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Effects of the nonlinear behavior of lead-rubber bearings on the seismic response of bridges

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Abstract. The main objectives of this work were to investigate the effects of the nonlinear behavior of the isolation pads on the seismic response of bridges with rubber bearings, and to identify when base isolation improved their seismic performance. To achieve these objectives a parametric study was conducted designing a set of bridges for three different soil types and varying the number of spans, span lengths, and pier heights. The seismic responses (accelerations, displacements and pier seismic forces) were evaluated for three different structural models subjected to three earthquakes with different dynamic characteristics. The first represented bridges without base isolation; the second corresponded to the same bridges including now rubber bearings as an isolation system, with linear elastic behavior that shifted the natural period of the bridge by a factor of 2 to 4. In the third model the seismic response of bridges supported on lead-Rubber bearings was studied accounting for the nonlinear behavior of the lead. The results show clearly the importance of the nonlinear behavior on the seismic performance of the bridges.

Keywords: lead-rubber bearings; base isolation; bridges; nonlinear behavior.

1. Introduction

The idea of incorporating isolation and energy dissipation devices into structures is not new, and these devices have been implemented in many countries around the world, especially in structures that may be subjected to the action of accidental dynamic loads such as earthquakes. The implementation of this approach in underdeveloped countries is lagging because they may not have available all the technology needed for the design, nor the appropriate construction equipment. When the technology has to be imported the cost may increase substantially. As a result many people believe that structures with these devices are very expensive, yet Imbsen (2001) concluded that retrofit with isolation pads leads to cost reductions of the order of 40%.

There are at present many different types of systems that can be used to improve the performance of a structure; they are known under the generic name of structural control. The structural control can be passive, active, or semi-active (hybrid). In the case of existing bridges, base isolation is the easiest passive control system to incorporate since rubber bearings are already required at each support by the design codes; the rubber bearings can be easily replaced using flat jacks. For this reason, in this work base isolation was selected as the control system to be investigated. Base

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isolation bearings consist of alternated layers of rubber and steel, and can also include an inner lead core; usually their shape is circular or square. The first effect of base isolation is to elongate the natural period of the structure reducing hopefully the seismic forces by pushing it to a region of the response spectrum with smaller amplitudes. This may lead however to rather large displacements. To reduce them it is important to provide also an energy dissipation mechanism. This can be achieved through the nonlinear, hysteretic, behavior of the lead core. The nonlinear behavior of isolation pads is thus crucial in the analytical model in order to account for the energy dissipation effects. Based on the importance of this issue, several constitutive models have been proposed in the literature to represent the nonlinear behavior of the control systems according to their hysteretic behavior (Hosam-Eddin *et al.* 1995 and Wen 1976).

Analytical studies of bridges with hysteretic isolation systems have been conducted by different authors. A parametric study for bridges with lead-rubber bearings subjected to two seismic records was conducted by Turkington *et al.* (1989). The parameters of interest were lead-plug's size and aspect ratio, bearing pad's thickness and yield strength, pier height, and abutment's and superstructure's stiffness. The results indicated an improvement in the response when lead rubber bearings were combined with stiff piers, with the lead-rubber bearings providing hysteretic energy dissipation, but less effect as the pier stiffness decreased. These authors among others (Hwang *et al.* 1997) stated also that the elastic response of a SDF system characterized by an effective period and an effective damping reproduced with good agreement the nonlinear response of typical bridges supported on lead-rubber bearings. Jangid (2004) found that neglecting the bidirectional interaction of the restoring forces led to the underestimation of the peak bearings displacements; further, Abe *et al.* (2004) found in an experimental study that the restoring forces of the LRB depended highly on the vertical load, and that there were also coupled effects.

Several studies demonstrate that hysteretic energy dissipation has a beneficial effect on the seismic bridge responses. Shen *et al.* (2004) shows that the nonlinear behavior of the LRB can reduce the bridge acceleration even in the case of near fault type earthquakes. Other authors (Li 1989, Ghobarah and Ali 1989, Briseghella *et al.* 1989, Maragakis and Saiidi 1993, Mahin 1993, Monti *et al.* 1995, Li and Xin 1998) studied not only the methodologies recently used to model isolated bridges, but also the new design procedures offered in the design codes with the aim to investigate the accuracy on the bridges' seismic response and the effects on the structural demands caused by the hysteretic behavior of the isolation systems.

Despite the important number of research studies of isolated bridges, there is not a specific study quantifying the nonlinear behavior of LRB based on the analyses of isolated bridges without any energy dissipating devices and bridge models supported on a typical rubber bearings used currently in most of the medium length bridges.

In this work we started designing according to Mexican code provisions a set of 12 bridges (with different number of spans, span lengths, and pier heights) for three types of soils leading to a total of 36 structures. The seismic response of the bridges subjected to three ground motions were studied under three conditions. The first considered the bridges without base isolation (NBI); the second incorporated base isolation pads between the girders and the bent caps but assuming linear behavior (BI Linear); the third considered the base isolation systems with the nonlinear behavior of the lead core (BI). The comparison of the results from these three models allows to illustrate the importance of the nonlinearity on the response. The different soil types was accounted only on the design, through the different design spectra, but soil-structure interaction effects were not considered.

2. Structural model

The bridges considered were typical continuous reinforced concrete (RC) bridges in Mexico, with 2 and 5 spans, span lengths of 20, 40 and 60 m, and pier heights of 10 and 30 m located on seismic zone D (potentially more dangerous for structures located in Mexico). The combination of the geometric parameters led to 12 different bridges that were designed for three soil types: hard soil (type I), medium soil (type II), and soft soil (type III), leading to 36 total cases. The design response spectra for seismic zone D (CFE 1993) considering a ductility factor of 2 and an importance factor of 1.5, as proposed in the code, were used for the design. The seismic response parameters considered were the relative displacements and the absolute accelerations of the deck, the relative displacements and seismic forces in the piers, and the ductility demands for the isolation pads. The seismic responses were obtained for three ground motions: the Manzanillo, the SCT and the El Centro earthquake records. The Manzanillo 1995 earthquake had a 7.9 surface wave magnitude, M_s, it was generated on the Colima state of Mexico in the subduction zone of the Pacific Coast, and its epicenter was located about 30 km southeast of the port of Manzanillo. The SCT accelerogram was recorded in the basement of the Ministry of Communication and Transportation building (SCT) of Mexico City in 1985. It is characterized by its nearly harmonic nature, and by being recorded on soft soil with a predominant period of 2 seconds. It had a surface wave magnitude, M_s, of 8.1 with the epicenter located on the Pacific Coast of Mexico, at a distance of 350 km, in the Cocos Plate subduction zone. The El Centro earthquake 1940 was generated by a rupture of the Imperial Fault located in Califormia, USA, it had a 6.9 moment magnitude, M_w. This accelerogram was selected because of its extensive use in many publications, and its characteristics similar to those of the Manzanillo record.

