SEISMIC CONTROL OF BENCHMARK CABLE-STAYED BRIDGE USING VARIABLE FRICTION PENDULUM SYSTEMS

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SUMMARY:
Earthquake response of phase–II benchmark cable-stayed bridge with variable curvature friction pendulum systems (VCFPS) is investigated. The performance of this isolator is compared with traditional friction pendulum system (FPS). The benefit of the friction isolators is that it ensures the maximum acceleration transmissibility equal to the maximum limiting frictional force. In VCFPS, the radius of curvature is lengthened with an increase of the isolator displacement. Hence, the fundamental period of the base-isolated structure can be shifted further away from the predominant periods of near-fault ground motions, and the resonant possibility of the superstructure with earthquakes can be prevented. The seismic response of the bridge is evaluated using the hysteretic model of FPS and VCFPS. The control force produced by VCFPS to reduce the seismic response bridge is much less than that of FPS. Comparing the evaluation criteria of the benchmark problem, it is observed that the performance of VCFPS is better than FPS.

***Keywords**: Benchmark cable-stayed bridge, FPS, VCFPS, seismic response, sliding base isolation.*

1 INTRODUCTION

Cable stayed bridges have become popular throughout the world and are also very important lifeline structures. The increasing popularity of these bridges among bridge engineers can be attributed to their appealing aesthetics, full and efficient utilization of structural materials and increased stiffness over substructure. However this structure is susceptible to earthquake motions due to its flexibility and low damping. For direct comparison among the different control strategies for a specific type of structure, benchmark problems were developed. Then researchers can compare their various algorithms, devices and sensors for a particular structure through same problem. On the basis of the Bill Emerson Memorial Bridge constructed in Cape Girardeau, Missouri, USA, benchmark problem on cable-stayed bridge have been developed by Dyke et al in 2003. It specifies some performance objectives from which direct comparison could be made. The control of energy that is transmitted from the ground or foundation to the structure is one of the most effective techniques for seismic design of structures. In recent years, there have been significant studies on the use of frictional isolators to add damping to the isolated structure. The performance of friction isolators is quite insensitive to severe variations in the frequency content of the base excitation, making them very robust (Mostaghel and Khodaverdian 1987). This feature is the most important advantage of the friction type isolators as compared to elastomeric bearings. The benefit of the friction isolators is that it ensures the maximum acceleration transmissibility equal to the maximum limiting frictional force(Mostaghel et al. 1987, Mostaghel and Tanbakuchi 1983).

There are several types of passive control devices used by various researchers to control the seismic response of the civil engineering structures. The sliding isolation system, which is based on the concept of sliding friction, is one of them. Among various friction base isolators, the Friction Pendulum System (FPS) is found to be most attractive due to its ease in installation and simple mechanism of restoring force by gravity action. In this isolator, the sliding and re-centering mechanisms are integrated in single unit. The sliding surface of FPS is spherical so that its time period of oscillation remains constant. Numerous studies had been carried out on behavior of FPS. Seismic control of bridge using sliding isolation was performed experimentally by Tsopelas et al. 1996. The finite element formulation of the FPS for seismic isolation had been carried out by Tsai 1997. A systematic method for the dynamic analysis of the continuous bridge with sliding isolation is developed by Wang et al. 1998. Jangid (2000) investigated the optimum friction coefficient of a sliding system with a restoring force for the minimum acceleration response of a base-isolated structure under earthquake ground motion. The performance of the optimum FPS system for near-fault ground motions has been investigated by Jangid (2005) and observed that, for lower values of friction coefficient, significant sliding displacement in the FPS. The seismic response of bridges isolated by elastomeric bearings and the sliding system is investigated by Kunde and Jangid (2006). Soneji and Jangid (2006) investigated the effectiveness of elastomeric and sliding isolation systems for the seismic response control of cable-stayed bridge. Comparative performance of isolation systems for benchmark cable-stayed bridge is carried out by Saha and Jangid (2008) and noticed that isolator displacement is very large for near-field earthquake (Gebze 1999 earthquake). The FPS had also been practically used for the seismic isolation for bridges such as Benicia-Martinez Bridge, California and American River Bridge, California etc. However, based on the literature survey on FPS, it is observed that the use of spherical sliding surface of the FPS results in several practical disadvantages. One main disadvantage is that FPS needs to be designed for a specific level (intensity) of ground excitation. This is primarily because the maximum intensity of excitation has a strong influence on FPS design, even though the performance of structures isolated by FPS is relatively independent of the frequency content of ground motion. In general, FPS designed for a particular intensity of excitation may not give very satisfactory performance during earthquakes with much lower or higher intensity. To overcome this problem, variable sliding isolators like VCFPS (Tsai et al. 2003) is developed modifying the parameters of the FPS. The success of variable sliding isolators for controlling the seismic forces leads us to study the performance of the different variable sliding isolation systems for phase-II benchmark cable-stayed bridge.

