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SEISMIC RESPONSE OF MULTI-SPAN HIGHWAY BRIDGE: EFFECTIVENESS OF USING ISOLATION SYSTEM

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ABSTRACT

Seismic response of isolated bridge under several earthquake ground motions is presented in this paper. Non-linear time history analysis is carried out for both non-isolated and seismically isolated bridge. Lead rubber bearing is employed to observe the isolated bridge behavior. Takeda trilinear model is used to model the mechanical behavior of the pier and simple elastic model is applied to model remaining portion of the bridge. Four different earthquake ground motions are considered and applied at the longitudinal direction of the bridge. It is found that the effectiveness of base isolation technique is highly influenced by the properties of the isolated bridge as well as by the ground motion. It is also revealed that to understand and design efficient isolated bridge system, analyzing the ground acceleration in frequency domain could be more efficient than time domain approach.

Keywords: Time history analysis; isolated bridge; response spectrum; power spectrum density (PSD).

1. INTRODUCTION

Bridges are one of the most important structures that plays a vital role in transportation network system and should be keep continuing during post disaster period. However, over the last three decades, bridges are suffering from damages under earthquake and causing huge economic losses [1-3]. It was mainly short to medium span bridge with short pier those are suffering from damages, as their fundamental period of vibration remains within the dominant period of vibration of the earthquake motion. It is already established that the restriction of the transmission of earthquake energy and force to the structure is a more promising approach rather stiffening the structure [4-5]. The first

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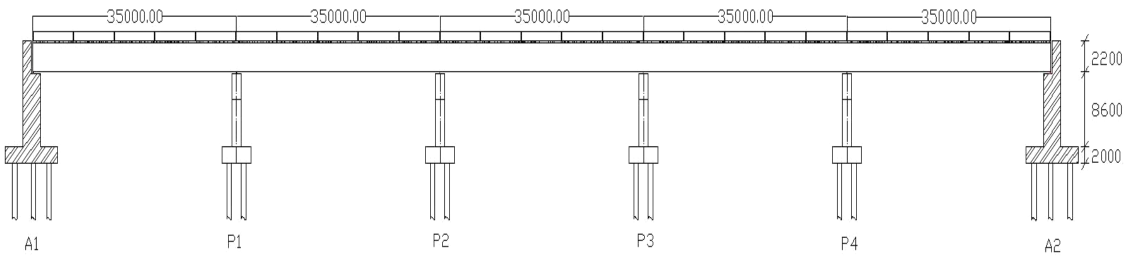
technique named as seismic isolation system [6] has got wide range of popularity and widely used in different countries of the world. Thus, a bridge is protected against damage from the earthquake by limiting the earthquake effects rather than resisting it [7-8].

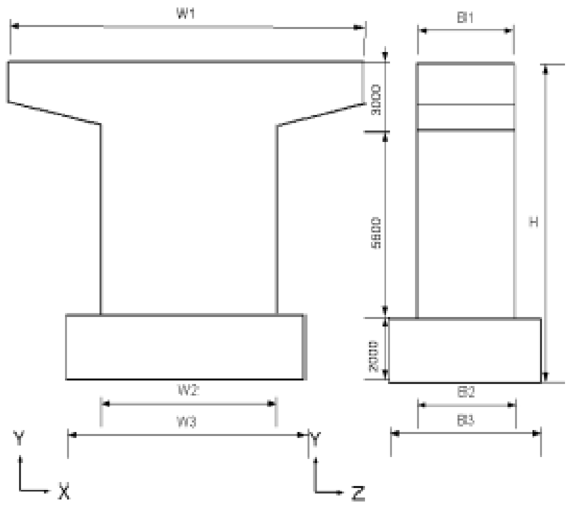
Different types of rubber bearings are available to employ in isolation system: High damping rubber bearing (HDRB), Lead rubber bearing (LRB) and natural rubber bearing (NBR). Among these three, LRB is the most effective one to reduce the response of bridge against earthquake [9] and have been considered in the present investigation also. Laminated rubber consists of several layers of rubber bonded to thin steel shims. Steel shims are provided to prevent the excessive lateral deformation of the rubber layer where the whole device provides enough vertical stiffness to bear the bridge load.