In the text each bridge is identified with a number and a letter referring to the number of spans, the span lengths, and the pier height. For example, the bridge referred to as 5S60L30H represents a 5-span bridge with 60 m spans and 30 m pier height. All the bridges were considered to be RC except those with 60 m spans that had steel plate girders. All of the bridges had RC circular piers, RC slabs and RC bent caps. The diaphragms used in each of the bridges were RC or steel sections depending on the girders. The modulus of elasticity, shear modulus and Poisson ratio were 2.5E10 Pa, 1.03E10 Pa and 0.2 for the concrete, and 2E11 Pa, 7.7E10 Pa and 0.3 for the steel. Fig. 1 shows a schematic elevation of the 5-span bridges.

The analytical models commonly used in research and proposed in design codes to represent a bridge structure (Spyrakos and Vlassis 2002, Saiidii and Maragaskis 1999, AASHTO 1995, Ciampoli M. and Pinto 1995, Turkington *et al.* 1989, CALTRANS 1987 and NZMWD 1983) introduce a number of approximations (two degrees of freedom systems or plane frames instead of a full 3D model, equivalent linearization techniques to simulate the nonlinear response of the isolation pads). The use of plane frames and two-degree of freedom systems can be appropriate for preliminary studies or to explore general trends but will not reproduce accurately in general the behavior of an actual bridge. More refined models are needed. To model the structural elements of the superstructure and substructure such as girders, diaphragms, shear keys, piers and bent caps one may use frame elements, finite elements or fiber models, the selection depends on the geometry and behavior (linear or nonlinear) of each structural element; in a similar way, the slab could be modeled with a mesh of shell finite elements or with the definition of a rigid body diaphragm. Supports and abutments can be modeled as common supports (fixed or simple supports), or accounting for the flexibility of the soil through springs and dashpots defined based on the soil

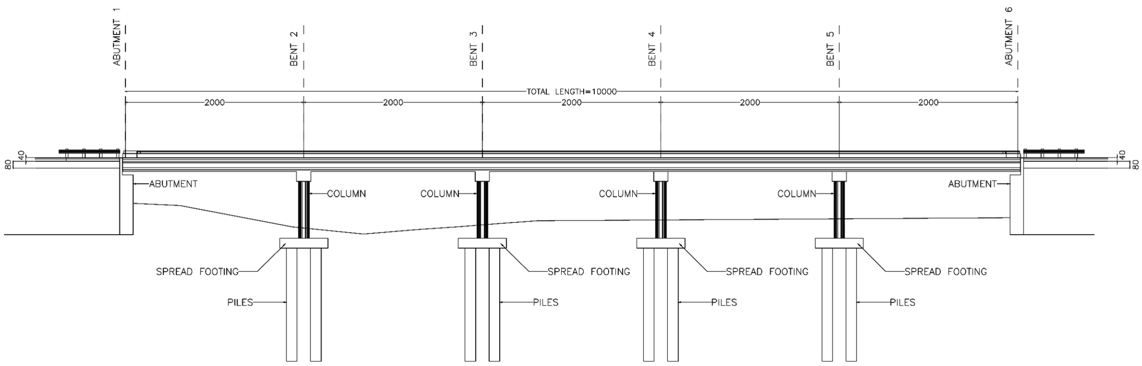

Fig. 1 Elevation view for the 5-span bridge model

properties. The more refined the model the more real will be the representation of the bridge's dynamic response, but to achieve the desired accuracy it is necessary to have a good understanding of the expected behavior of each of the elements. Otherwise the model will not be representative of the reality in spite of being very detailed.

Several options have been used to model the nonlinear behavior of LRB. One of them is the use of a simplified equivalent linearization procedure as proposed in the codes. This technique can be of value for a preliminary design but only for the types of seismic motions considered in their derivation (AASHTO 2000, CALTRANS 1987, Hwang 1996, Hwang and Sheng 1994 and NZMWD 1983). The accuracy of the equivalent linearization was investigated by Olmos (2008) but is not addressed here. In this study we consider a full nonlinear model for the isolators and we compare the results to those of a fully linear analysis with the material properties corresponding to the initial elastic branch (small strains) and without any hysteretic damping, since the objective is to investigate the effect of the nonlinear behavior. Commercial programs offer different nonlinear elements to follow the most common hysteretic rules.

In this work, the bridges were modeled as 3D structures and the analyses were carried out with the nonlinear SAP2000 program. In general the structural elements were represented with frame and shell finite elements with linear behavior, and LRB with nonlinear link elements of the rubber isolator type. To achieve the objectives of this work, three different bridge models were used. Their particular characteristics are described in the following paragraphs.

The first bridge model (NBI) corresponded to a continuous bridge without base isolation. The members (girders, diaphragms, bent caps, and piers) were modeled as beam elements, and the RC slab was modeled with a mesh of rectangular thin shell (plate bending and stretching) finite elements. The abutments were not included in the model, considering the beams supported at their ends by traditional laminated-rubber bearings with linear behavior. The bearings had an area of 0.09 m^2, and a shear modulus of 981 kPa; the right and the left bearings had heights of 57 and 41 mm, leading to stiffness values of 1962 and 2759 kN/m at each support, respectively. The base supports of the piers were considered fixed, neglecting the flexibility of their foundations. The beam elements used to model the girders were divided into small discrete segments with the mass distributed equally to each node, and the same amount of mass acting in the longitudinal and transverse directions. The behavior of all bridges' components was considered linear.

