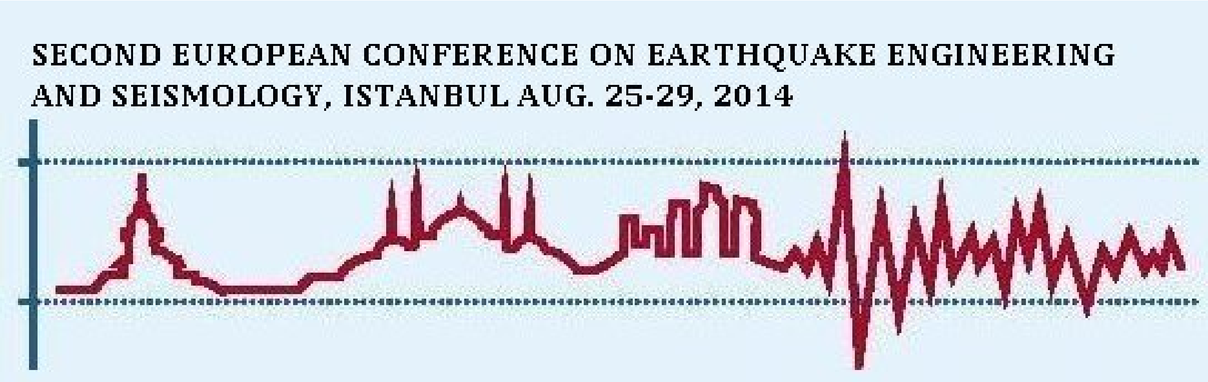

Dynamic Behavior Evaluation of a Continuous Concrete Box Girder Bridge with Seismic Isolator in Near-field Earthquake

Hamid Reza Ebrahimi Motlagh[1] and Alireza Rahai[2]

ABSTRACT

Ground motions in an area close to the failure surface of strong faults in near-field earthquakes, due to having high-amplitude pulses of velocity, impose enormous energy on a structure in a short period of time, and put it under the influence of far more seismic demand. The philosophy of designing earthquake-resistant important structures is to achieve a structure that acts at the level of life safety under massive earthquakes with rather high recurrence interval. One of the methods to reduce forces imposed on bridge piers is the use of seismic isolation. The present article evaluates the use of Lead Rubber Bearing (LRB) as a seismic isolator in one typical bridge. Non-linear static analysis was carried out to compare its performance levels with a typical bridge without seismic isolation. Nonlinear time history dynamic analysis was performed using strong records of near- and far-field on models to compare dynamic behavior against strong ground motions. The results show that seismic isolation within reduces forces imposing on piers considerably, increases superstructure displacement and by Reducing seismic demand level, improves capacity to demand ratio on the structure.

Keywords: Lead Rubber Bearing (LRB), seismic isolator, near-field earthquake, nonlinear static analysis, nonlinear time history dynamic analysis

1. INTRODUCTION

Major earthquakes occurred during recent century led to gain more knowledge about the energy imposed on a structure during an earthquake. Especially after San Fernando earthquake of 1971 and Parkfield earthquake of 1966, seismologists and seismic engineers observed more serious damages imposed on the structures located near the failure surface of fault. The incidents occurred in Kobe earthquake of 1995 and Northridge earthquake of 1994 also confirmed above observations (Greg et al.,2002 & Housner et al., 1967). Many highway bridges were damaged seriously due to these earthquakes. Such incidents revealed that seismic codes had not covered as well as complexity of their seismic behavior during several decades ago (US-Japan Workshop, 1998). Due to having more limited frequency content in higher frequency values, near-field earthquakes are differentiated from far-field earthquakes (Hall et al., 1995). Such specifications in near-field earthquakes change behavior of a structure from mode-like state into wave-like state. In this state, structure behavior is induced by accumulation of the effects of the waves passing through a construction (Iwan., 1995). Due to accumulation of shear waves on the direction of fault rupture, when these waves are propagating toward a station and/or site as fast as ground rupture velocity, they make a pulse-like motion with

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long-period at the beginning of the records registered in the stations near the fault (Malhota., 1999). The pulse-like motions of ground and a greater ratio of PGV/PGA in near-field earthquake have a greater sensitive width than acceleration in elastic response spectrum. Due to energy transfer in short time caused by long pulses, circumstance of nonlinear hinges in a construction is changed and nearly most energy may be absorbed in the first hinge (Kikuchi et al. 2000). Moreover, this value can be recognized as a key parameter to control construction response in a near-field earthquake and demand for base shear and displacement in a structure with seismic isolation are affected by this parameter (Liao et al., 2012).