Several past studies have confirmed the effectiveness of this isolation bearing to improve the seismic performance. Ghobara and Ali [10], Turkington et al. [11-12] and Jangid [13] have shown the suitability and effectiveness of LRBS in reducing earthquake response. The main concept was just to increase the natural period of vibration of the non-isolated bridge, which is already been discussed. But alteration of the natural time period may not be always effective depending on the type of earthquake, the structure is going to face.

The current study is devoted towards evaluating the seismic response of multi-span continuous highway bridge isolated with lead rubber bearings (LRB) under four different earthquake ground motions. To this end, the responses of the non-isolated bridge is also determined and compared with isolated bridge. It was found that under certain ground motions the response under isolated condition could be higher than non-isolated condition. It is also observed that the percent reduction of response under isolated condition could be half of the normal reduction range under certain condition, though same isolation properties is taken into consideration. At last it is shown that Power spectrum density (PSD) of the ground acceleration could be an alternate of the Response spectrum (RS) and used to explain this phenomenon.



(a)



(b)

Figure 1. Physical model of a five-span continuous seismically isolated highway bridge (a) longitudinal sectional elevation of the bridge, and (b) transverse sectional elevation of the bridge; all dimensions are in [mm] [14].

2. MODELING OF THE BRIDGE

2.1 Physical Model

A typical five-span highway bridge with 35000 mm span as used in this study is shown in Figure. 1 (a). The superstructure consists of 280 mm continuous composite slab with 100 mm of asphalt supported on two continuous steel girders. The depth of the continuous steel girder is 2200 mm. The substructure of bridge consists of rigid abutments at the two ends and four intermediate reinforced concrete piers. The footings are supported on pile foundation. Figure. 1(b) shows typical cross section of the bridge. Lead rubber bearing (LRB) is used for isolation mechanism. The dimensions and material properties of the bridge deck, piers with footings are given in Table 1 and those of the isolation bearings are presented in Table 2.

2.2 Analytical Model

The analytical model of the bridge is shown in Figureure 2. The entire bridge is approximated by a 2-D model bridge. The bridge deck is idealized as a rigid body ignoring flexibility of the bridge deck. The piers are restricted to participate in energy absorption in the entire bridge system to some extent in addition to the isolation bearings. The secondary plastic behavior is expected to be lumped at bottom of each pier where plastic hinge is expected to occur. The plastic hinge of the pier is modeled by a nonlinear link element. The nonlinear link elements are modeled using the Takeda tri-linear model [15].

Table 1: Geometric and material properties of the bridge

Properties	Specifications
	Piers with LRBs
Cross-section of the pier cap (mm^2), (B1x W1)	1800x9000
Cross-section of the pier body (mm^2), (B1xW2)	1500x5000
Cross-section of the footing (mm^2), (B3xW3)	5000x8000
Number of piles in each pier	4
Young's modulus of elasticity of concrete(N/mm^2)	25000
Young's modulus of elasticity of steel (N/mm^2)	200000

Table 2: Properties of the isolation bearings

Dimension	Specifications
	LRBs
Length (mm)	650.0
Width (mm)	650.0
Thickness of rubber layers (mm)	81.3

Table 3: Parameters of the bilinear model for LRB

Effective Stiffness, (kN/mm)	16.347
Initial Stiffness, K_1(kN/mm)	69.665
Post yield ratio	0.1655
Yield Strength(kN)	962.7

This model works on three connecting curves standing for un-cracked, cracked and post-yielding stages. The primary plastic behavior of the bridge is expected to occur at the isolation bearings. The isolation bearing are modeled using the nonlinear shear elements. In order to describe the mechanical behavior of isolation bearing, the bilinear model as specified in JRA [7] with characteristics properties given in Bhuiyan [14] was used in the study. In this case, three parameters are required to represent the nonlinear mechanical behavior (hysteresis loop) of the bearings: initial stiffness k1, post yield stiffness k2 and the yield strength of the bearings qd as shown in Figure 3. In the subsequent numerical study, these parameters are assigned. The property of the LRBs is presented in Table 3. The steel girder, the pier cap, the pier body, the footing, and the two ends of the plastic hinge are modeled using the simple elastic frame elements.

