Strength and stiffness parameters of isolation systems for bridges with irregular pier heights

J.M. Jara, M. Jara & B.A. Olmos
Civil Engineering School, University of Michoacán (UMSNH),México

D. Villanueva
Civil Engineering School, University of Sonora, México

SUMMARY:
Most of the bridges built today have a strong irregularity in lateral stiffness of the substructure, mainly due to the height differences of the piers. This paper was motivated by the increasing use of isolation systems for bridges and the large number of structures that by their very nature must be supported by pier of different lengths. The bridges located in areas relatively close to the seismic sources are very attractive to incorporate seismic isolation systems in them, due to the high seismic activity and the frequency content of the earthquakes. This study presents an analysis of three types of pier height irregularities typical found in medium length bridges. A group of analytical models was created and isolation systems with bilinear behavior were incorporated. The results were evaluated based on the maximum response of the bridges and they were identified the best properties of seismic isolators which better improve the structural behavior of the systems.

Keywords: Bridges, Seismic isolation, Strength and stiffness parameters

1. INTRODUCTION

The base isolation systems are an attractive structural alternative for constructing and retrofitting bridges aimed at improving their expected seismic behavior. It can be promising the incorporation of isolation systems to reduce the acceleration or force demands, to generate force redistributions in piers and foundations and, at the same time, to restrain displacements by improving their energy dissipating capabilities. Bridges closely located to active seismic sources are very attractive to incorporate seismic isolation systems, mainly due to the high seismic activity and the typical frequency content of the seismic records in these areas. Many bridges have piers with different length which eventually induces lateral force demand concentrations on certain elements.

This study presents the analyses of a group of isolated bridge models with several strength and stiffness isolation properties to quantify the device importance on the seismic response of irregular medium length RC bridges. The bridges were subjected to a collection of scaled seismic signals recorded close to the subduction zone of the Pacific Coast in Mexico.

2. SEISMIC DEMAND

The bridge models are assumed to be located close to the Pacific Coast in Mexico. Ten accelerograms recorded in hard soil and close to the subduction seismic source were selected for conducting the non-linear dynamic analyses. Table 1 shows the description of the seismic records and figure 1 displays the elastic response spectrum and four inelastic response spectra of nine of the ten the accelerograms for a damping ratio of 5%. The maximum amplitudes of the spectra are in the range of 150 to 1000 gals and all of them show important frequency content for periods below 0.5 sec. There is only one response spectrum (station Zaca 21/09/85) with important amplifications in two periods range, for periods below 0.5 s and for periods in the range of 1.0 to 1.6 s.

Table 1. Sesimic records selected for conducting the non-linear dynamic analyses

Seismic station	Earthquake date	Magnitude	Epicenter location	Location of the seismic station
Viga	14-sep-1995	7.2	16.31 Lat. N 98.88 Long. W	16.757 Lat. N 99.236 Long. W
Papn	21-sep-1985	7.6	18.021 Lat. N 101.479 Long. W	17.328 Lat. N 101.040 Long. W
Zaca				18.009 Lat. N 102.178 Long. W
Azih				17.603 Lat. N 101.455 Long. W
Papn	19-sep-1985	8.1	18.021 Lat. N 102.942 Long. W	17.328 Lat. N 101.040 Long. W
Azih				17.603 Lat. N 101.455 Long. W
Zaca				18.009 Lat. N 102.178 Long. W
Sicc	25-oct-1981	7.3	17.880 Lat. N 102.150 Long. W	17.933 Lat. N 102.200 Long. W
Apat				19.083 Lat. N 102.350 Long. W
Sicc	14-mar-1979	7.0	17.490 Lat. N 101.260 Long. W	17.933 Lat. N 102.200 Long. W