One of the methods to improve capacity to seismic demand ratio in bridges is to increase the structure natural period and decrease structure base shear. It decreases seismic demand through increasing damping and departing from peak value of response area. This is important, especially for strong ground motions (WSDOT., 2012 & Park et al., 2002). Rubber with lead core damps part of earthquake energy by departing the structure response from resonance area and other part by yield mechanism (Park et al., 2002). Lead has a considerable ability to deform plastic forms, so that its plastic deformation at 20°C is equal to plastic deformation of steel at a temperature higher than 450°C. Lead also has a favorable resistance against cyclic loads. Therefore, it has a high resistance against fatigue (Robinson., 2011).

According to AASHTO code, the structures with high and very high importance, such as bridges, should be designed and constructed in a manner in addition to have performance level of immediate occupancy under an earthquake with hazard level No. 1 (with recurrence interval of 475 years), equivalent to design earthquake, the structures should maintain their occupancy with minimum destruction under the influence of maximum considered earthquake or hazard level No. 2 (with recurrence interval of 2475 years) (AASHTO., 2002). Structure behavior indicates its capacity reply to seismic requirement. After achieving a structure capacity curve, a performance point is obtained where it meets an earthquake demand curve. The performance point is an estimation of the real displacement of a structure for a certain earthquake. This point can be used to classified the damage to the structure and compare it with the corresponding performance objectives (ATC40., 1996).

The present article compares seismic behavior of a typical bridge with continuous deck in two models, with/without lead rubber bearing (seismic isolator). A three-span continuous concrete box girder bridge was selected for this purpose. Bridge loading was carried out based on Iran Bridges Loading Code (Standard No. 139). It was designed using American Association of State Highway and Transportation Officials (AASHTO) code (AASHTO., 2002 & Standard Loads., 2005).

For the isolated bridge, the bearing system in longitudinal direction includes sliding bearings in two abutments and seismic isolator as lead rubber bearing (LRB) on piers. Bearings system in longitudinal direction on the non-isolated bridge includes sliding bearing in two abutments and moment hinges above piers.

To evaluate performance level of the structure and the effect of seismic isolation on it, the nonlinear static analysis (Pushover) was performed on the models. Moreover, some near- and far- field important records were selected to compare the effect of earthquakes record in a near field to the fault and earthquakes far from failure surface in dynamic response of the system.

2. SPECIFICATIONS AND BRIDGE MODELING

The models include two typical three-span bridges with continuous box deck. Lengths of spans were selected as 24, 32, and 24 meters, respectively. Figure (1) shows longitudinal view of two bridges. Figure (2) shows transvers section of deck, pier, and cross section of piers column. There are different deck bearings on piers in the models. The steel ratio for the piers is 2%. To compare better the seismic isolattion performance, all dimensions and sizes in the models were selected identically. Figure (2) shows dimensions of lead rubber bearings used in the isolated model. Area of rubber is achieved according to AASHTO code based on permissible stresses under service and earthquake loading. The criteria concerning shear strain in different loading states were controlled (AASHTO., 2002). Isolators design forces were modeled with respect to the lateral force-displacement curve shown in Figure (2) and considering 7% preliminary stiffness for secondary stiffness (Constantinou et al., 2011).

The piers were considered to be on rigid foundation. Also, to study the effect of pier bearings type, longitudinal movement is free on abutments. As the abutments do not participate in seismic bearing, dynamic interaction of the structure and soil was ignored and the roller support was used on abutments. In this research, only longitudinal movement of deck was considered.

Mander model were used to determine stress-strain relationship for confined and unconfined concrete of piers (Caltrans., 2004).

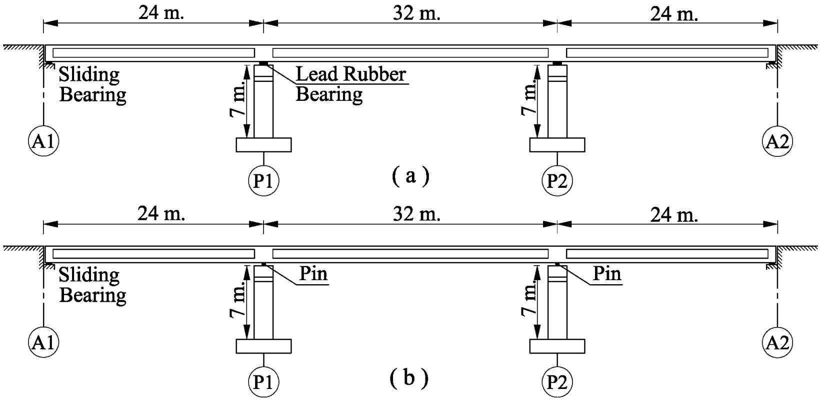

Figure 1. Longitudinal view of bridge; a) the bridge with seismic isolator; b) the bridge without seismic isolator