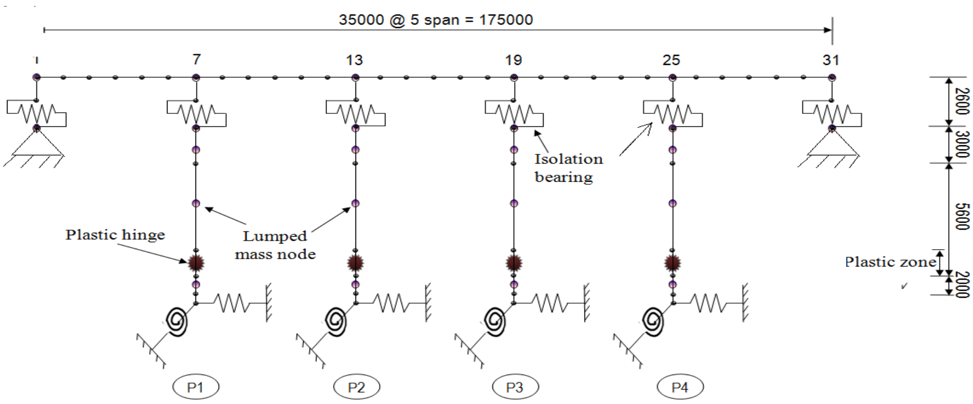


Figure 2. Analytical model of the bridge

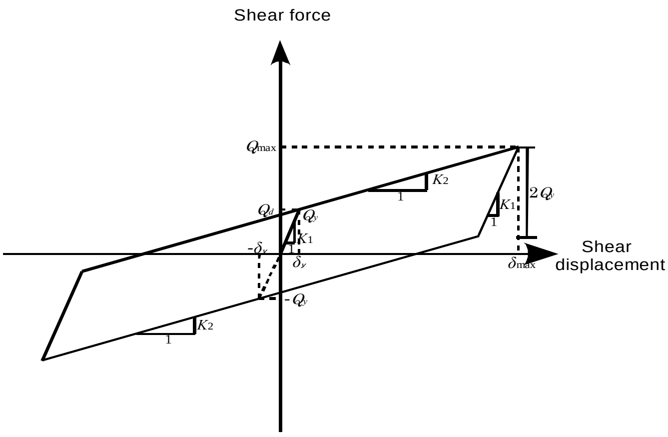


Figure 3. Bilinear force-displacement relationship of the bearings [7]

3. EQUATION OF MOTION

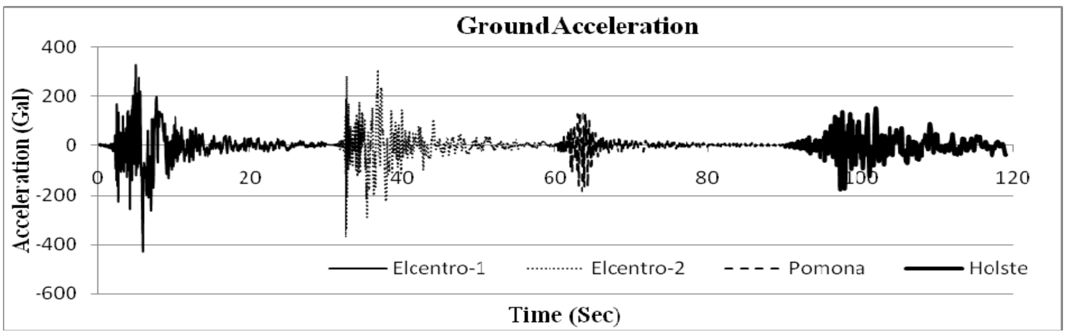
Equations that govern the dynamic response of the bridge can be derived by considering the equilibrium of all forces acting on it using the d'Alermbert's principle. In this case, the internal forces are the inertia forces, the damping forces, and the restoring forces, while the external forces are the earthquake induced forces. The equations of motion in incremental form can be written as