The aim of the present study is (i) to investigate the effectiveness of the VCFPS for seismic response control of the phase-II benchmark cable-stayed bridge subjected to specified earthquake ground motions, and (ii) to investigate the influence of variation in important parameters of the isolators on the seismic response of the bridge.

2. BENCHMARK CABLE-STAYED BRIDGE

Bill Emerson Memorial Bridge crossing the Mississippi River near Cape Girardeau, Missouri (Fig. 1), designed by the HNTB Corporation, and is the benchmark cable-stayed bridge used for this study. As the bridge is the primary crossing of river and is in the New Madrid seismic zone, seismic forces were strongly considered in this bridge design.

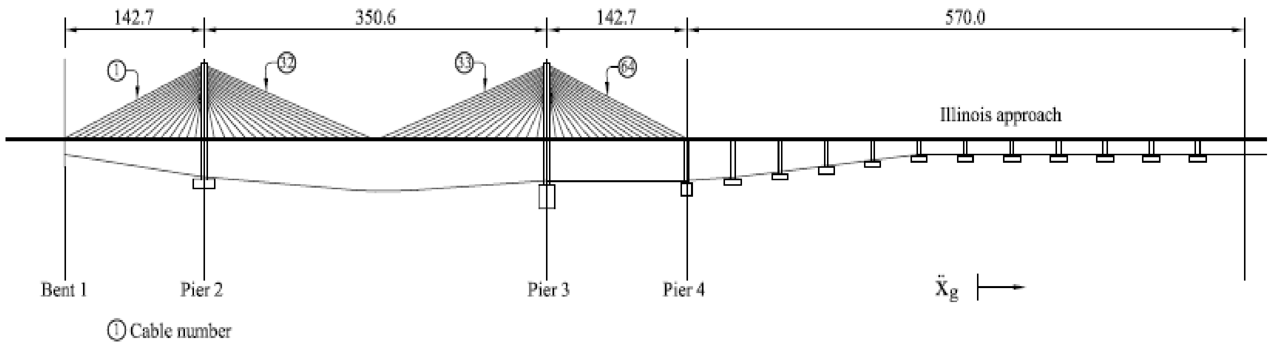


Figure 1. Drawing of the Cape Girardeau Bridge (Dyke et al. 2003)

3. EVALUATION CRITERIA

For each control design, the evaluation criteria should be evaluated for each of the following three earthquake records provided in the benchmark problem (Caicedo et al. 2003): (i) *El Centro*, recorded at the Imperial Valley Irrigation District substation in El Centro, California, during the Imperial Valley, California earthquake of 18, May, 1940; (ii) *Mexico City*, recorded at the Galeta de Campos station with site geology of Meta-Andesite Breccia on 19, September, 1985; (iii) *Gebze, Turkey*, the north-south component of the Kocaeli earthquake recorded at the Gebze Tubitak Marmara Arastirma Merkezi on 17, August, 1999.

4. RESPONSE OF BRIDGE WITHOUT CONTROL

For a cable-stayed bridge subject to an earthquake in the longitudinal direction of the deck, there are three response variables of interest. Those are (i) the actions on the towers; (ii) the displacement of the deck and (iii) the variations of force in the stays, which should be confined in the range $0.2\,T_f$ – 0.7 T_f , (with T_f denoting the failure tension) (Dyke et al. 2003). The bridge without control can assume two distinct configurations: (a) a configuration in which the deck is restrained longitudinally to the main piers; (b) a configuration in which the deck is not restrained longitudinally to the piers and the tie in this direction is supplied only by the stays. In configuration (a) the bridge shows limited displacements, but a high shear at the base of the towers as well as unacceptable variations of tension in the cables. In particular these are found in the cables anchored in the highest positions on the towers; such cables are those with the greatest tensions. In configuration (b), even though there are maximum values of shear and moment respectively equal to 45.6 % and 58.7% of those of configuration (a), one sees an unacceptable sliding of the deck, with a maximum displacement equal to 0.77 m (Dyke et al. 2003).