The 3D models used for the second case (BI Linear) were the same described in the first case (NBI), but incorporating the isolation pads without lead-core. The isolators had the same properties

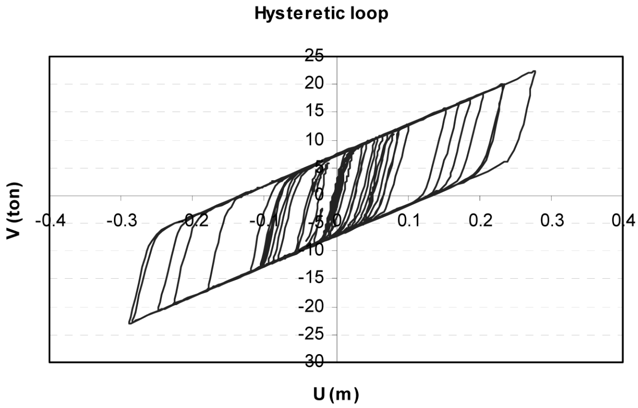

Fig. 2 Base isolator hysteretic behavior

Table 1 Hysteretic properties for the rubber isolator

Bridge name	Bridges located on soil type I			Bridges located on soil type II			Bridges located on soil type III		
	F_y (kN)	κ_E (kN/m)	Δ_y (mm)	F_y (kN)	κ_E (kN/m)	Δ_y (mm)	F_y (kN)	κ_E (kN/m)	Δ_y (mm)
2S20L10H	88.29	2943	30	88.29	4905	18	88.29	2943	30
2S40L10H	88.29	2453	36	88.29	2943	30	88.29	2453	36
2S60L10H	88.29	2943	30	88.29	5886	15	88.29	4905	18
2S20L30H	49.05	1962	25	49.05	1962	25	49.05	1962	25
2S40L30H	88.29	2453	36	49.05	1962	25	88.29	2453	36
2S60L30H	88.29	2943	30	49.05	1962	25	88.29	2453	36
5S20L10H	49.05	1962	25	88.29	2943	30	88.29	4905	18
5S40L10H	49.05	1962	25	88.29	2943	30	49.05	1962	25
5S60L10H	88.29	2943	30	88.29	5886	15	88.29	4905	18
5S20L30H	39.24	981	40	49.05	1962	25	88.29	2453	36
5S40L30H	49.05	2453	20	49.05	1962	25	49.05	2453	20
5S60L30H	88.29	2943	30	88.29	2943	30	49.05	1962	25

in the longitudinal and transverse directions. These were the properties of the initial branch of their load-deformation curve with purely elastic behavior. No attempt was made to simulate the nonlinear behavior using equivalent linearization. The bearings at each of the beam supports were modeled also with linear link elements of the rubber isolator type. Their properties were computed to produce a shift in the longitudinal natural period of the bridges by a factor of two to four without exceeding an allowable deformation (AASHTO 2000, Priestley *et al.* 1996 and CALTRANS 1987). The analysis for the base isolated bridge models assumed that all elements had linear behavior.

The third model corresponded to the base isolated bridges with LRB (BI). This model was analogous to the second but accounting for the nonlinear behavior of the lead core and the resulting energy dissipation. The analysis assumed that the base isolation bearings were the only elements behaving nonlinearly and that the remaining elements had linear behavior. The isolators' nonlinear behavior was modeled with NLink elements of the rubber isolator type that is a biaxial hysteretic isolator with coupled plasticity properties for the two shear deformations and linear effective-stiffness properties for the remaining four modes of deformation (vertical translation and rotations). The plasticity model is based on the hysteretic behavior proposed by Wen (1976) and Park *et al.* (1986). The base isolators were defined as bilinear inelastic with a post-yield stiffeness of 0.1 of the elastic stiffness (κ_E). Fig. 2 shows a typical isolator's force-displacement curve of one of the analyses conducted. Table 1 shows the yield stress (F_y), the elastic stiffness, and yield displacement (Δ_y) of the isolators implemented in each of the bridges; they were assumed with the same characteristics in both orthogonal directions, as stated earlier.

3. Seismic excitation

Three strong motion records were used to study the seismic responses of the bridges. Two of the records selected are from Mexico: the SCT 1985 Mexico City and the 1995 Manzanillo earthquakes, and another from the USA: the 1940 El Centro record. The first record was selected because it is the one that caused devastating damage in Mexico City, and the second is the most recent high intensity earthquake that also caused considerable material losses. These two records have different characteristics: the first ground motion is almost harmonic and was recorded in soft soil; the second has high frequency content and was recorded close to the epicenter on hard soil. The third earthquake was selected because it has somewhat similar characteristics to the Manzanillo

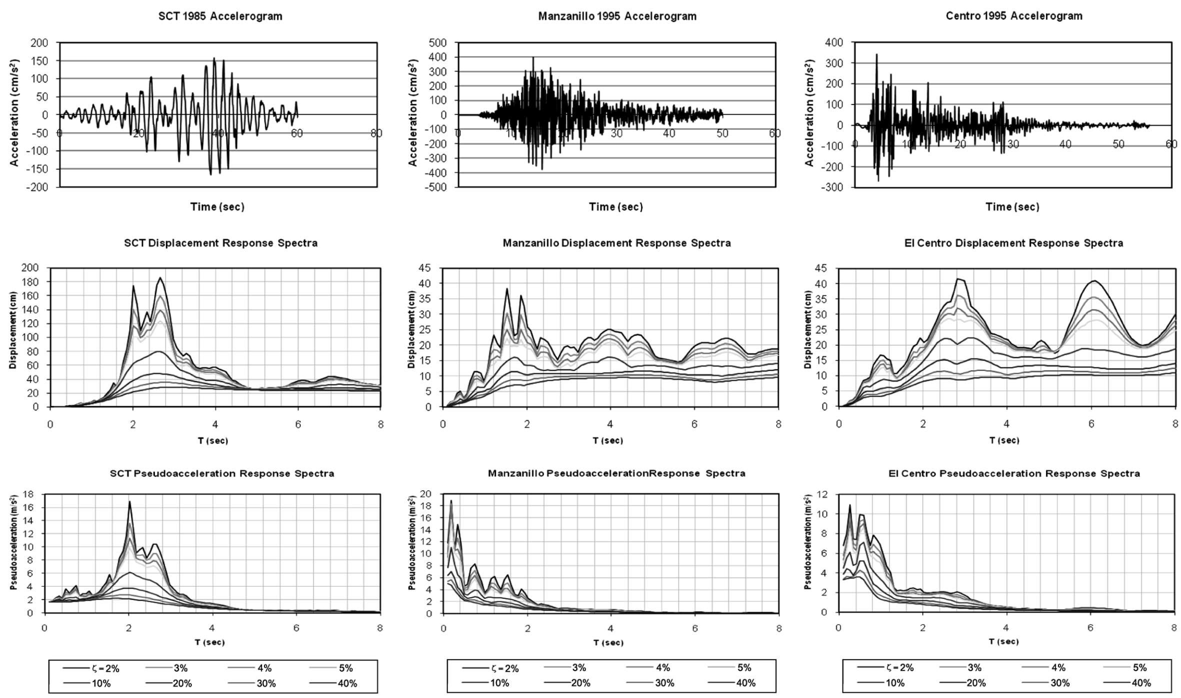

Fig. 3 Acceleration time histories, displacement and pseudo acceleration response spectra for the SCT, Manzanillo and El Centro earthquakes

record and has been used in the majority of the studies reported in the literature. Fig. 3 shows the acceleration time history of each earthquake and their linear relative displacement and pseudo acceleration response spectra for different values of damping to help visualize the effects of change in period and damping on the results.