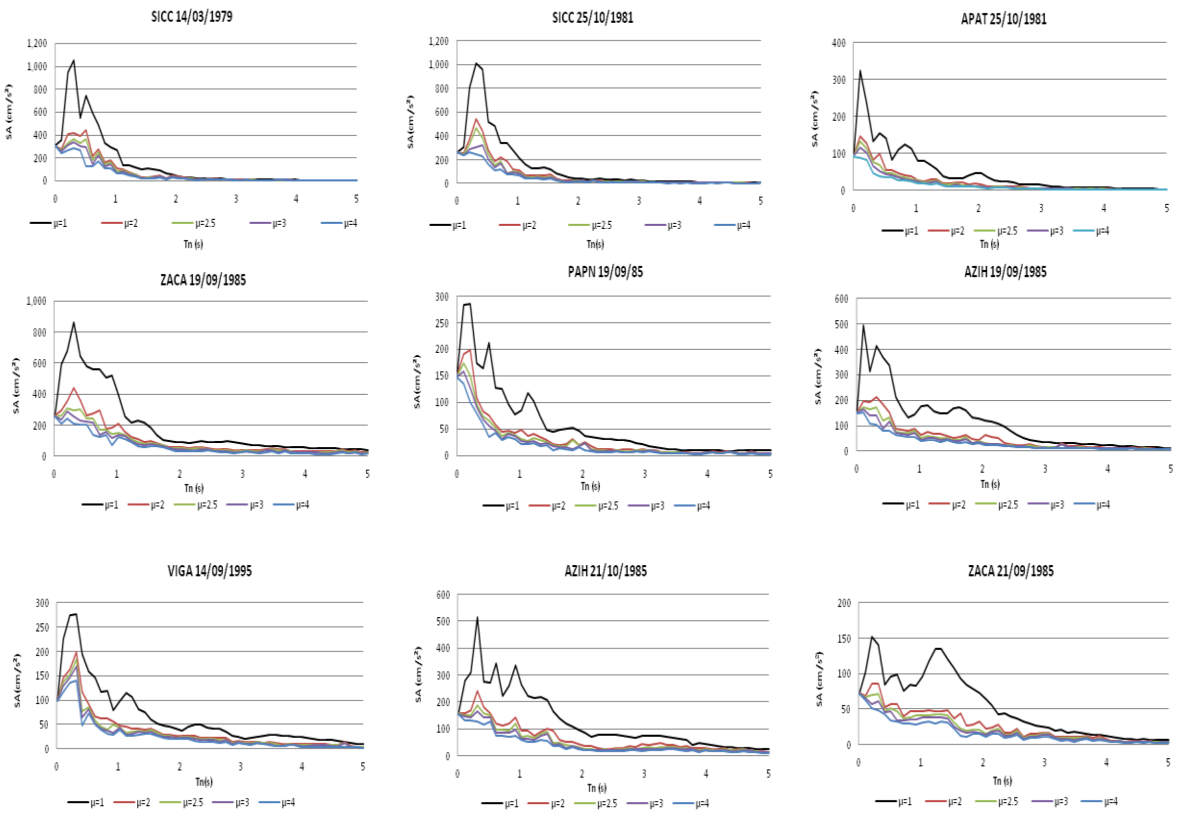


Figure 1. Elastic and inelastic response spectra of the selected seismic records

The seismic records were scaled to four earthquake return periods according to the study of Jara and Jara (2007). The scaling factor was determined as the ratio of the spectral ordinate of a uniform hazard spectrum corresponding to the bridge fundamental period to the response spectrum of each seismic record for the same period. Figure 7 shows the five uniform hazard response spectrum used in thus study for scaling the seismic signals. The return periods are in the range of 30 years to 2500 years for considering the frequent and rare type earthquakes.

3. NUMERICAL MODELS

A family of irregular five and six-span simple supported bridges were analyzed to quantify the contribution of the isolation devices to reduce the strength demand concentration on piers. Several common configurations were subjected to the group of the seismic records previously described. The bridges are assumed to be fixed at the abutments. The superstructure in all the models analyzed has the same geometrical and mechanical characteristics.

