results in 47351 kg, including 8908 kg self-weight and 38443 kg additional masses.

Table 1. Self-weight and additional mass of the model

Component	Self-weight (kg)	Additional mass (kg)	Number	Total mass (kg)
Tower	1590	6896	2	16972
Bent	479	1251	4	6920
Deck	3740	19647	1	23387
Cable	3	/	24	72

The Buckingham π theorem of dimensional analysis [10] was used to design the tower and bent models. The dimensions of the towers and bents are shown in Figure 2. For the pylons and the bents, the prototype concrete, longitudinal bars and reinforcement stirrups were substituted by micro-concrete, the Φ6 steel bars and the galvanized wires respectively. The averaged measured yield stress for steel bars was 260 MPa, and the averaged elastic modulus of the micro-concrete at the time of testing was 10.3 GPa. The cross section of the prototype deck is a streamlined, flat, thin-walled steel box, which would be difficult to be manufactured if strictly meeting the requirement of scale factor. Therefore, the model deck was designed to a box section in 10mm thick with equivalent bending moments of inertia along both strong and weak axes, as shown in Figure 2. According to the principles of equivalent cable forces and the dynamic characteristics from the prototype, the numbers of cables were simplified as shown in figure1 with the cross sectional area of the 7.85×10-5mm. The assembled bridge model is shown in figure 3.

3. SEISMIC STRUCTURAL SYSTEMS

In the test model, two different seismic structural systems were considered in the longitudinal and transverse directions respectively, as summarized in Table 2. VFD and YSD were used as passive energy dissipation devices, as shown in Figure 4 and Figure 5. The damping constant and the velocity exponent of VFD equaled to 25 kN $s^{0.3}$ $m^{-0.3}$ and 0.3, respectively. The yield force, the yield displacement and the ratio of post- to pre-yield stiffness for YSD were 1.5 kN, 2 mm and 0.24, respectively.

Table 2. Seismic structural systems compared in tests

Direction	Model	Deck-bent/deck-tower connections					
		Bent 1	Bent 2	Tower 1	Tower 2	Bent 3	Bent 4
Longitudinal	LA	Movable	Movable	Movable	Movable	Movable	Movable
	LB	Movable	Movable	VFD	VFD	Movable	Movable
Transverse	TA	Fixed	Movable	Fixed	Fixed	Movable	Fixed
	TB	YSD	YSD	YSD	YSD	YSD	YSD

Note: LA, LB,TA and TB means the different structural system of bridge model as indicated in the table.