From Fig. 3 it can be seen that for the Manzanillo and the El Centro ground motions if the natural period of the bridge without base isolation is between 0.5 and 1.0 seconds, an elongation of this period by a factor of 2 to 4 would result in considerable reductions in the pseudo accelerations but increases in the relative displacements. These increases could be reduced due to the hysteretic behavior of the isolators. For the SCT motion on the other hand both displacements and pseudo accelerations would increase for bridges with initial periods smaller than 1.5 seconds due to the period elongation.

4. Dynamic characteristics of the bridges

The dynamic characteristics of the bridges were obtained from modal analyses. The natural periods for the NBI bridges (T), and also the elastic and inelastic natural periods (T_E and T_Y) for the BI bridges in the longitudinal and transverse directions are listed in tables 2 and 3 for the 2-span and 5-span bridges, respectively. The natural periods for each bridge are presented in two rows where the first one corresponds to translation in the longitudinal direction and the second to the transverse direction. The T_E are calculated based on the bridge structural elements' stiffness and the elastic isolators' stiffness (table 1), whereas the T_Y are evaluated based on the post-yield stiffness of the isolators ($0.1\ K_E$) and the bridge structural elements′ stiffness. The inelastic periods are shown only as an indicator of the degree of stiffness change when the isolators get into the nonlinear branch; the actual stiffness and period would actually vary in time. In the majority of the cases the lower, 1st and 2nd, mode shapes of the NBI bridges corresponded to torsion and translation in the transverse direction whereas the isolated bridges had their first and second mode shapes in the longitudinal and transverse directions, respectively. The natural periods of the NBI bridges on soil types II and III were very similar, as expected, since the design response spectra for those soil types were alike. When BI was incorporated the differences in the periods of the bridges in soil types II and III had a tendency to increase. This did not happen in the transverse direction because in this direction the piers form frames with the bent caps making the bridge stiffer. As could be expected, the bridges with pier heights of 10 m are stiffer than those with 30 m; on the other hand, the number of spans does not influence the natural periods.

In spite of the variations imposed on the bridge parameters (span length, number of spans and pier height), and the soil types (I, II and III), the natural periods of the bridges formed two narrow bands grouped around the 10 m and 30 m pier height structures. The variation of the natural periods in the study was thus relatively limited. This suggests that the seismic codes used to design bridges located on hard, medium and soft soil, tend to produce structures with similar stiffness characteristics.

The beneficial or detrimental effects of the base isolation on the seismic responses depend on the natural period of the bridge, the hysteretic energy dissipated by the nonlinear behavior, and the type of excitation. If one of the bridges considered is subjected to ground motions such as El Centro or Manzanillo one would expect from the above discussion of the response spectra that there could be important reductions in the acceleration and shear forces due to the period shift caused by the base

Table 2 Natural periods of vibration for the modes in the longitudinal and transverse translations for the 2-span bridges

Bridge	Soil type I			Soil type II			Soil type III		
	T (sec)	T_E (sec)	T_Y (sec)	T (sec)	T_E (sec)	T_Y (sec)	T (sec)	T_E (sec)	T_Y (sec)
2S20L10H	0.78	2.40	5.73	0.47	1.54	4.32	0.48	1.89	5.54
	0.73	1.04	3.19	0.46	0.79	2.46	0.42	1.01	3.18
2S40L10H	1.13	2.81	7.80	0.65	2.31	7.03	0.65	2.51	7.69
	0.69	1.44	4.44	0.50	1.31	4.05	0.50	1.43	4.43
2S60L10H	0.71	3.09	9.09	0.54	2.10	6.32	0.54	2.28	6.92
	0.77	1.71	5.22	0.66	1.22	3.65	0.64	1.32	3.40
2S20L30H	1.86	5.80	8.48	1.32	4.63	7.77	1.19	3.18	7.10
	1.39	1.46	4.00	1.31	1.40	3.96	1.13	1.32	3.93
2S40L30H	1.69	5.57	9.03	1.64	4.42	9.18	1.59	3.75	8.12
	1.61	1.63	4.51	1.44	1.71	4.99	1.34	1.52	4.46
2S60L30H	1.79	5.62	10.05	1.95	7.20	12.67	1.69	5.09	11.30
	1.84	1.88	5.22	2.03	2.29	6.50	1.76	2.11	6.22

Table 3 Natural periods of vibration for the modes in the longitudinal and transverse translations for the 5-span bridges

Bridge	Soil type I			Soil type II			Soil type III		
	T (sec)	T_E (sec)	T_Y (sec)	T (sec)	T_E (sec)	T_Y (sec)	T (sec)	T_E (sec)	T_Y (sec)
5S20L10H	0.67	2.13	5.46	0.41	1.53	4.38	0.41	1.26	3.42
	0.67	1.45	4.36	0.41	1.15	3.55	0.41	0.91	2.76
5S40L10H	0.76	2.43	6.86	0.52	1.82	5.55	0.52	2.20	6.79
	0.56	1.87	5.57	0.63	1.52	4.55	0.63	1.83	5.56
5S60L10H	0.76	2.68	7.24	0.43	1.74	5.12	0.42	1.88	5.58
	1.05	2.04	5.88	0.38	1.45	4.24	0.86	1.57	4.62
5S20L30H	1.76	5.06	8.65	1.35	2.97	5.76	1.24	2.53	5.09
	1.81	2.42	6.38	1.31	1.63	4.44	1.19	1.45	3.97
5S40L30H	2.06	4.54	7.17	1.72	3.18	7.09	1.72	3.05	6.42
	1.94	2.27	5.15	1.37	2.06	5.62	1.37	1.92	5.04
5S60L30H	2.23	5.71	8.86	1.57	3.82	7.92	1.56	4.02	9.21
	2.43	2.88	6.29	1.68	2.42	6.21	1.65	2.70	7.32

isolation. On the other hand there would be increases or decreases on the relative displacements of the deck depending not only on the natural periods but also on the energy dissipation and the ground motion characteristics.