$$[M]\{\Delta\ddot{U}\}_{t+\Delta t} + [C]\{\Delta\dot{U}\}_{t+\Delta t} + [K]\{\Delta U\}_{t+\Delta t} + \{\Delta R\}_{t+\Delta t} = \{P\}_{t+\Delta t} - [M]\{\ddot{U}\}_t - [C]\{\dot{U}\}_t - [K]\{U\}_t - \{F_b\}_t - \{F_s\}_t \quad (1)$$

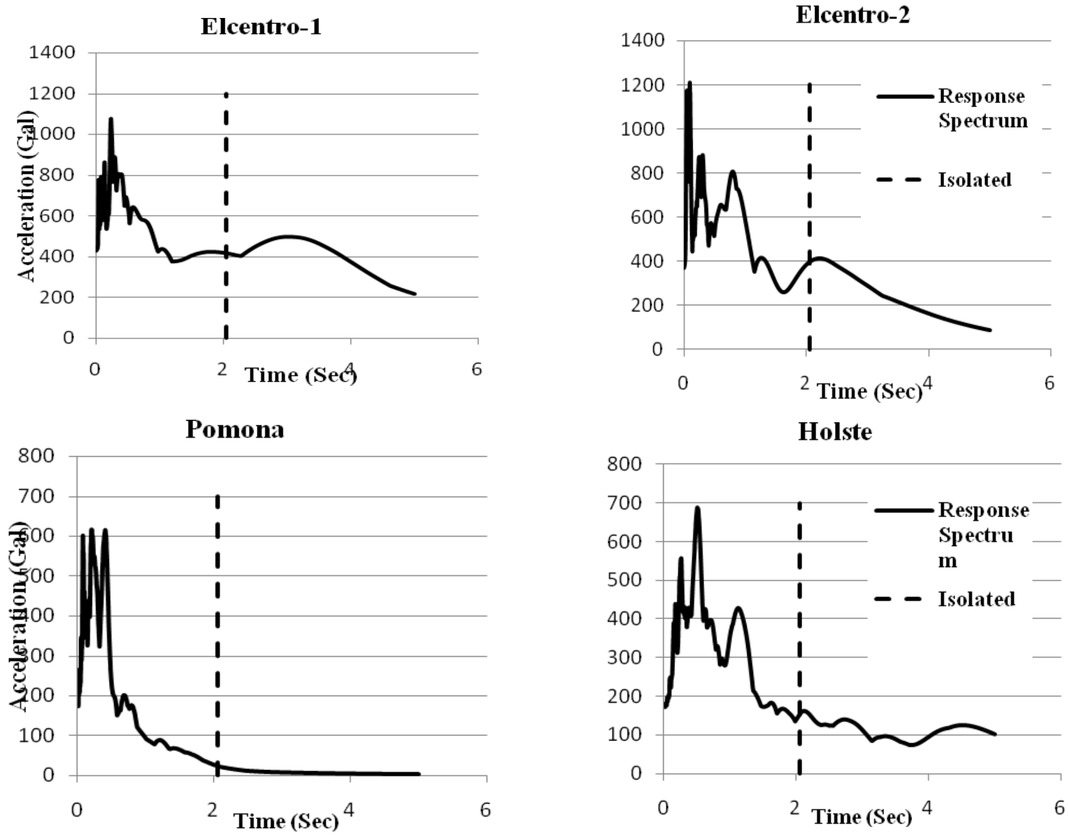
where $[\mathbf{M}]$ is the mass matrix; $[\mathbf{C}]$,the damping matrix; $[\mathbf{K}]$, the tangent stiffness matrix; $\{\Delta\mathbf{U}\}_{t+\Delta t}$, the vector of the increment of displacement over the time integration; $\{\Delta\dot{\mathbf{U}}\}_{t+\Delta t}$, the vector of the increment velocity over the time increment; $\{\Delta\ddot{\mathbf{U}}\}_{t+\Delta t}$, the vector of the increment of acceleration over the time increment; $\{\mathbf{U}\}_t$ the vector of the displacement at the beginning of the time step t; $\{\dot{\mathbf{U}}\}_t$, the vector of the velocity at the beginning of time step t; $\{\ddot{\mathbf{U}}\}_t$, the vector of the acceleration at the beginning of time step t; $\{\mathbf{F}_s\}_t$, the internal force of the bridge excluding isolation bearing at the time step t; $\{\Delta\mathbf{R}\}_{t+\Delta t}$, the total unbalanced force vector, and $\{\mathbf{P}\}_{t+\Delta t}$, the external force vector at the end of time step $t+\Delta t$; $\{\mathbf{F}_b\}_t$, the internal force vector derived from the isolation bearings at the beginning of the time step t. A solution algorithm [14] comprised of the solution of equations of motion using the unconditionally stable Newmark's constant-average-acceleration method and the solution of the differential equation governing the strain-rate dependent behavior of isolation bearings is used as the time integration scheme. Furthermore, the Newton-Raphson iteration procedure consisting of corrective unbalanced forces is employed within each time step until equilibrium condition is achieved.