5. FRICTION PENDULUM SYSTEM (FPS)

The FPS is a frictional isolation system that combines a sliding action and a restoring force by geometrical properties. The FPS isolator, shown schematically in Fig. 2, has an articulated slider that moves on a stainless steel spherical surface. As the slider moves over the spherical surface, it causes the supported mass to rise and provides the restoring force for the system. The natural period of the FPS depends on the radius of curvature (r_c) of the concave surface. The natural period vibration (T_b) of a rigid mass supported on FPS connections is determined from the pendulum equation:

$$T_b = 2\pi\sqrt{\frac{r_c}{g}} \tag{1}$$

Where, g is the acceleration due to gravity. The isolated period becomes active once the friction force level of the isolator is exceeded. The ideal force-deformation behavior of FPS is also shown in Fig. 2. The resisting force (f) provided by the FPS is given by:

$$f = k_b x_b + F_x \tag{2}$$

Where, k_b is the bearing stiffness provided by virtue of inward gravity action at the concave surface; x_b is the device displacement and F_x is the frictional force.

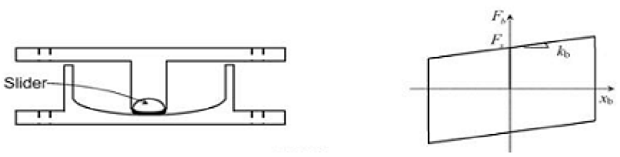
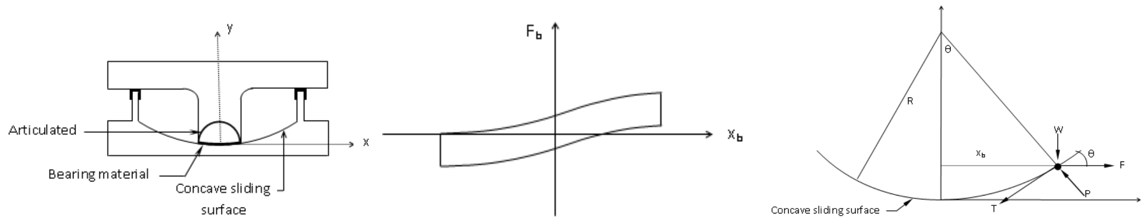


Figure 2. Friction Pendulum System (FPS)

6. VARIABLE CURVATURE FRICTION PENDULUM SYSTEM (VCFPS)

An advanced isolator called variable curvature friction pendulum system (VCFPS) (refer Figure. 3 a & b) is found to be very effective for structures adjacent to active earthquake faults (Tsai et al. 2003). In this isolator, the radius of curvature is lengthened with an increase of the isolator displacement. Hence, the fundamental period of the base-isolated structure can be shifted further away from the predominant periods of near-fault ground motions, and the resonant possibility of the superstructure with earthquakes can be prevented (Tsai et al. 2003).



(a) (b) (c)

Figure 3. (a) Schematic diagrams, (b) ideal force-deformation behavior, (c) forces acting on concave sliding surface of VCFPS.

The geometric function used to describe the VCFPS base isolator can be expressed (Tsai et al. 2003) in the following

$$y = R - \sqrt{R^2 - x_b^2} - f\left(x_b\right) \tag{3}$$

$$f\left(x_b\right) = C\,\mathrm{sgn}\left(x_b\right)x_b^3 \tag{4}$$

where R the radius of curvature at the center of the sliding is surface of the VCFPS; x_b is the horizontal displacement of the isolator; $f\left(x_b\right)$ is the function to describe the increase of the radius of curvature with an increase of the horizontal displacement; and C is the parameter that describes the variation of curvature of the concave surface. If the restoring force (refer Figure 3(c)) (Tsai et al. 2003) that can bring the slider back to the initial position within the sliding displacement x_0 with initial static force $T_0 = \mu W_d$, then the parameter C can be determined (Tsai et al. 2003) as:

$$C = \frac{\dfrac{W_d x_0}{\sqrt{R^2 - x_0^2}} - \dfrac{\mu g}{\cos\theta_0}}{3W_d\,\mathrm{sgn}\left(x_0\right)x_0^2} \tag{5}$$

The horizontal stiffness of the VCFPS can be written (Tsai et al. 2003) as:

$$k_b\left(x_b\right) = m_d\omega_b^2\left(x_b\right) \tag{6}$$

$$\omega_b^2\left(x_b\right) = \frac{g}{\sqrt{R^2 - x_b^2}} - 3gC\,\mathrm{sgn}\left(x_b\right)x_b \tag{7}$$

$$f = k_b(x_b).x_b + F_x \tag{8}$$

where μ is the coefficient of the friction at the sliding surface of the VCFPS; g is the acceleration due to gravity; $W_d = m_d g$ is the weight supported by the isolator; and m_d is the total mass of the deck.