In the case of ground motions with the characteristics of the SCT, the relative displacements of the deck and the acceleration would have similar tendencies. The majority of the input energy of the earthquake lies within a band of periods suggesting that the seismic responses would be reduced if the structure shifted out of this range. The energy dissipated caused by the nonlinear behavior of the isolators should play a key role in this case.

5. Nonlinear effects of base isolation

The seismic responses of the bridges with and without base isolation are presented and discussed in this section. Since the bridges are symmetric about two orthogonal axes, the central and the extreme piers have very similar seismic demands, which allowed to draw general conclusions based on the response of only one pier. As a result only the response of the central pier for each bridge is discussed. The seismic demands of interest were the maximum relative displacements at the top of the piers, the maximum shear force on the piers, the maximum relative displacements and absolute accelerations at deck level, and the isolator's maximum ductility demands for the earthquakes acting independently in the horizontal or transverse direction. The time history analyses were conducted by direct integration of the dynamic equilibrium equations using the Newmark method for $\gamma = 0.5$ and $\beta = 0.25$, also known as the constant average acceleration method. A damping ratio of 5% was assumed for the structure using Rayleigh damping defined at the first two natural frequencies of the bridges (different for the NBI and BI cases).

The seismic responses for all the bridges were evaluated for the three earthquakes, but the results are presented only for the bridges designed for soil types I and II in the case of the Manzanillo because the results obtained for the designs on soil types I and II showed similar trends for the Manzanillo and El Centro earthquakes, and the designs for soil type III with the SCT ground motions. The selection of the soil type and the ground motions was based on the comparison of the pseudo acceleration response spectra and the code design spectra for each soil type as previously discussed.

Figs. 4 to 6 show the maximum seismic responses in the longitudinal direction of the bridges: the relative displacements (U_{max}) and the maximum shear force (V_{max}) for the piers, and the maximum relative displacements (U_{max}) and the absolute accelerations (A_{max}) at the deck level. Figs. 7 and 8 show in the same way the seismic responses in the transverse direction. Fig. 9 shows the maximum ductility demands of the isolators.

The seismic responses for the designs for soil types I and II in the longitudinal direction for the Manzanillo ground motion are presented in Figs. 4 and 5. For the bridges on soil type I (Fig. 4) we can see that in the case of linear base isolators, the relative displacements for the deck have increases for the 10 m pier height bridges, by factors of the order of 1.5 to 2 and there are two cases with factors of 3 and 5. The nonlinear effects of the isolators are more important for the cases with the larger increases. On the other hand, the 30 m pier height bridges show reductions on the relative displacements at the deck level for linear isolation systems whereas the nonlinear behavior of the isolation systems has a very small effect indicating that the reduction of the displacements of the deck may be due primarily to the period shift. The designs on soil type II (Fig. 5) with linear

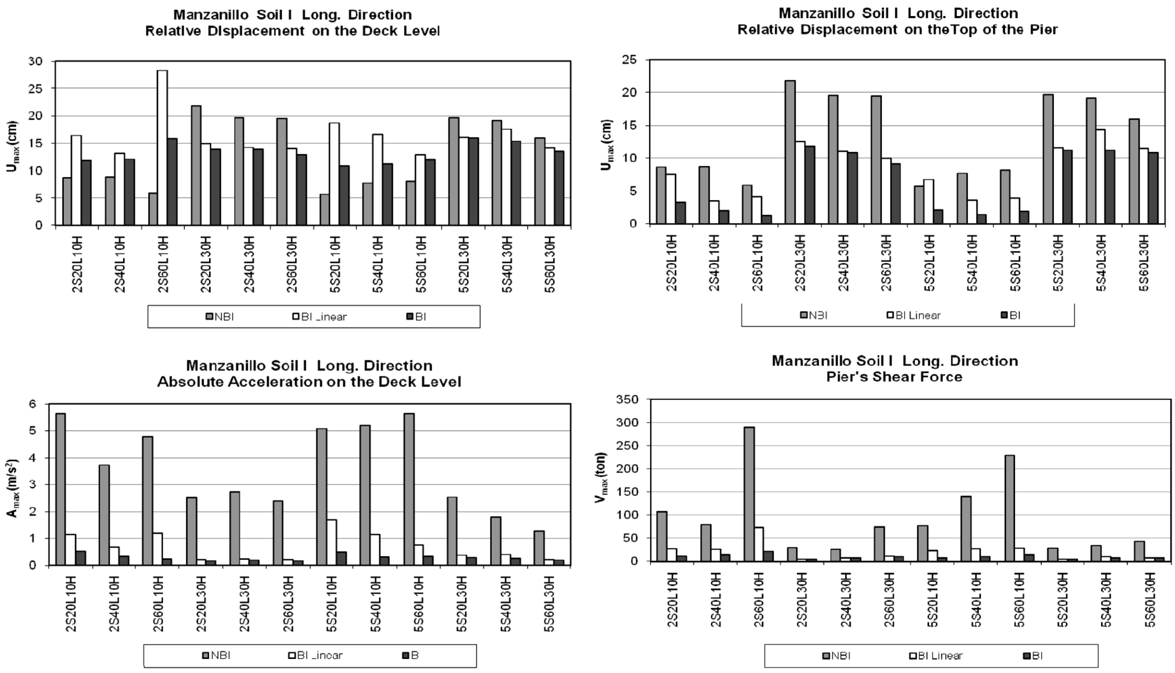

Fig. 4 Maximum seismic responses in the longitudinal direction for the central pier of the bridges on soil type I subjected to the Manzanillo ground motion