4. EARTHQUAKE GROUND MOTIONS

Four different earthquake time histories are considered in the subsequent analysis. The first two ground motions refer to the 1940 El-Centro earthquake occurred Huston Road in California, with different PGA values. The third and last one refer to the 1906 Holste earthquake in Hollister and 1992 Pomona earthquake in California correspondingly. Figure 4(a) shows typical ground acceleration time histories for four earthquake ground motions. The characteristic properties of the ground motion are given in Table 4. Response spectrum of these all four ground motions are evaluated and shown in Figure 4(b). Frequency domain approach is also adopted in this paper as each of the earthquakes consists of wide range of frequency. Figure 4(c) represents the power spectrum of the four considered earthquake. Fourier transform with hanging window was used to convert the time domain ground acceleration data to frequency domain data. The dotted line in the figure shows the location of the time period and frequency of the isolated bridge in the Figure 4(b) and (c) respectively.



(a)


Elcentro-1
Acceleration (Gal)
Time (Sec)
Elcentro-2
Response Spectrum
Isolated
Time (Sec)
Pomona
Acceleration (Gal)
Time (Sec)
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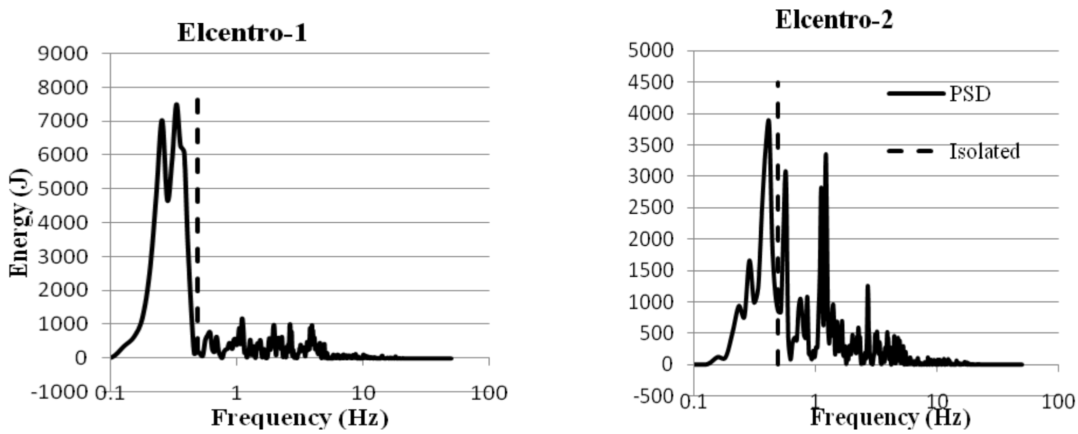
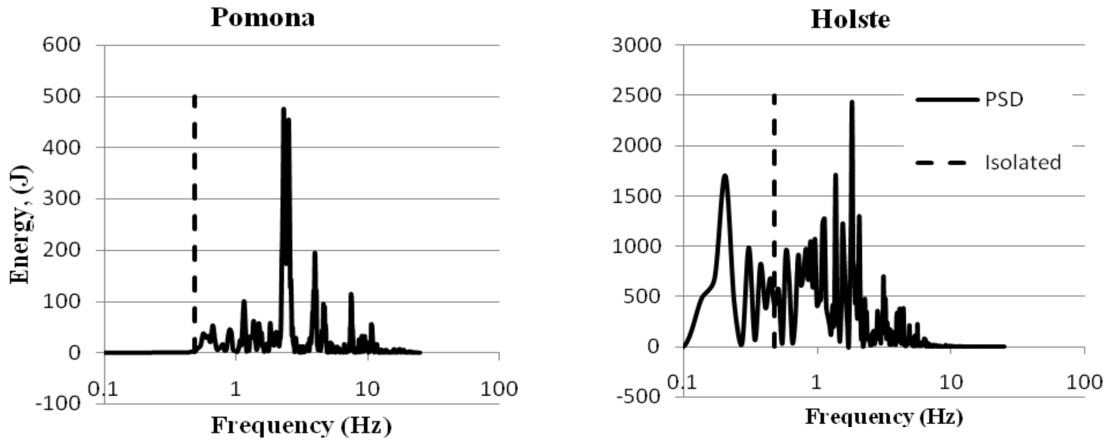
(b)


Elcentro-1
Energy (J)
Frequency (Hz)
Elcentro-2
PSD
Isolated
Frequency (Hz)



(c)

Figure. 4 Earthquake ground motion, (a) acceleration-time history and (b) acceleration response spectrum (c) acceleration power spectrum.

5. SEISMIC RESPONSE OF THE BRIDGE