The restoring force of the VCFPS is expressed by Equation (8), where F_x is the frictional force in the VCFPS which can be derived using the hysteretic model, keeping the parameters same as FPS; $k_b(x_b)$ is the stiffness of the VCFPS which can be determined by Equation (6); and x_b is the isolator displacement. The important parameters of the VCFPS are initial isolation period $\left(T_i\right)$ and μ.

7. GOVERNING EQUATIONS OF MOTION

The general equation of motion for a structural system subjected to seismic loads when the excitation has a single component or when the excitation is uniformly applied at all supports of the structure

$$\mathbf{M\ddot{U}} + \mathbf{C\dot{U}} + \mathbf{KU} = -\mathbf{M}\Gamma\ddot{x}_{\boldsymbol{g}} + \Lambda\mathbf{f} \tag{9}$$

For multiple supports,

$$\mathbf{M\ddot{U}} + \mathbf{C\dot{U}} + \mathbf{KU} = \Lambda\mathbf{f} - \ \mathbf{MR}_s + \mathbf{M}_g \ \ \mathbf{\ddot{U}}_g - \ \mathbf{CR}_s + \mathbf{C}_g \ \ \mathbf{\dot{U}}_g \tag{10}$$

where $\ddot{\boldsymbol{U}}$ [m/s^2] is the second time derivative of the displacement response vector $\boldsymbol{U}$ [m], **M**, **C**, and **K** are the mass, damping and stiffness matrices of the structure, $\mathbf{f}$ [N] is the vector of control force inputs, $\ddot{x}_g$ [m/s^2] is the ground acceleration, Γ is a vector of zeros and ones, relating the ground acceleration to the bridge degrees of freedom (DOF), Λ is a vector relating the force(s) produced by the control device(s) to the bridge DOFs (Dyke et al. 2003). Matrices $\mathbf{M}_g$, $\mathbf{C}_g$ and $\mathbf{K}_g$ are the mass, damping and elastic coupling matrices expressing the forces developed in the active DOFs by the motion of the supports. $\mathbf{R}_s$ $\mathbf{R}_s = -\mathbf{K}^{-1}\mathbf{K}_g$ is the pseudo-static influence vector which describes the influence of support displacements on the structural displacements (Caicedo et al. 2003). Equation (10) is used for phase II benchmark cable-stayed bridge problem.

8. NUMERICAL STUDY

A set of numerical simulation is performed in MATLAB (2002) and SIMULINK (1997) for the specified three historical earthquakes to investigate the effectiveness of the sliding isolation systems of the phase-II benchmark cable-stayed bridge. The fundamental frequency of the bridge in evaluation model (a) is 3.45 sec and that of evaluation model (b) is 6.18 sec. To implement the isolation systems, a total numbers of 24 isolators were used in 8 locations between the deck and pier/bent, 3 at each location with device configuration [3 3 3 3 3 3 3 3]. Time history analysis is performed for the three earthquake ground motions specified in the benchmark problem (Caicedo et al. 2003) to obtain the structural responses of the bridge. For the parametric studies, normal evaluation model 2 with incidence angle 15^0 has been chosen (Caicedo et al. 2003).

The sliding isolator VCFPS has one limitation for this benchmark bridge. It cannot operate for the value of initial isolation period (T_i) is less than 2.0 sec since the radius of the VCFPS is less than the recommended sliding displacement ($x_0 = 0.8\mathrm{m}$)(Tsai et al. 2003). Hence, the investigation is carried out by varying the parameter T_i from 2.0 sec to 7.0 sec. The variation of peak responses for different initial isolation period of VCFPS considering μ = 0.05 is shown in Figure 4. It is observed from the figure that the variations of the responses are not significant except the deck displacement of Gebze (1999) earthquake which is increasing significantly with increase in isolation period. Hence initial isolation period of 2.0 sec is preferred as optimum value of T_i for this study.