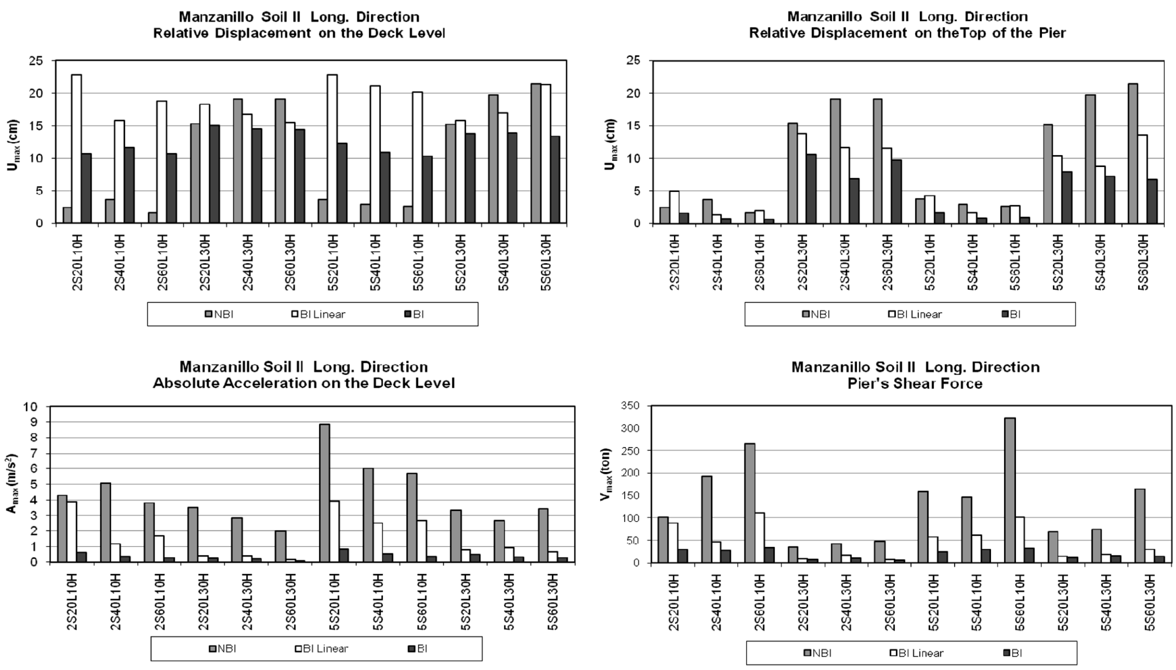

Fig. 5 Maximum seismic responses in the longitudinal direction for the central pier of the bridges on soil type II subjected to the Manzanillo ground motion

pads show larger increases on the relative displacements for the deck with smaller reductions due to the nonlinear isolation pads' behavior for piers 10 m height. The ones with 30 m height have small changes on the deck's relative displacements for the linear isolation cases, and there are smaller reductions when considering them with nonlinear behavior. The relative displacements at the top of the pier for both designs, soil types I and II, had in general reductions when the linear pads were incorporated on the bridges although there are two cases that have small increases. The decreases on the relative displacements of the piers are larger when the isolation systems are considerer nonlinear. The nonlinear effects are smaller for the 30 m pier height bridges than for the cases with 10 m pier height. All cases show considerable reductions on the absolute accelerations at the deck level when linear pads are included; the reductions are particularly important for designs on soil type I and for some cases on soil type II. When accounting for the nonlinear behavior of the isolation systems, the results show larger reductions for the accelerations. The seismic shear forces in the piers present similar tendencies to the ones described for the absolute accelerations of the deck. The behavior for the designs on soil types I and II under the El Centro earthquake was similar to the ones described.

Fig. 6 shows the seismic responses for bridges on soil type III in the longitudinal direction and the SCT record. In most cases the relative displacements of the deck have considerable increases when linear isolation is included; the increases can reach factors of 20. In the case of nonlinear isolators there still are increases for the relative displacements of the deck, but they are smaller and in a few cases there are small reductions. The responses for the BI cases show the importance that the additional energy loss has in reducing the relative displacements at the deck level through the isolators' hysteretic behavior. The relative displacements on top of the piers have increases in all

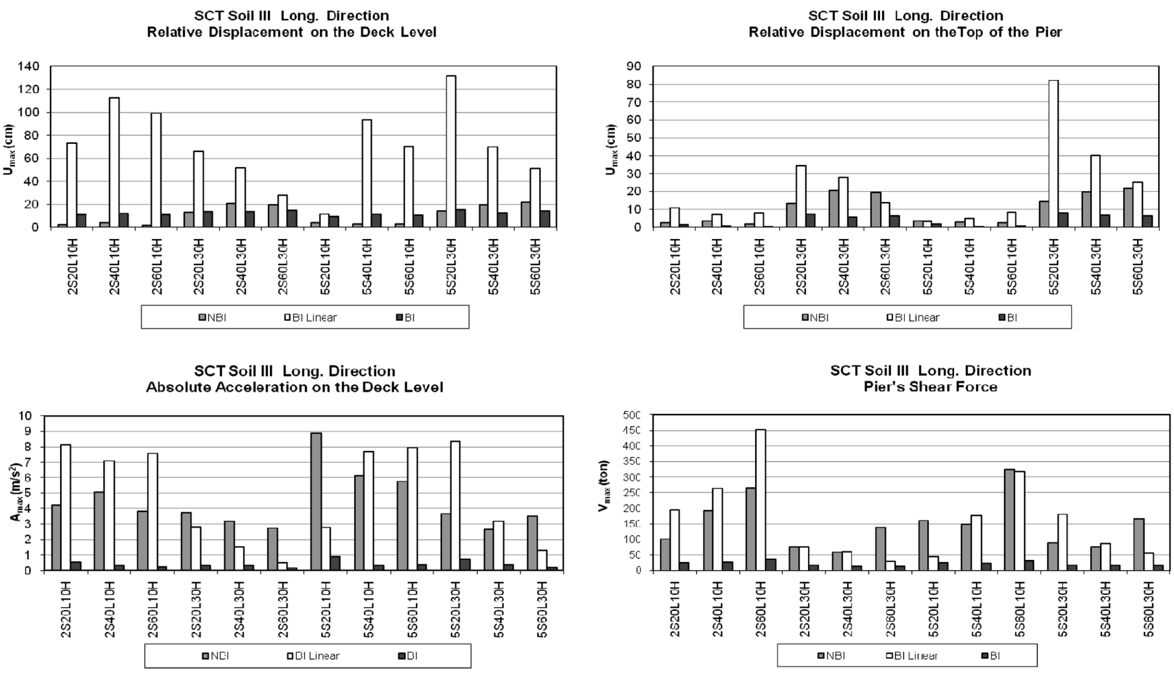

Fig. 6 Maximum seismic responses in the longitudinal direction for the central pier of the bridges on soil type III subjected to the SCT ground motion

cases with linear isolators whereas their nonlinear behavior causes decreases for all the structures. The same results apply for the seismic shear forces in the pier.