The modal characteristics of the both isolated and un-isolated bridge are shown in Table 5 and 6. It is clearly observed that the isolation technique shifts the time period as well as the frequency of the structure significantly. However, the mode characteristics under both of these situations remain unchanged. It is mainly the first mode that is mainly affected by the isolation technique, higher the mode, lesser the alteration of modal time period.

Each of the earthquakes consists of a wide range of frequency; alteration of frequency of the structure may not be effective for all types of earthquake. The response of the isolated structure will be different for different earthquake. As can be seen from Figure 5 (a), (b) and (c) that the isolated response (Iso) as compared to non-isolated condition (Non) under first two earthquakes, is much larger than the response under last two ground motions. Especially for the case of deck acceleration the response of the isolated bridge under Elcentro-2, is very close to the response of the non-isolated one. However, the non-isolated response of the all four earthquake is not so different. Moreover, the peak responses of the bridge are grasped in Table 7; it is easily apprehensible that the percent reduction of the response under first two earthquakes is much lower than the last two earthquakes. In some cases the percent reduction of response is as high as 80% (Pomona) where as in some other cases it around 6% (Elcentro-2) only. Although from Figure 4(b) it is seen that for the all four earthquakes the time period of the isolated structure is far from the peak zone and doesn’t bear any special characteristics for first two earthquakes. So it is difficult to explain this behavior from time domain approach.