The models are composed by five 30 meters long spans and the superstructure transverse section is composed by slab deck of 0.18 m thick and eight AASHTO Type IV prestressed beams, placed at every 1.3 m, with parapets at both ends. RC diaphragms of 0.38 x 0.77 m were also considered in each third and each end of the spans. The prestressed beams were modelled with a concrete compressive strength of f'c = 350 kg/cm2 and the rest of the structural elements with f'c = 250 kg/cm2. The beams are simple supported on 41 mm and 57 mm thick laminated elastomeric rubber bearings.

The pier bent consist of four circular columns with diameter in the range of 1.1 m and 1.3 m, as function of the pier height. In general, this type of bridges produce a pier cantilever behavior in longitudinal direction and a frame type behavior in transverse direction, characterized by a lateral deformed shape with an inflexion points in columns.

Table 2 shows the characteristics of the bridge models. The models are identified with an increasing number and each column presents the pier’s height. The models 1 to 9 have four piers and the models 10 to 13 are five pier bridges. The last column of the table presents the ratio of the highest to the lowest length pier in the bridge model. Model M-1 is the regular bridge model with equal length piers.

Table 2. Numerical bridge models

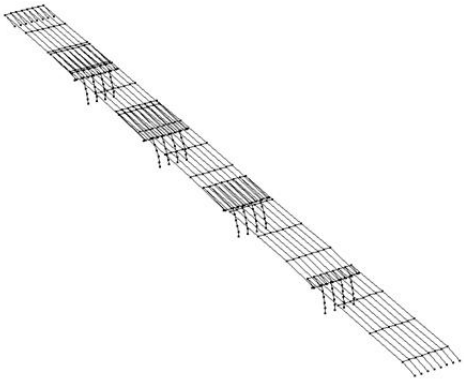
Model	Pier height					
	Pier 1 (m)	Pier 2 (m)	Pier 3 (m)	Pier 4 (m)	Pier 5 (m)	Hmax / Hmin
M-1	5.0	5.0	5.0	5.0		1.0
M-2	5.0	7.5	7.5	5.0		1.5
M-3	5.0	10.0	10.0	5.0		2.0
M-4	5.0	15.0	15.0	5.0		3.0
M-5	5-0	5.0	7.5	5.0		1.5
M-6	5.0	5.0	10.0	5.0		2.0
M-7	5.0	5.0	15.0	5.0		3.0
M-8	7.5	7.5	10.0	7.5		1.3
M-9	7.5	7.5	15.0	7.5		2.0
M-10	5.0	7.5	10.0	7.5	5.0	2.0
M-11	5.0	7.5	15.0	7.5	5.0	3.0
M-12	7.5	10.0	15.0	10.0	7.5	2.0
M-13	5.0	7.5	10.0	7.5	7.5	2.0

The models were designed with the response spectrum of firm soil (table 3) for a seismic zone close to the Pacific Coast in Mexico (CFE-2008). It was considered a reduction factor of Q=2 and an importance factor of 1.5. and applying the load combinations specified in AASHTO (2005). The live loads used are three types of trucks circulating in Mexico highways, namely: HS-20 (W=32.5 t), T3-S3 (W=48.5 t) and T3-S2-R4 (W=66.5 t).

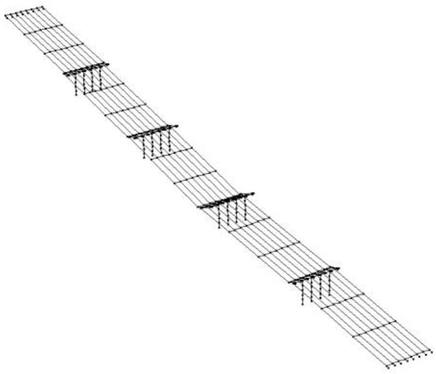
The longitudinal reinforcement ratios of the pier columns were in the range of 1.7% to 2.9%. It was decided to keep the same size and steel reinforcement in all columns of the bridge models. Figure 2 shows two initial shape modes of the regular model, the first one in longitudinal direction and the second one in transverse direction. It must be pointed out that there is only a small range variation in the period values among the analyzed models.