The variation of peak responses for VCFPS, varying friction coefficient from 0.05 to 0.15 for T_i equal to 2.0 sec, is presented in Figure 5. Examining Figure 5, it is observed that increase in friction coefficient of VCFPS; decrease the deck displacement response for all the specified earthquakes, especially Gebze (1999) earthquake. With the increase in μ, peak responses show optimum value of μ for the specified earthquakes but for this study μ equal to 0.05 is chosen which is recommended for traditional FPS.

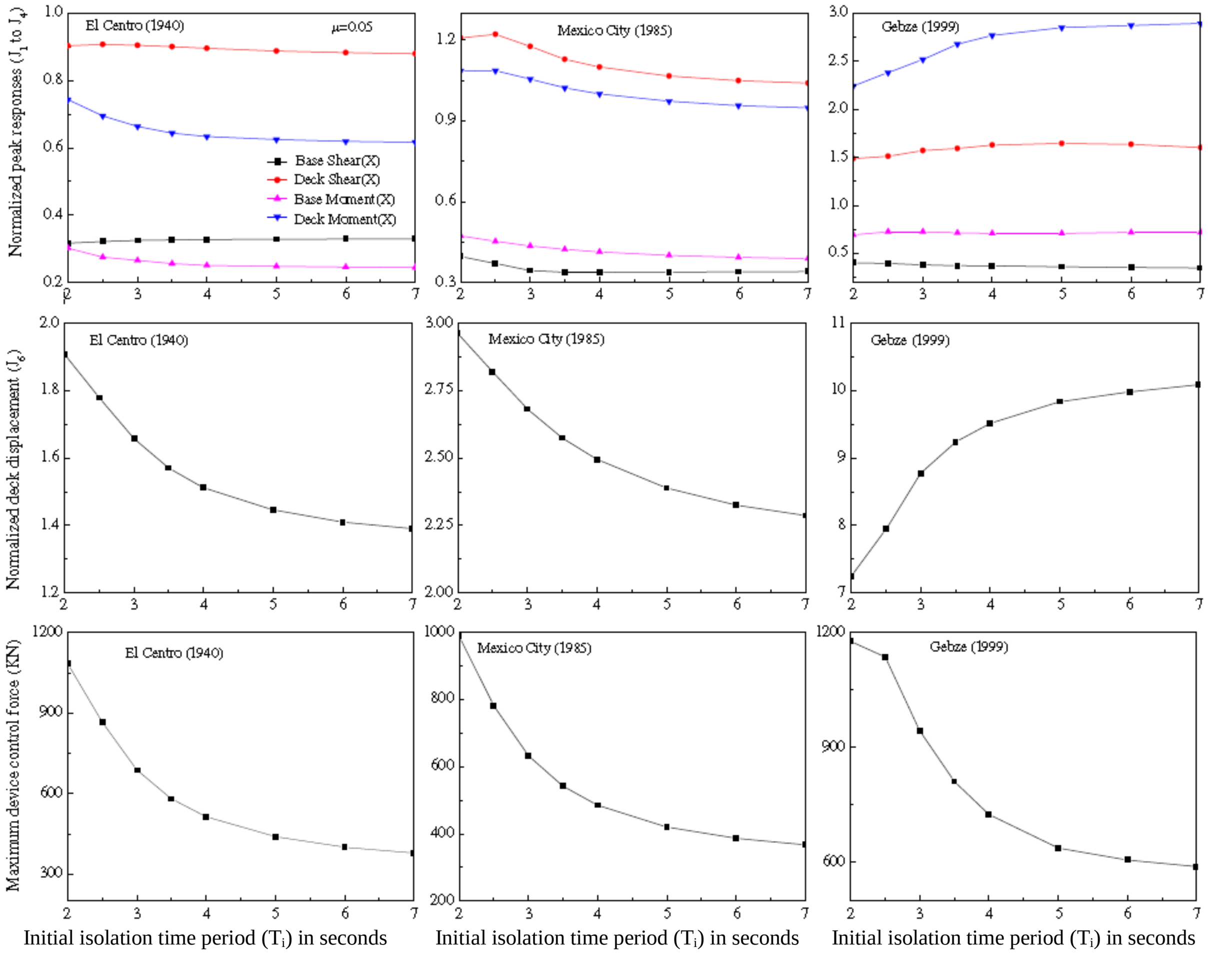


Figure 4. Effect of initial time period of VCFPS on normalized responses of the bridge

The time variation of the base shear response (X direction) at pier 2 of the earthquakes for the optimum parameters value considered above are shown in Figure 6 for VCFPS. From the figure, it can be observed that around 68% reduction for El Centro (1940) earthquake, 60% reduction for Mexico City (1985) earthquake and 64% reduction for Gebze (1999) earthquake can be achieved by this isolator. It can be noted that maximum reduction of base shear response in longitudinal direction, for Gebze (1999) earthquakes is achieved by VCFPS. The force-deformation behavior of the VCFPS at pier 2 (tower), for the specified earthquakes is shown in Figure 7.