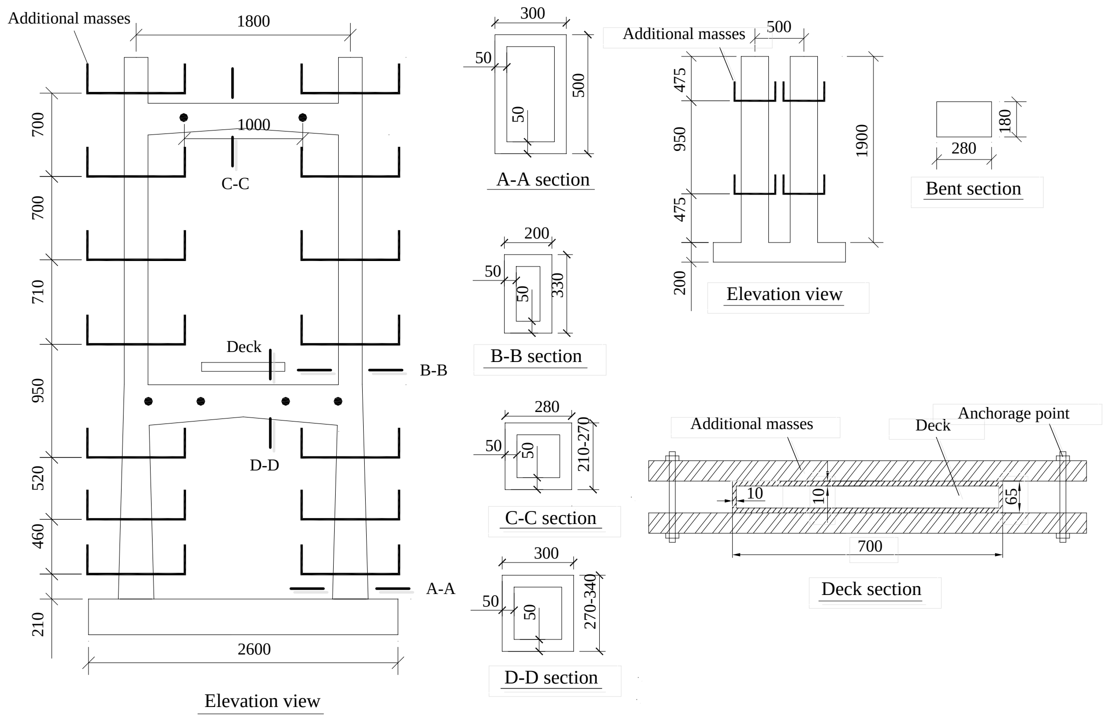


Figure 2. Details of towers, bents and deck (unit:mm)

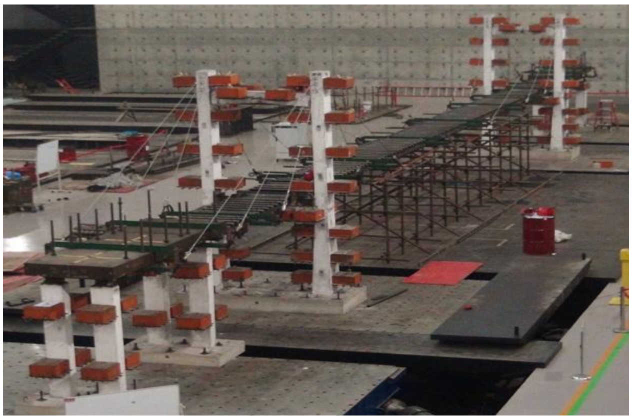


Figure 3. Completed bridge model

The deck of the bridge model was supported on bents and towers through bearing systems which can slide both in longitudinal and transverse directions. As a result, the Teflon sliding type rubber bearing (see figure 6) was developed to provide the vertical support with very little friction effect in both longitudinal and transverse movements.

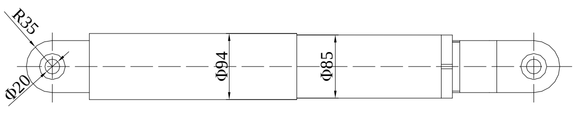
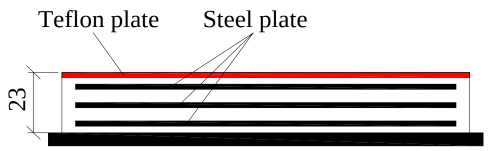


Figure 4. Elevation of VFD [mm]



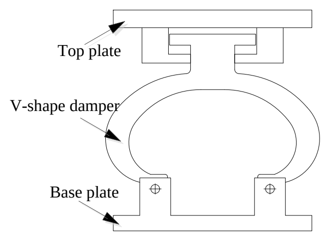
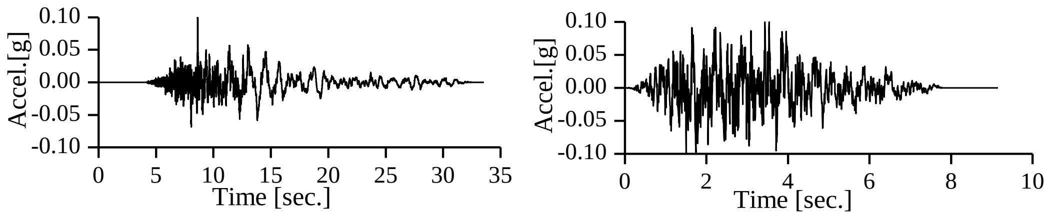


Figure 5. Elevation of YSD

Figure 6. Elevation of Teflon slide type rubber bearing [mm]

3.1 Testing Protocol

The north component of the 1999 Chi-Chi Earthquake measured at the CHY002 station and a site specific artificial wave were applied in the tests. The time axis of the motion was accordingly compressed by 0.2236 (square root of the model scale of 1/20). Figure 7 shows the input records with a compressed time axis and scaled amplitude (0.1g).



（a）Chi-Chi wave （b）Artificial wave

Figure 7. Input motions with compressed time axis and scaled amplitude

The bridge models were subjected to a series of uniaxial seismic inputs in longitudinal and transverse, respectively. White noises were also applied between each test runs to evaluate the frequency of the test model. The cases arranged for Bridge Model LA and TA are listed in Table 3. For Models LB and TB in Table 2, the input ground motions were the same.