Table 3. Response and uniform hazard spectra of a zone close to the Pacific Coast in Mexico

	Response spectrum	Tr=30 years	Tr=500 years	Tr=1000 years	Tr=2500 years
T (s)	Sa (g)	Sa (g)	Sa (g)	Sa (g)	Sa (g)
0	0.40	0.13	0.40	0.48	0.65
0.1	1.00	0.27	0.83	1.04	1.38
0.6	1.00	0.23	0.67	0.86	1.10
0.8	0.86	0.19	0.57	0.73	0.94
1.0	0.77	0.16	0.47	0.60	0.77
1.4	0.65	0.14	0.40	0.52	0.67
1.8	0.58	0.11	0.33	0.43	0.56
2.0	0.55	0.10	0.30	0.38	0.50
2.5	0.48	0.09	0.26	0.33	0.44



T=0.84 s (longitudinal)



T=0.80 s (transverse)

Figure 2. First two shape modes of the model M-1

Four stiffness values of the isolation system were selected aimed at increasing the fundamental period of the models in the range of two to four times. Because of the narrow interval of bridge period variation, the same isolation stiffness was selected for all models. In all cases the post-elastic stiffness was assumed of 10% of the elastic one.

Initially, the bridge yield forces were obtained by a pushover analysis in longitudinal and transverse directions. The isolation device strength was also another important parameter studied as well. Three values of the isolation yield force were analyzed, namely: minimum, medium and maximum. The minimum value was selected to keep the isolation system in the elastic range of behavior for the maximum braking forces of the three load trucks used. The braking forces were assessed using the expression proposed in AASHTO (2005) for each span of the bridge. This value was in the range of 8% to 10% of the pier yield shear force in all models. The medium device yield strength was the 50% of the bridge yield force and the maximum device yield force analyzed was the shear yield of the bridge in both directions.

3. NON-LINEAR DYNAMIC ANALYSES

The analyses of the bridge structures were conducted by creating 3D analytical models with the Perform 3D software (CSI, 2006). The moment-axial load interaction diagrams and moment curvature curves obtained from SAP 2000 were used to define the non-linear properties of the columns in the NLTHA. The girders, diaphragms and cap beams were modeled with beam type elements. In order to consider the distributed mass of the columns, these elements were divided in one meter long sub-elements.

The Perform 3D models were subjected to the suit of seismic records described above. The identification of trends in behavior was conducted by drawing a set of graphs with the seismic response of typical engineering demands as, drift, bending moments, shear forces and ductility against normalized parameters as the stiffness ratio of the seismic isolators to the bridge. In the next paragraphs some relevant results are commented.

The results of the first four bridge models (M-1 to M-4) are jointly presented in the figure 3. It shows a group of graphs of the maximum drift demands (θmax) on the short piers in the transverse direction of the bridge models. The graphs in the left column display the models with the minimum value of the isolator yield strength and in the right column the results for the medium strength value are presented. In both cases, the horizontal axis displays the isolator to bridge stiffness ratio.