The evaluation criteria of the bridge considering the above isolation parameters for incidence angle (θ) equals to 15^0 and 45^0 are shown in Table 1for the maximum values of the evaluation criteria for all the three earthquakes. To investigate the robustness of the control strategies, an alternate model is developed in phase-II problem considering the snow load. The evaluation criteria of the bridge for incidence angle (θ) equals to 15^0 and 45^0 with snow load are shown in Table 1.

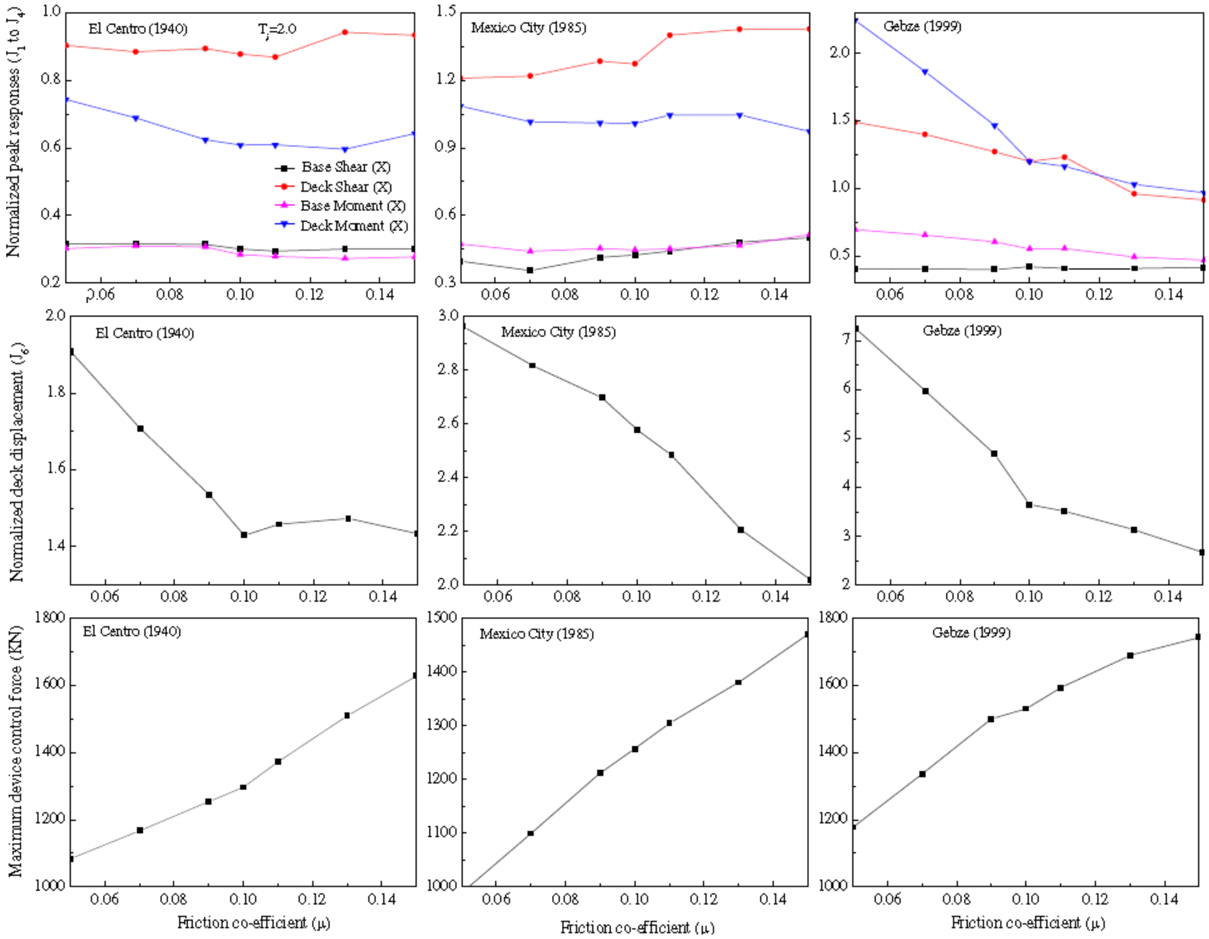


Figure 5. Effect of friction coefficient of VCFPS on normalized responses of the bridge