Specified concrete compressive strength for the piers and deck and yield stress of the used reinforcements were considered as f'_c=35 MPa and f_y=400 MPa, respectively. The models were made in 3D forms using SAP 2000 (v.14.2.4) program (CSI., 2012). The models encompass all components of the bridge and boundary conditions. Therefore, bilinear model with Link element in Rubber Isolator program was used for LRB modeling.

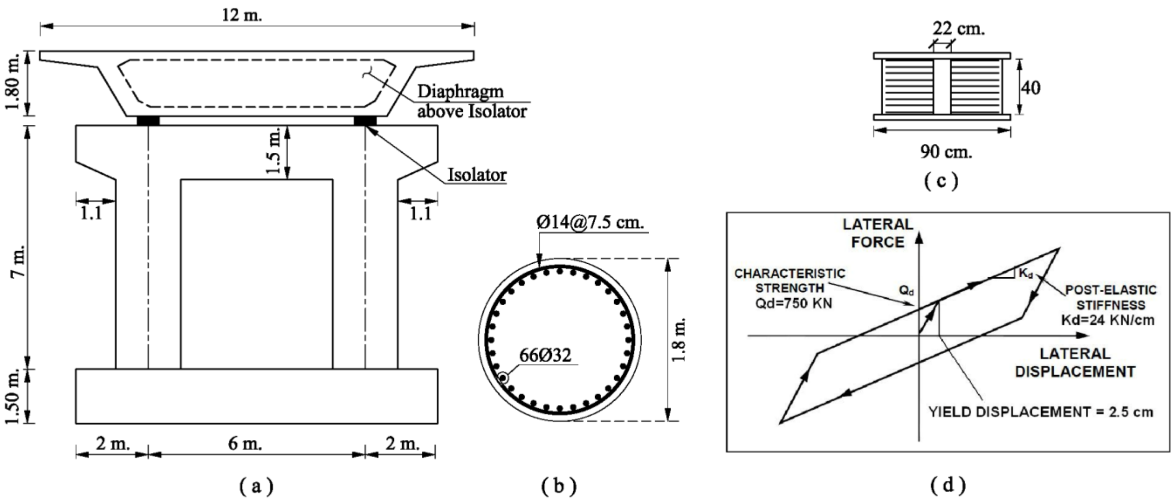

Figure 2. a) Details of transvers section of deck and pier, b) Cross section of piers column, c) Detail of lead rubber bearing, d) Hysteresis model of lateral load-displacement of LRB

As damage is caused by nonlinear responses, the only method to estimate damage and real behavior of a structure in strong earthquakes is to take nonlinear behavior of a structure into consideration. Generally, two nonlinear behavior, including nonlinear behavior of materials and nonlinear geometrical behavior in modeling were considered.

ATC-32 code proposes to model at least 3 elements along a column as linear elastic (Aviram et al., 2008, ATC-32., 1996). Superstructure elements are modeled using linear beam-column element with reinforced concrete cracked materials. According to SDC 2004 code, effective moment of inertia of superstructure with concrete box girder depends on crack expansion and effect of cracks on elements stiffness. Therefore, according to this reference, the effective moment of inertia (I_{eff}) of concrete box girder with respect to the type of concrete material was used as 0.75 of moment of inertia of its section (Caltrans., 2004).