Fig. 7 shows the seismic responses for the bridges on soil type II under the Manzanillo motion in the transverse direction. The incorporation of linear isolators cause increases in the relative displacements for the deck in the majority of the cases, affecting greatly the 10 m pier height bridges, by factors of the order of 6 to 11. The nonlinear effects of the isolators are more important for the cases with the larger increases, and with larger beneficial effects for the 5-span bridges. The relative displacements at the top of the pier for both designs had in general reductions when the LRB were incorporated on the bridges with the exception of only one case that had small increases. The decreases on the relative displacements of the piers are larger when the isolation systems are considered nonlinear. The nonlinear effects are more important for the 30 m pier height bridges. The 2-span bridges with 30 m pier height on soil type I had small increases in their absolute accelerations at the deck whereas the remaining cases, soil types I and II, had reductions. On the other hand, all cases show considerable reductions of the absolute accelerations at the deck level when nonlinear LRB are included. The seismic shear forces at the piers present reductions for all cases with linear isolators; the reductions are slightly larger when accounting for the isolators' nonlinear behavior.

The transverse seismic responses of the bridges on soil type III under the SCT (Fig. 8) showed that when considering isolators with linear behavior the relative displacements at the deck level have considerable increases by factors that can reach values up to 7.5 for the bridges with 30 m pier height; for the other cases the increases are smaller. The nonlinear behavior of the isolators results in almost all cases in smaller increases of the displacements at the deck and a few cases even

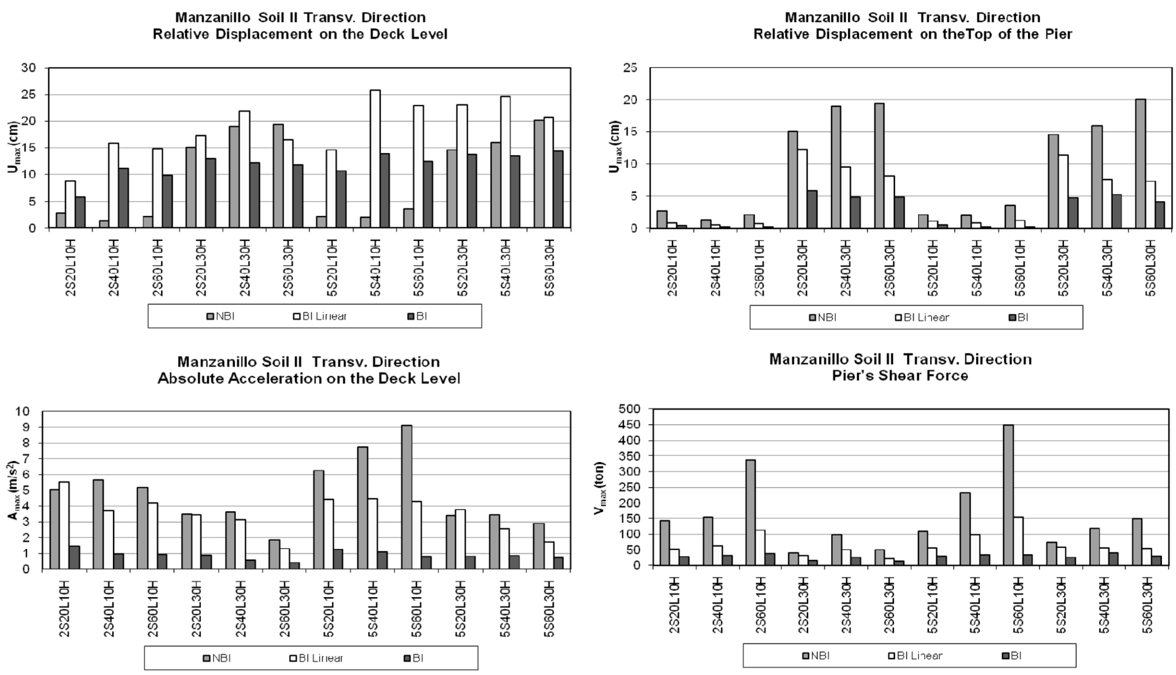

Fig. 7 Maximum seismic responses in the transverse direction for the central pier of the bridges on soil type II subjected to the Manzanillo ground motion

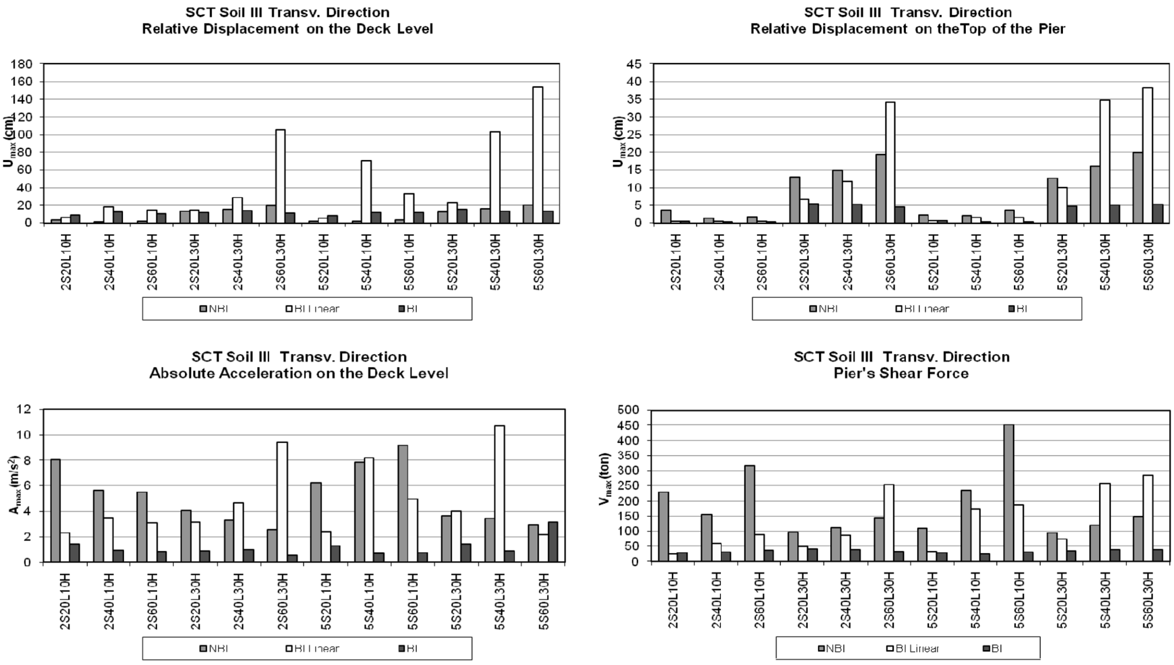

Fig. 8 Maximum seismic responses in the transverse direction for the central pier of the bridges on soil type III subjected to the SCT ground motion

decreases. The relative displacements on the top of the piers show generally reductions but increases for three of the 30 m pier height cases. With the nonlinear behavior there are always reductions, very substantial in some cases. Similar observations can be made for the deck accelerations and the shear forces in the piers.