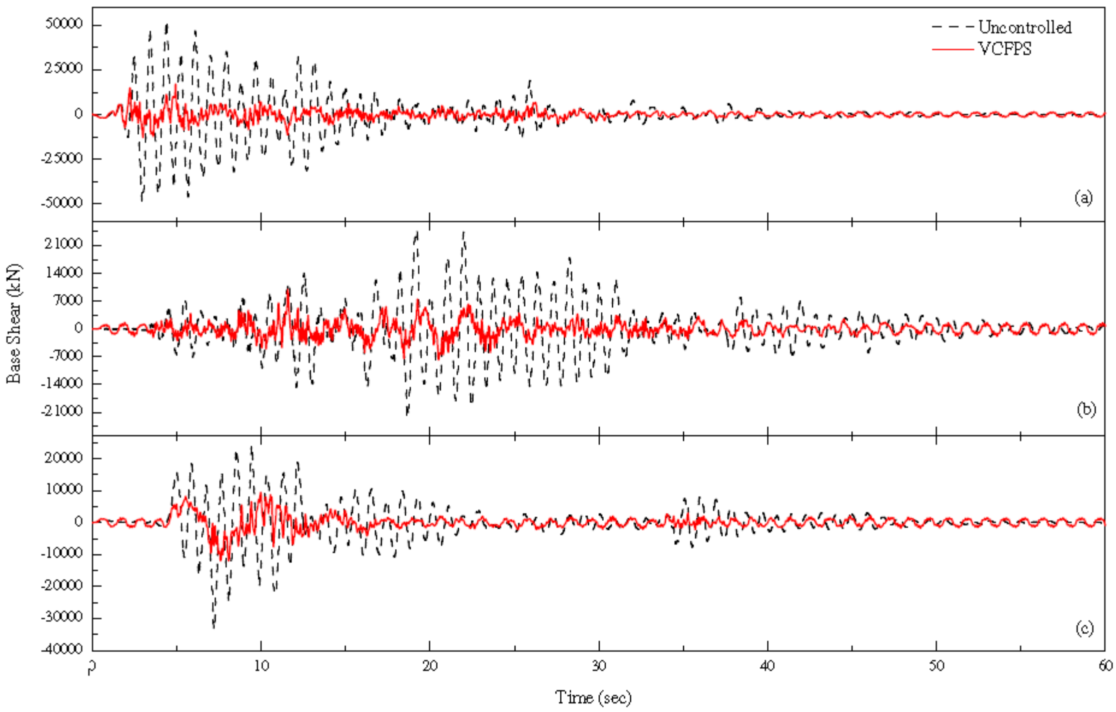
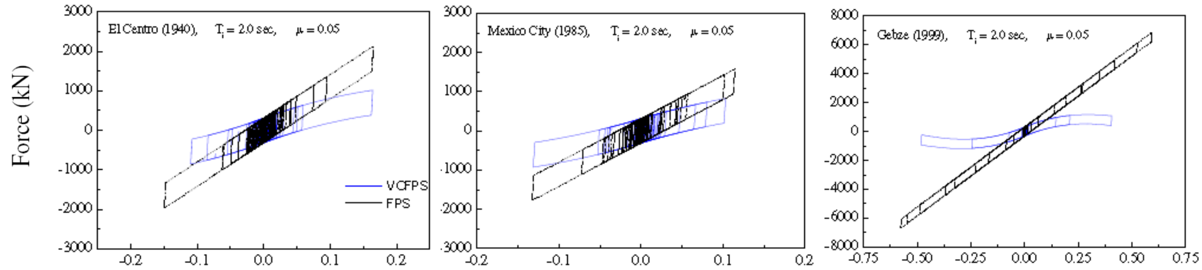


Fig. 6 Uncontrolled and VCFPS controlled Base Shear response for (a) El Centro (1940), (b) Mexico City (1985) and (c) Gebze (1999) earthquakes at pier 2



(a) (b) (c)

Fig. 7 Force deformation behavior of VCFPS for (a) El Centro (1940), (b) Mexico City (1985) and (c) Gebze (1999) earthquakes at pier 2