In seismic analysis, the effective moment of inertia of column is used along its length. Also, torsional stiffness of concrete elements decreases considerably after cracking; torsional moment of inertia of columns should be reduced as per Eq. (1). In this equation, J_{eff} and J_g are effective torsional moment of inertia and torsional moment of inertia of column section, respectively (Aviram et al., 2008).

$$J_{eff} = 0.2 J_g \tag{1}$$

According to SDC 2004, to estimate expected shear capacity of column, shear area of column is reduced due to combining effect of bending and axial load. Therefore, modification factor of shear area for column elements is used in the analysis using Eq. (2). In this equation, $A_{v.g}$ and $A_{v.eff}$ are shear area and effective shear area of column, respectively.

$$A_{v.g} = A_{v.eff} \tag{2}$$

Plastic moment capacity of the column is achieved using moment-curvature analysis (M-Φ) based on expected characteristics of materials. Therefore, the obtained moment-curvature curve can be replaced by an ideal elastic-plastic curve to estimate plastic capacity of section. Effective bending stiffness of the column cross section for all its length is obtained using Eq.(3) where M_y, Q_y and E are moment, curvature at yield point of section and modulus of elasticity of column concrete, respectively.

$$I_{eff} = M_y / EQ_y \tag{3}$$

Nonlinear and hysteresis behavior is provided by plastic hinge at the bottom of columns where maximum moment exists, as per Figure (3). In the models, an approximate plastic hinge length was used according to Eq.(4) as per SDC 2004 code to convert plastic curvature into plastic rotation. In this equation, L, f_y and d_{bl} are member length between maximum and zero moments, which is considered in column length for cantilevered column, yield stress of longitudinal reinforcement of column and diameter of longitudinal reinforcements of column, respectively (Aviram et al., 2008 & Caltrans., 2004).

$$L_p = 0.08L + 0.022 f_y d_{bl} \geq 0.044 f_y d_{bl} \tag{4}$$

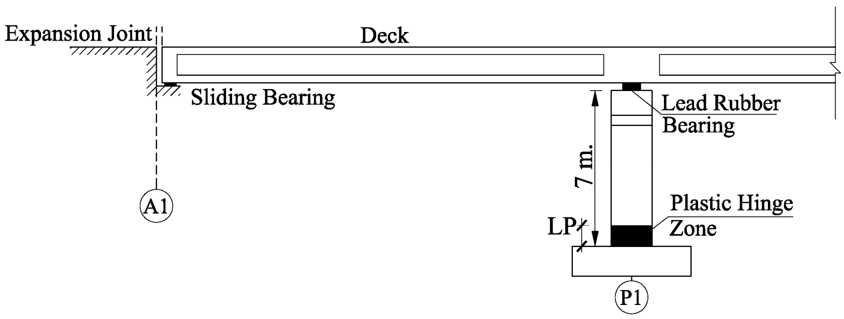

Figure 3. Layout of seismic isolator and position of plastic hinge of column in longitudinal view of the bridge

The P-M_2 and P-M_3 hinges model of Caltrans code were used in the program with the length of plastic hinge obtained from Eq. (4) for modeling plastic hinge of column.

3. EARTHQUAKE RECORDS

Nonlinear time history dynamic analysis was carried out using near- and far-field earthquakes on the designed bridge model to evaluate behavior of the structure under real earthquake. As regards the longitudinal movement of the bridge is discussed in the present research, only the horizontal records perpendicular to the fault direction were used.

Two criteria of distance from fault failure plate and existence of velocity pulses with high amplitude and high period were considered in selecting near-field records. To compare the structure behavior against the records registered in the areas far from the failure surface of fault, three far-field records were also selected.

Therefore, according to Table (1), the group of near-field earthquake includes Northridge earthquake of 1994 (Sylmar station), 1978 Tabas (Tabas station), 2003 Bam (Bam station), 1977 Naghan (Naghan station), and 1992 Landers (Lucerne station) and the group of far-field earthquakes records includes 1994 Northridge (Moorpark station), 1978 Tabas (Dayhook station), 1979 Loma (Anderson Dam station). Data of using accelerograms were prepared from Peer website affiliated to Berkeley University of California (PEER., 2013).

Table 1. Specifications of selected earthquake record

Group	Earthquake	Magnitude (Mw)	Distance (km)	PGA (g)	PGV (cm/s)
				Horizontal comp.TR	Horizontal comp.TR
Near-field	Northridge-1994 (Sylmar)	6.7	6.4	0.843	129.36
	Tabas-1978 (Tabas)	7.4	3	0.852	121.22
	Bam-2003 (Bam)	6.6	1	0.778	121.47
	Naghan-1977 (Naghan)	6.4	5	0.730	62.89
	Landers -1992 (24Lucerne)	7.4	1.1	0.720	97.6
Far-field	Tabas-1978 (Dayhook)	7.4	17	0.406	25.96
	Northridge-1994 (Moorpark)	6.7	28	0.292	20.49
	Loma Prieta-1979 (1652 Anderson Dam)	7.1	21.4	0.244	20.3

Accelerogram of the corresponding earthquake were applied to the structures by their real values without applying coefficients for scaling to the standard design spectrum. It was done so aiming to study the behavior of structures under the effect of earthquakes with real values. This is important, especially for the accelerograms registered in the near-field of a fault. It means that such accelerograms are registered at the near distances to the ground failure surface and they indicate real behavior of ground at the time of fault fails (Movahed et al., 2012).