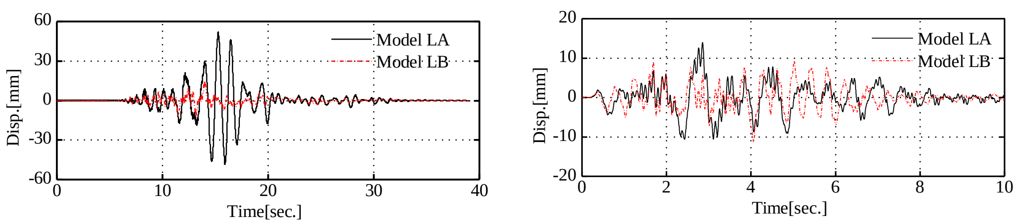
Table 3. Cases arranged for Bridge Model LA and TA

Direction	Model	Cases	Input motion	PGA(g）
Longitudinal	LA	LA1-LA2	Chi-Chi wave	0.1g-0.2g
		LA3-LA7	Artificial wave	0.1g-0.5g
Transverse	TA	TA1-TA4	Chi-Chi wave	0.1g-0.4g
		TA5-TA8	Artificial wave	0.1g-0.4g

3.2 Test results

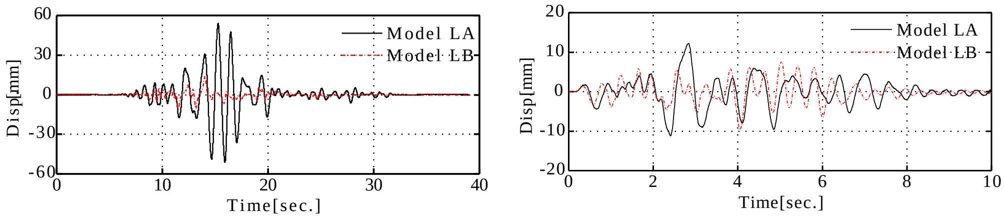
3.2.1 Displacement Responses

Comparisons of longitudinal and transverse displacement time histories at tower top and midpoint of deck are shown in Figures 8-11. Peak longitudinal and transverse displacement responses for different models are listed in Tables 4 and 5. Seen from these figures and tables, the longitudinal and transverse displacements at tower top and midpoint of deck of Model LB and TB were generally smaller than those of Model LA and TA. And owing to the differences of the spectral characteristics and the duration between Chi-Chi wave and artificial wave, the transverse displacement at midpoint of deck of Model TB was slightly greater and has a slower attenuation than that of Model TA.



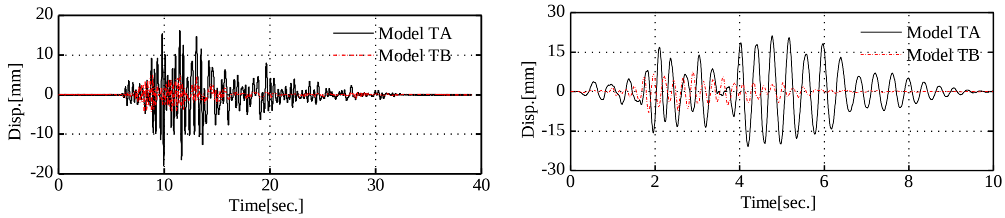
（a）Chi-Chi wave with a PGA of 0.2g （b）artificial wave with a PGA of 0.2g

Figure 8. Comparisons of longitudinal displacement at tower top between Model LA and Model LB



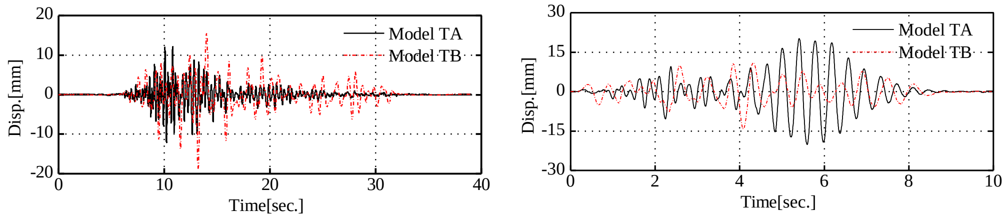
（a）Chi-Chi wave with a PGA of 0.2g （b）artificial wave with a PGA of 0.2g