Table 1 Maximum evaluation criteria for the all three earthquakes

Evaluation Criteria		Incidence angle θ = 15^0				Incidence angle θ = 45^0			
		Without snow load		With snow load		Without snow load		With snow load	
		FPS	VCFPS	FPS	VCFPS	FPS	VCFPS	FPS	VCFPS
J_1	Peak base shear (X)	0.484	0.403	0.500	0.538	0.491	0.442	0.577	0.656
	Peak base shear (Z)	1.192	1.200	1.135	1.168	1.049	1.119	1.153	1.136
J_2	Peak shear at deck (X)	1.648	1.489	1.295	1.324	1.370	1.232	1.354	1.311
	Peak shear at deck (Z)	1.079	1.080	1.022	1.025	1.106	1.106	1.053	1.053
J_3	Peak base moment(X)	0.885	0.693	0.749	0.571	0.986	0.745	0.876	0.662
	Peak base moment(Z)	1.136	1.142	1.164	1.166	1.259	1.327	1.159	1.172
J_4	Peak moments at deck (X)	2.629	2.246	1.625	1.184	2.531	2.090	1.606	1.055
	Peak moments at deck (Z)	1.137	1.148	1.067	1.077	1.022	1.022	1.011	1.009
J_5	Peak cable tension	0.362	0.271	0.275	0.251	0.351	0.291	0.279	0.281
J_6	Peak deck displacement	9.049	7.249	5.650	3.107	11.218	9.278	7.135	3.610
J_7	norm base shear (X)	0.623	0.470	0.667	1.034	0.662	0.535	0.759	1.244
	norm base shear (Z)	1.132	1.139	1.098	1.152	1.103	1.099	1.067	1.102
J_8	norm shear at deck (X)	2.140	1.489	1.627	1.543	2.196	1.588	1.738	1.707
	norm shear at deck (Z)	1.002	1.003	0.999	0.999	1.037	1.037	1.020	1.021
J_9	norm base moment (X)	1.490	0.805	1.100	0.878	1.582	0.934	1.263	1.049
	norm base moment (Z)	1.100	1.105	1.074	1.108	1.083	1.080	1.055	1.078
J_{10}	norm moments at deck (X)	3.560	1.949	2.086	1.642	3.771	2.271	2.333	1.920
	norm moments at deck (Z)	1.216	1.216	1.012	1.013	1.048	1.049	1.027	1.026
J_{11}	norm of cable tension	0.058	0.030	0.034	0.037	0.052	0.032	0.037	0.042
J_{12}	peak control force (X)	0.014	0.002	0.019	0.005	0.014	0.002	0.019	0.004
	peak control force (Z)	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
J_{13}	max device stroke (X)	3.922	3.142	2.465	1.725	4.897	4.050	3.168	1.681
	max device stroke (Z)	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
J_{16}	number of control device	24	24	24	24	24	24	24	24

From the results presented in Table 1, it can be deduced that isolation can substantially reduce the seismically induced responses in the bridge. Considering all the cases in this study, VCFPS is most consistent and robust isolator. Higher values of μ than the normal is beneficial for securing the displacement and base shear response of the bridge for VCFPS The control force produced by this variable sliding isolator to reduce the seismic response bridge is much less than that of FPS. So we can conclude that the performance of VCFPS is better than FPS.

9. CONCLUSIONS

In this study, an attempt is made to present benchmark cable-stayed bridges with various variable sliding seismic isolators. The performance of the bridge under different sliding isolators is investigated using the hysteretic model and the results are tabulated in the form of evaluation criteria’s mentioned in the benchmark problem for direct comparison. Parametric studies have been carried out by varying the important parameters of each isolator to find out the optimum value. Based on the investigation performed on the seismic response control of the bridge, the following conclusions are drawn:

1. Despite being a flexible structure, significant seismic response reduction of the bridge can be achieved by installing VCFPS in the benchmark cable-stayed bridge.
2. Reduction in the base shear response of the towers is achieved about 42 to 68% for all the types of specified isolator and earthquake ground motion.
3. The reduction of the seismic responses depends on the types of isolator as well as types of earthquake ground motions.
4. Initial isolation time period has a significant effect on the seismic responses and there exist an optimum value for each isolator but that value again depends on the types of earthquake ground motion.
5. Comparing the values of evaluation criteria presented, it can be deduced that VCFPS is more robust and the performances of the variable sliding isolators are better than traditional FPS.

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