4. DYNAMIC RESPONSES OF BRIDGES

The models were modeled based on the criteria discussed in section 2. Nonlinear time history dynamic analysis on models was carried out using input accelerograms introduced in section 3.

Figure (4) shows pushover curve of models. Figure (4) also shows spectral acceleration (S_a) versus spectral displacement (S_d) for different types of soils according to Iranian Code of Practice for Seismic Resistant Design of Buildings (Standard No.2800., 2004).

Pushover curve of Figure (4-a) shows that the force level of the structure with seismic isolation is at a lower level as compared with the structure without seismic isolation, especially in linear behavior section and the non-isolated structure is damaged in much smaller displacement. This confirms much greater damping of structure with seismic isolator.

Figure (4-b) shows that performance level of immediate occupancy and life safety in isolated bridge is greater than seismic demand. Whereas, such values in a structure without seismic isolation are less than seismic demand in soils type III and IV. It indicates that in hazard level No. 1 earthquake, before the real performance of structure at performance point, the non-isolation structure goes beyond life safety phase. This is not acceptable for a very important structure like an important bridge.

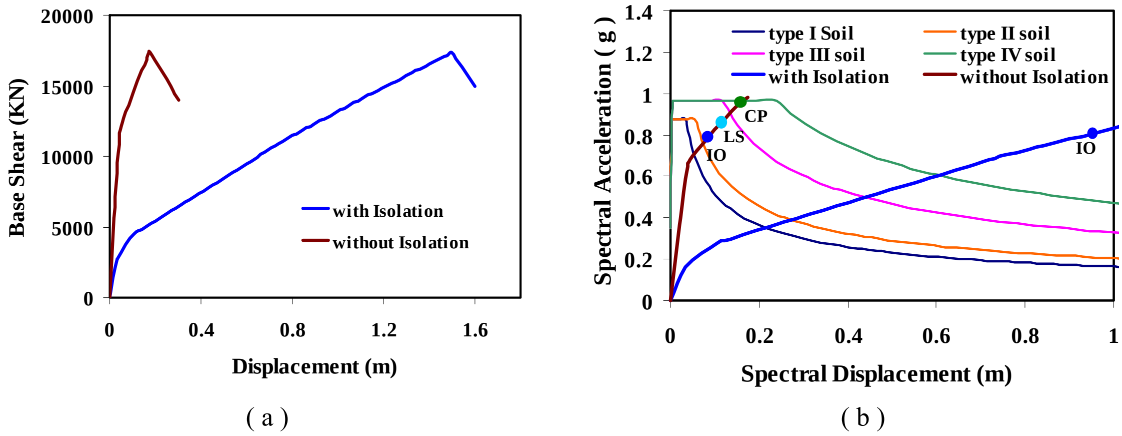

Figure 4. a) Pushover curve of models, b) spectral acceleration - spectral displacement curve of models in different soils

Figure (4-b) also shows that it is possible to reduce the demand imposed by an earthquake up to an acceptable level by reducing seismic demand rather than increasing seismic capacity. Therefore, by increasing the main period of structure using LRB, as the seismic isolation system, the structure is imposed under far less lateral force and also high dissipation of energy will be possible.

Table (2) shows the values of spectral acceleration in terms of g and spectral displacement in terms of meter (m) according to criteria of ATC-40 code at performance point for the two models in different soils.

Table 2. Spectral acceleration and spectral displacement at performance point

Soil	Performance Point			
	With Isolation		Without Isolation	
	S_a (g)	S_d (m)	S_a (g)	S_d (m)
Type I	0.298	0.142	0.684	0.046
Type II	0.325	0.175	0.705	0.054
Type III	0.402	0.292	0.756	0.072
Type IV	0.489	0.425	0.756	0.072

As Table (2) shows, when the structure main period in non-isolated bridge versus isolated one increasing from 0.55 to 1.45 Sec., spectral acceleration were reduced to the half amount in four types of soils on average and spectral displacement were increased averagely 3.75 times.