Figure 9. Comparisons of longitudinal displacement at midpoint of deck between Model LA and Model LB:



（a）Chi-Chi wave with a PGA of 0.2g （b）artificial wave with a PGA of 0.2g

Figure 10. Comparisons of transverse displacement at tower top between Model TA and Model TB



（a）Chi-Chi wave with a PGA of 0.2g （b）artificial wave with a PGA of 0.2g

Figure 11. Comparisons of transverse displacement at midpoint of deck between Model TA and Model TB

Table 4. Peak longitudinal displacement responses for Model LA and LB

Input motion	PGA(g）	Peak displacement at tower top(mm)		Peak displacement at midpoint of deck(mm)	
		Model LA	Model LB	Model LA	Model LB
Chi-Chi wave	0.1	8.0	6.5	12.5	6.0
	0.2	51.9	15.0	53.8	14.1
Artificial wave	0.1	7.0	6.4	7.0	5.6
	0.2	14.0	11.3	12.2	9.7
	0.3	19.8	13.7	18.5	10.4
	0.4	31.9	18.5	31.7	15.2
	0.5	41.1	24.2	41.9	20.2

Table 5. Peak transverse displacement responses for Model TA and TB

Input motion	PGA(g）	Peak displacement at tower top(mm)		Peak displacement at midpoint of deck(mm)	
		Model TA	Model TB	Model TA	Model TB
Chi-Chi wave	0.1	8.6	2.8	6.6	8.8
	0.2	17.8	5.2	12.2	18.8
	0.3	26.8	11.9	16.9	31.3
	0.4	36.0	13.4	19.9	35.5
Artificial wave	0.1	11.2	4.0	9.1	9.5
	0.2	21.2	8.2	20.2	14.1
	0.3	30.4	13.0	22.1	17.5
	0.4	47.0	12.2	20.2	16.3

3.2.2 Strain Responses

The peak steel strains at the bottom of the tower leg under the input motions in longitudinal and transverse directions are listed in Table 6 and 7, respectively. Seen from these tables, the peak steel strains at the bottom of the tower leg of Model LB and TB were generally smaller than those of Model LA and TA. Under Chi-Chi wave with a PGA of 0.2g, the peak steel strains at the bottom of the tower leg of Model LB and TB were 61% and 69% smaller than those of Model LA and TA. And artificial wave with a PGA of 0.4g, those of Model LB and TB were 11% and 84% smaller.

Table 6. Peak steel strains at the bottom of the tower leg for Model LA and LB

Input motion	PGA(g）	Peak steel strains at the bottom of the tower leg(με)	
		Model LA	Model LB
Chi-Chi wave	0.1	324	495
	0.2	2252	871
Artificial wave	0.1	240	458
	0.2	608	696
	0.3	813	769
	0.4	1065	945
	0.5	1546	1237

Table 7. Peak steel strains at the bottom of the tower leg for Model TA and TB

Input motion	PGA(g）	Peak steel strains at the bottom of the tower leg(με)	
		Model TA	Model TB
Chi-Chi wave	0.1	523	163
	0.2	1400	440
	0.3	2249	636
	0.4	3225	695
Artificial wave	0.1	496	213
	0.2	1332	475
	0.3	2753	853
	0.4	4729	756

4. CONCLUSIONS

This paper presents the design and testing of a cable-stayed bridge model with different seismic structural systems that completed by the four-shake-table system at Tongji University, Shanghai. VFD and YSD were used as passive energy dissipation devices in the longitudinal and transverse directions, respectively. Chi-Chi wave and an artificial wave were applied as a series of uniaxial motions in longitudinal and transverse directions, respectively. The seismic responses of the bridge with and without passive devices were compared; the following conclusions are the main findings of the test results:

1. The passive energy dissipation devices in the test model can significantly reduce the displacement at the tower top in the longitudinal and transverse directions, respectively. However, owing to the effects of the spectral characteristics and the duration of Chi-Chi wave, the transverse displacement at midpoint of deck increases slightly with YSD.
2. With these passive energy dissipation devices, a reduction up to 69% and 84% in the strain response of the isolated bridge can be achieved in the longitudinal and transverse directions, respectively.

Therefore, the proposed energy dissipation systems, consisting of VFD and YSD in the longitudinal and transverse directions, respectively, can be applied to the cable-stayed bridges to achieve a better seismic performance.

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