Figs. 6 and 8 show larger beneficial effects of the nonlinear behavior and corresponding energy dissipation in the longitudinal than in the transverse direction. The relative displacements of the deck can still have, however, very large amplifications with respect to the case without isolation; one must question therefore the use of base isolation for bridges that may be exposed to this type of earthquakes although the shear forces in the piers will still decrease in many cases.

Fig. 9 shows the maximum ductility demands of the isolation systems for each earthquake and in both directions: longitudinal (first column) and transverse (second column). In the case of the El Centro and Manzanillo ground motions (Fig. 9, first and second row, respectively) the required ductility to be developed by the lead core increased with the natural period of the bridge up to a certain point reaching values of the order of 5 to 10. The results showed larger ductility demands for bridges on soil type II in both directions, with a few cases where bridges on soil type I under the El Centro quake had larger demands. In the case of these two earthquakes, the ductility demands were larger in the longitudinal than in the transverse direction, but in all cases the maximum ductility to be developed by the isolators had reasonable values. For bridges on soil type III under the SCT earthquake (third row of Fig. 9) the ductility demands were substantially larger, on the order of 10 to 25. The ductility demands were very large for several cases reinforcing the concern about implementing these devices in bridges that would be subjected to ground motions with similar characteristics to the SCT quake.

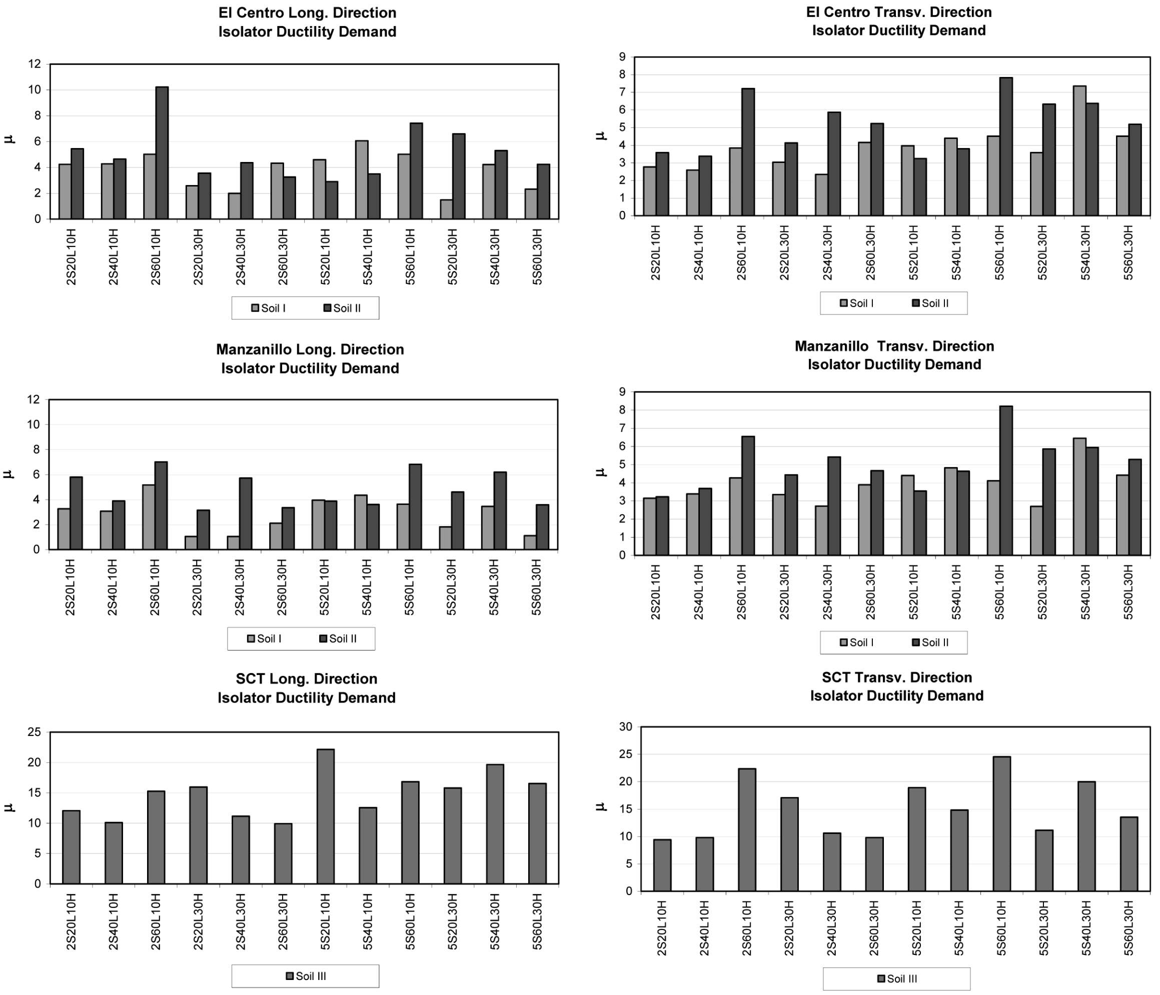

Fig. 9 Isolator ductility demands for the bridges subjected to the El Centro, Manzanillo and SCT ground motion

6. Conclusions

The first objective of this work was to assess the nature and importance of the nonlinear behavior of LRB on the seismic response of base isolated bridges. To achieve this, 36 typical medium length bridge models were designed and their response was obtained for three different seismic motions. The results of the study show in general increases in the relative displacements of the deck and important reductions on the deck absolute accelerations, the relative displacements and shear forces on the piers, as expected from the examination on the response spectra. The effect of the nonlinear behavior of the LRB system and the additional energy dissipation due to hysteretic behavior was always beneficial: small in some cases where the period shift was the primary factor and very important on others where the additional damping was crucial. With respect to the variables studied, the main differences in the isolation effects were associated with the pier heights.

The second objective was to assess under what conditions the use of LRB was beneficial. When accounting properly for the nonlinear behavior of the systems the results indicate that the base isolation would be very beneficial for bridges subjected to earthquakes with similar characteristics to the Manzanillo or El Centro records. There is on the other hand some question as to convenience of using base isolation for bridges to be subjected to ground motions similar to the SCT record. Although the isolation will result in reducing the shear force in the piers, the deck relative displacements and the ductility demands of the isolators may be very high. The use of isolation in this case would require very detailed analyses.

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