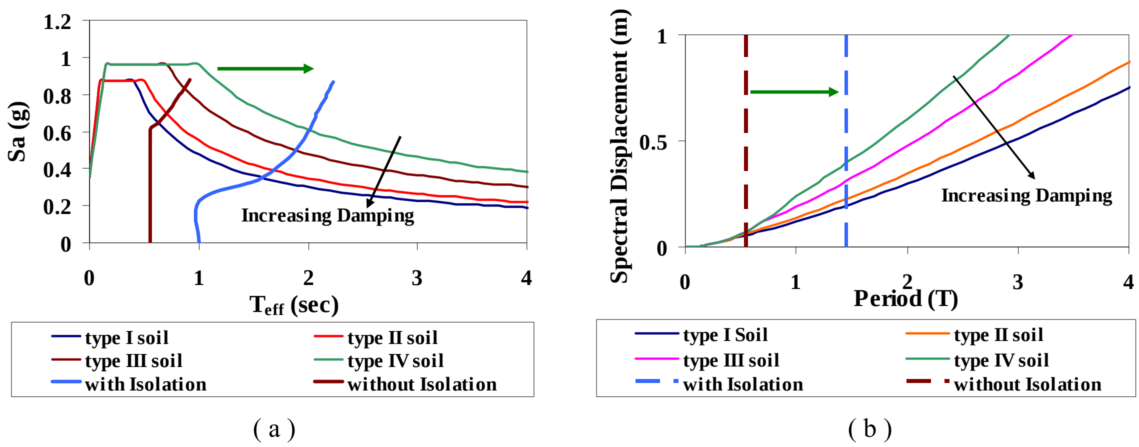

Figure 5. a) spectral acceleration-effective period curve in models, b) spectral displacement-natural period curve in different soils

By increasing damping, spectral acceleration level was decreased. It can be noted that this issue is in proportion to reduction of soil type in design spectra. Figure (5) shows, increasing natural period and/or effective period is followed by reducing level of demand spectral acceleration and increasing demand displacement.

Figure (6) shows time history of base shear formed in terms of Mega-Newton in the bridge with seismic isolator and the bridge without seismic isolator under earthquakes introduced in the earlier section. In all the cases, it shows reduction of base shear in the model with seismic isolator in near- and far-field earthquakes.