Power Spectrum Density (PSD) of all these four ground motions are already plotted in Figure 4(c). It is tried to elucidate the aforesaid phenomenon from the frequency domain representation. For the case of first two earthquakes the frequency of the isolated structure is very close to the peak frequency of the earthquake where the maximum energy concentrates and for the case of last two earthquakes which exists far beyond of that peak frequency. As the frequency of the isolated bridge is close to the maximum energy containing frequency of the earthquake, it has higher responses under these first two earthquakes. Though PSD of the earthquake ground motions may have two and three peak frequencies also can be confirmed from Figureure 4(c), second and third peak may also be able to influence the response of the isolated bridge. In future work, such kind of investigation would be explored.

6. CONCLUSION

Seismic response of both isolated and non-isolated bridge is determined under four earthquake ground motions. All four earthquakes are applied at the longitudinal direction of the bridge. To determine the responses, the whole structure is defined by finite element model and non-linear time history analysis is performed. Bearing is modeled by employing bilinear model, following the specification of manual for highway bridges [7]. In this paper, the bridge responses are discussed in terms of the base shear of the pier, deck acceleration, displacement of deck & pier top force. It can be concluded that for efficient isolation system it not only depend on the properties of the non-isolated condition but also on the isolated condition. If the isolated frequency remains near the peak frequency of the earthquake then the efficiency of the isolation system reduces significantly. Moreover PSD could be an alternate of the response spectrum to understand the response of the structure on a better way.

Table 4: Properties of the ground motions

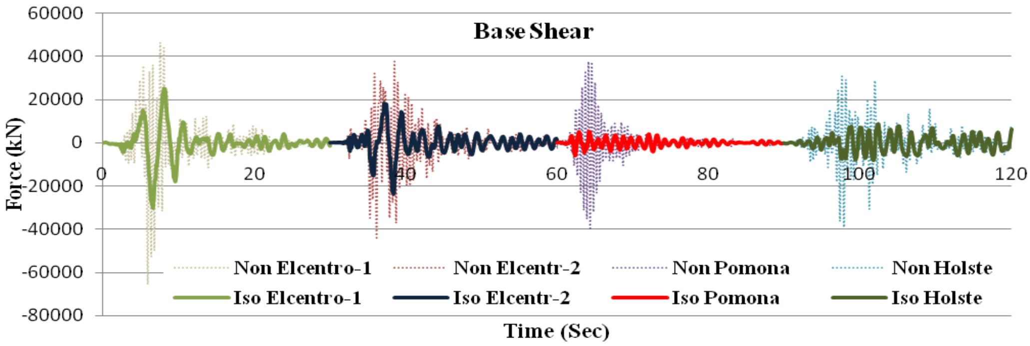
Name of the Earthquake	Maximum Acceleration (Gal)
Elcentro-1	+325.4 -428.09
Elcentro-2	+305.92 -368.67
Pomona	+145.04 -182.12
Holste	+150.83 -174.55

Table 5: Modal data of un-isolated bridge

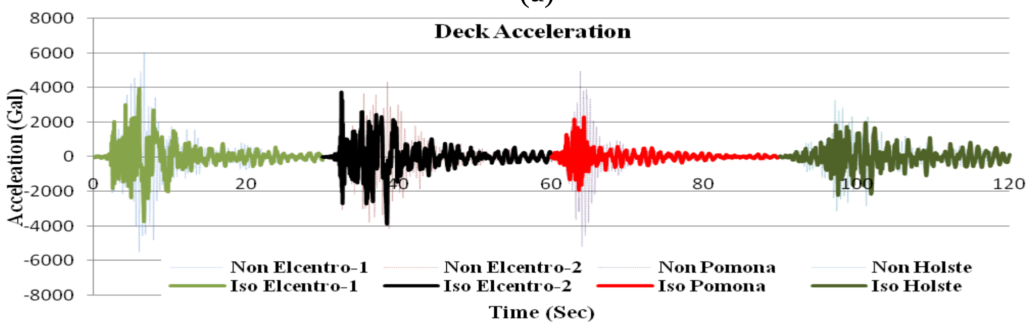
Mode No.	Time Period (Sec)	Frequency (Hz)	Mode Characteristics
Mode -1	0.35632	2.806	Translation
Mode -2	0.07529	13.282	Translation
Mode -3	0.05098	19.615	Bending
Mode- 4	0.05003	19.988	Bending
Mode -5	0.04118	24.284	Translation