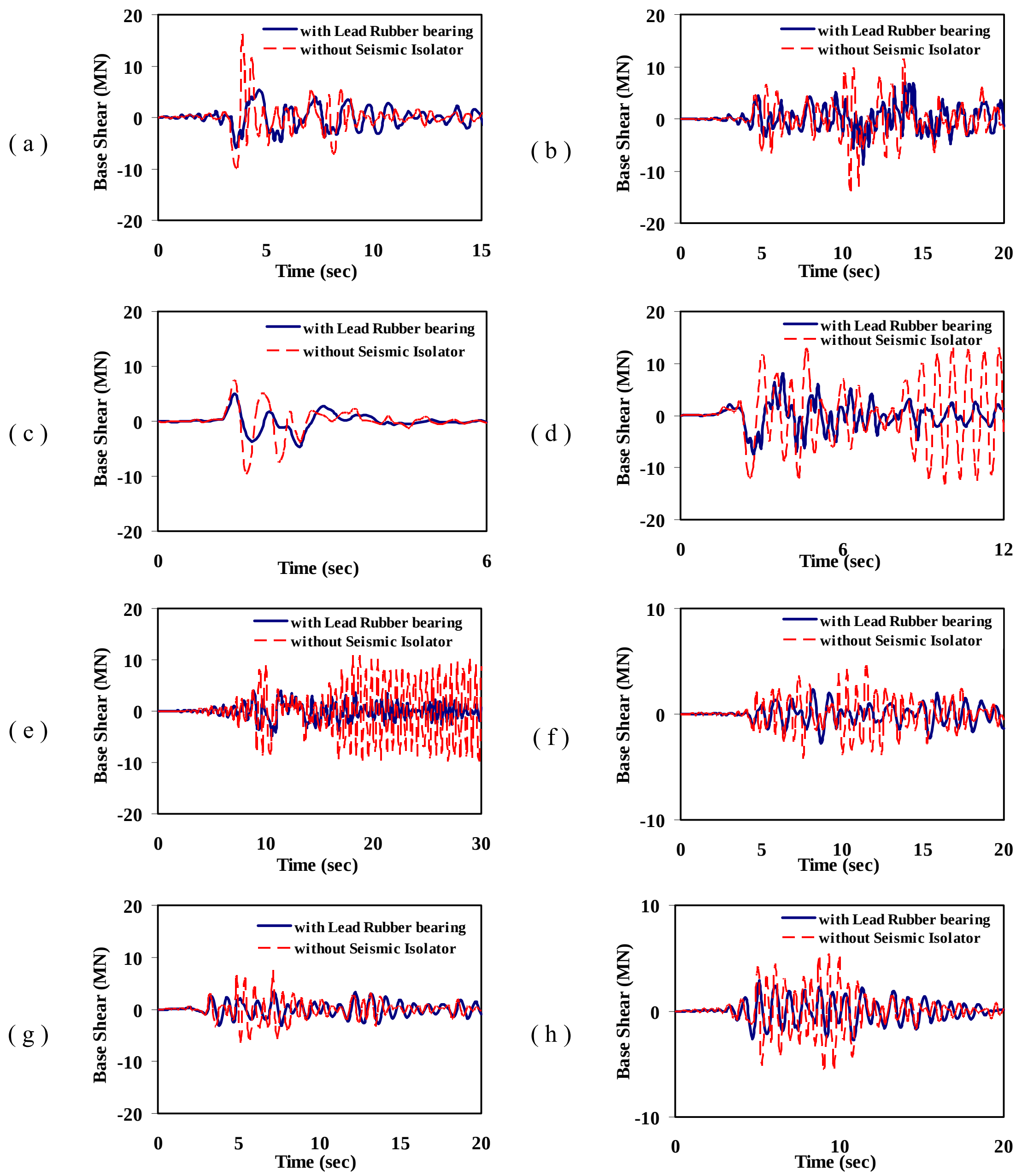

Figure 6. Time history of the base shear generated in the bridge with and without seismic isolation due to earthquake; a) Northridge (Sylmar), b) Tabas (Tabas), c) Naghan (Naghan), d) Bam (Bam), e) Landers (Lucerne), f) Northridge (Moorpark), g) Tabas (Dayhook), h) Loma (Anderson Dam)

Table 3. Maximum values of base shear and superstructure displacement in models

Earthquake	Base Shear (MN)		Superstructure Displacement (cm)	
	With Isolation	Without	With Isolation	Without
Northridge-1994-(Sylmar)	5.94	16.04	26.26	6.67
Tabas-1978-(Tabas)	8.80	13.70	25.85	6.40
Bam-2003-(Bam)	7.84	13.53	39.09	9.09
Naghan-1977-(Naghan)	4.93	7.42	5.825	4.11
Landers -1992-(24Lucerne)	4.74	10.90	13.86	5.32
Tabas-1978-(Dayhook)	3.37	7.52	5.55	3.40
Northridge-1994-(Moorpark)	2.77	4.63	3.22	2.09
Loma Prieta-1979-(1652 Anderson Dam)	2.89	5.41	4.02	2.64