Table 6: Modal data of isolated bridge

Mode No.	Time Period (Sec)	Frequency (Hz)	Mode Characteristics
Mode -1	2.050	0.4878	Translation
Mode - 2	0.158	6.329	Translation
Mode -3	0.156	6.410	Bending
Mode- 4	0.155	6.451	Bending
Mode -4	0.154	6.49	Translation



(a)



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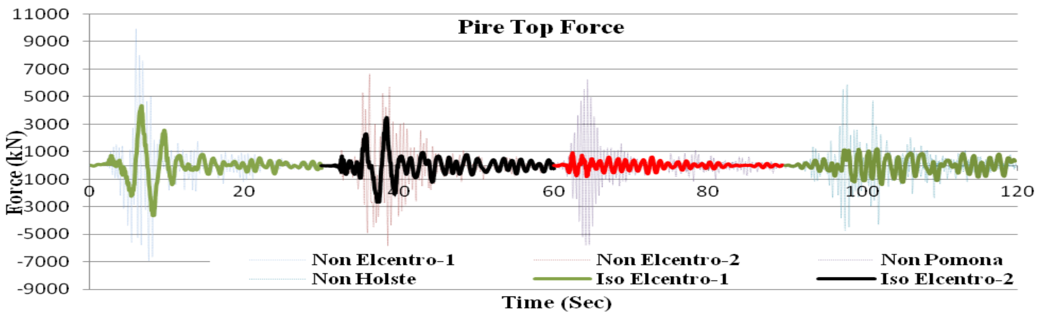

(c)

Figure 5 (a) Time variation of pier base shear for both isolate and non-isolated bridge (b) Time variation of deck acceleration for both isolated and non-isolated bridge (c) Time variation of pier top force for both isolated and non-isolated bridge. None indicates non-isolated bridge and Iso indicates isolated bridge.

Table 7: Peak Response Quantities of the Bridge for both Isolated and Non-isolated

Condition

Response	Non-isolated	Isolated	Percent Reduction	Remarks
Base Shear (kN)	+46468.26	+25426.78	**45.28**	Elcentro-1
	-65546.7	-30142.82	**54.13**	
	+37926.99	+18276.27	**51.81**	Elcentro-2
	-44194.00	-23772.00	**46.21**	
	+37891.00	+5244.92	86.15	Pomona
	-40581.20	-5961.48	85.30	
	+30745.97	+8816.44	71.32	Holste
	-39261.89	-7877.17	79.93	
Deck Acceleration (mm/sec^2)	+6022.18	+3930.30	**34.73**	Elcentro-1
	-5505.09	-3719.29	**32.43**	
	+4349.53	+3735.62	**14.11**	Elcentro-2
	-4142.53	-3882.84	**6.26**	
	+4955.99	+2268.68	54.22	Pomona
	-5158.07	-1937.34	64.22	
	+3262.42	+1966.81	39.71	Holste
	-3133.72	-2216.99	29.53	
Pier Top Force (kN)	+9952.47	+4337.91	**56.41**	Elcentro-1
	-6954.37	-3662.06	**47.34**	
	+6671.37	+3508.12	**47.41**	Elcentro-2
	-5803.27	-2659.97	**54.16**	
	+6297.62	+948.457	84.93	Pomona
	-5810.26	-819.38	85.89	
	+5895.52	+1226.06	79.20	Holste
	-4702.48	-1377.13	70.71	

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