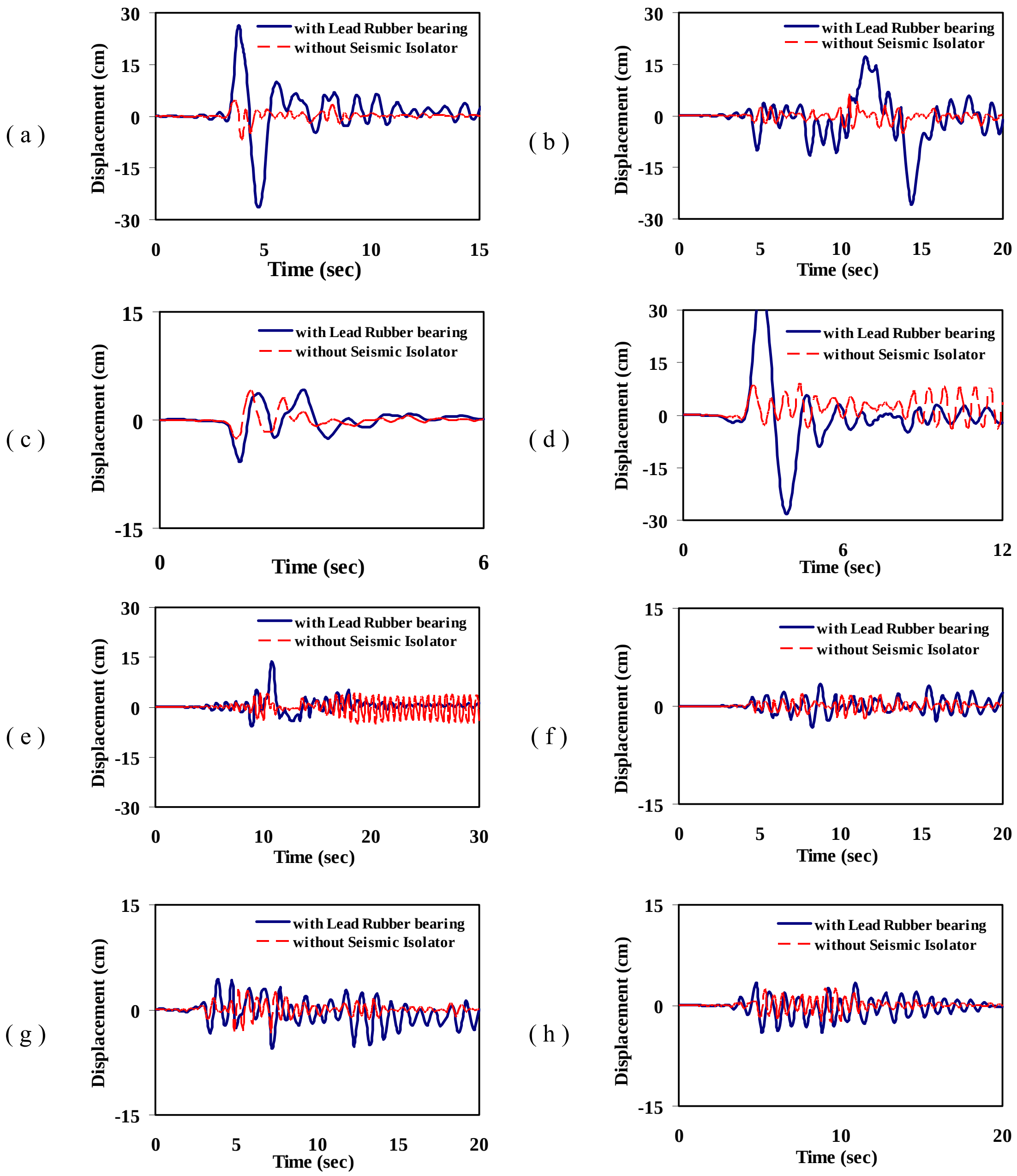

Figure 7. Time history graph of the displacement response on a bridge superstructure with and without seismic isolator due to earthquake; a) Northridge (Sylmar), b) Tabas (Tabas), c) Naghan (Naghan), d) Bam (Bam), e) Landers (Lucerne), f) Northridge (Moorpark), g) Tabas (Dayhook), and h) Loma (Anderson Dam)

Figure (7) shows time history graph of the displacement response on the bridge superstructure in centimeter (cm). These graphs show that capacity of superstructure displacement in a bridge with seismic isolation exceeds from the non-isolated bridge.

Table (3) shows maximum values obtained for base shear and displacement in models for different accelerogram.

As the values of this table shows, dynamic responses of base shear with a seismic isolator reduce for about 95 percent on average in all cases and displacement in near-field earthquakes in a bridge with the seismic isolator greater than its values are versus far-field records.

By using seismic isolator in models, displacements increase 3 times in the near-field records and 50 percent in far-field records with compared with non-isolated structure. Therefore, by increasing the displacement capacity of structures, their vulnerability against imposed forces is reduced.

5. CONCLUSION

The present research compared performance levels of a typical three-span bridge with and without seismic isolation, using nonlinear static analysis. It then compared nonlinear dynamic responses of the models versus strong ground motions in near- and far-field. It also discussed the results of nonlinear dynamic time history analyses, including base shears and superstructure displacement under such earthquakes on both models.

By studying isolated structure capacity curve, it is concluded that the isolated structure exhibits a more flexible behavior and by going further toward nonlinear area before entering nonlinear hinges to the immediate occupancy point, it damps further energy level as compared with a structure without seismic isolation. Whereas, structure without seismic isolation reaches immediate occupancy point with less displacement. It was also concluded that using of the seismic isolator helps to increase capacity to demand ratio in structure with demand reduction.

The values obtained for base shears in the models with the seismic isolator shows a considerable reduction compared with values of the registered base shears in the models without seismic isolator. The reduction in values of base shear was calculated about 90 percent on average.

The results also show that superstructure displacement in the structure with seismic isolator increases compared to the one without using the seismic isolator and in the minimum state showed about a 50 percent increase and it leads to increasing a structure capacity.

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