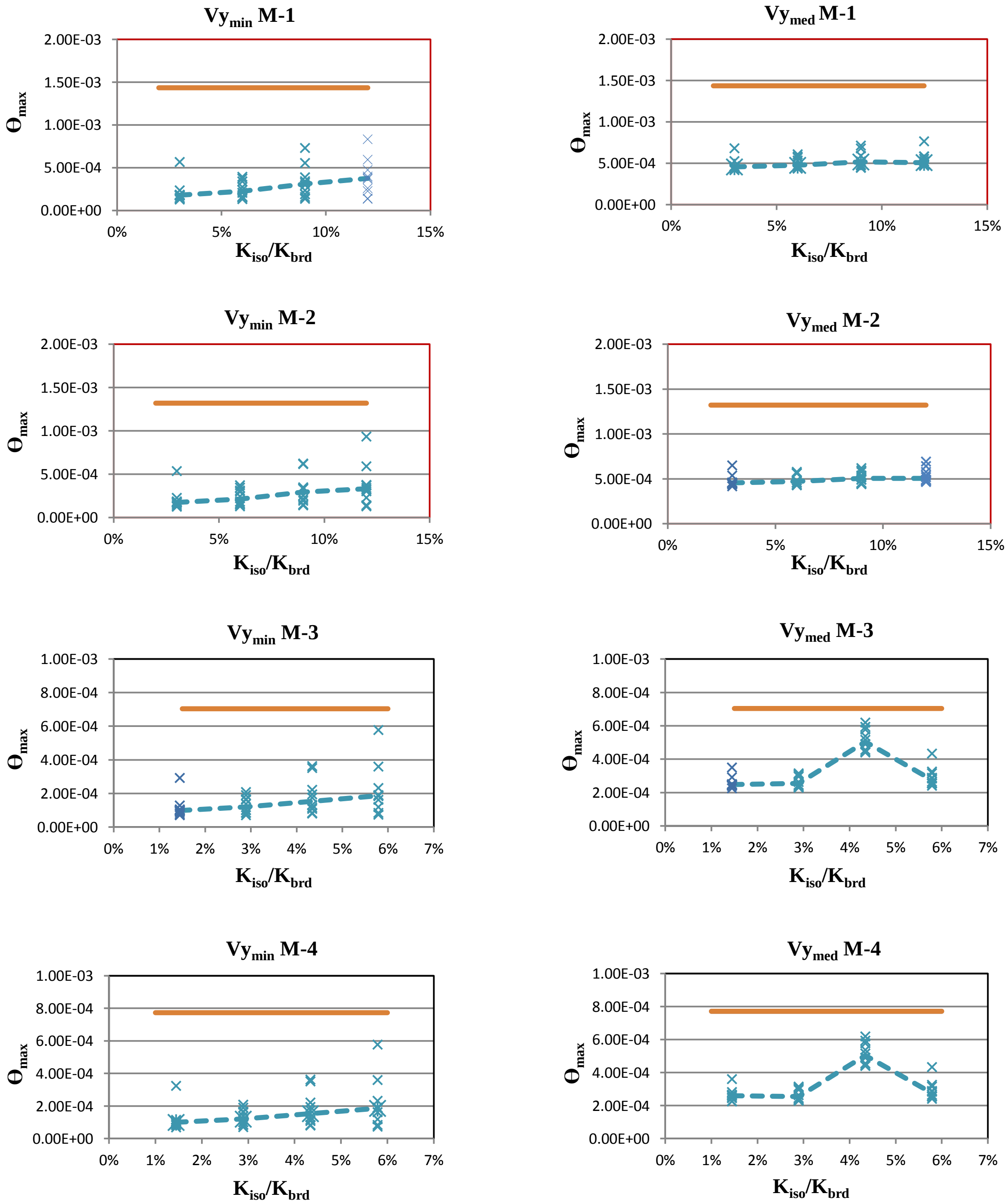


Figure 3. Drift demands in transverse direction in short piers (M-1 to M-4 models)

Figure 3 also shows with a dashed line the median of the data and with a continuous horizontal line the bridge responses of the models supported on neoprene bearings.

The seismic response of the models M-1 and M-2 are quite similar showing that the increase of the central piers height from 5.0 to 7.5 meters is almost negligible on the drift demands. However, the height increase from 5.0 (M-1) to 15.0 m (M-4) appreciably reduces the drift demands in short piers. Increasing the bridge irregularity makes also important the isolator strength on the pier responses. Hence, the differences between pier drift demands with the minimum and medium isolator strength in M-1 and M-2 are smaller than the differences presented between M-3 and M-4.

Bridges supported on piers with different lengths usually presents irregular damage after an earthquake occurrence. Seismic demands concentrations in some piers of traditional supported bridges can be one of the reasons of the damages.

As mentioned before, one of the objectives of the study was to assess the influence of the isolation system on the demand concentration on short length piers. Figure 4 displays the ratio of the maximum to the minimum drift value presented in transverse direction of the bridge piers. Again, the horizontal continuous line in each graph is the drift value of the bridge over neoprene bearings.

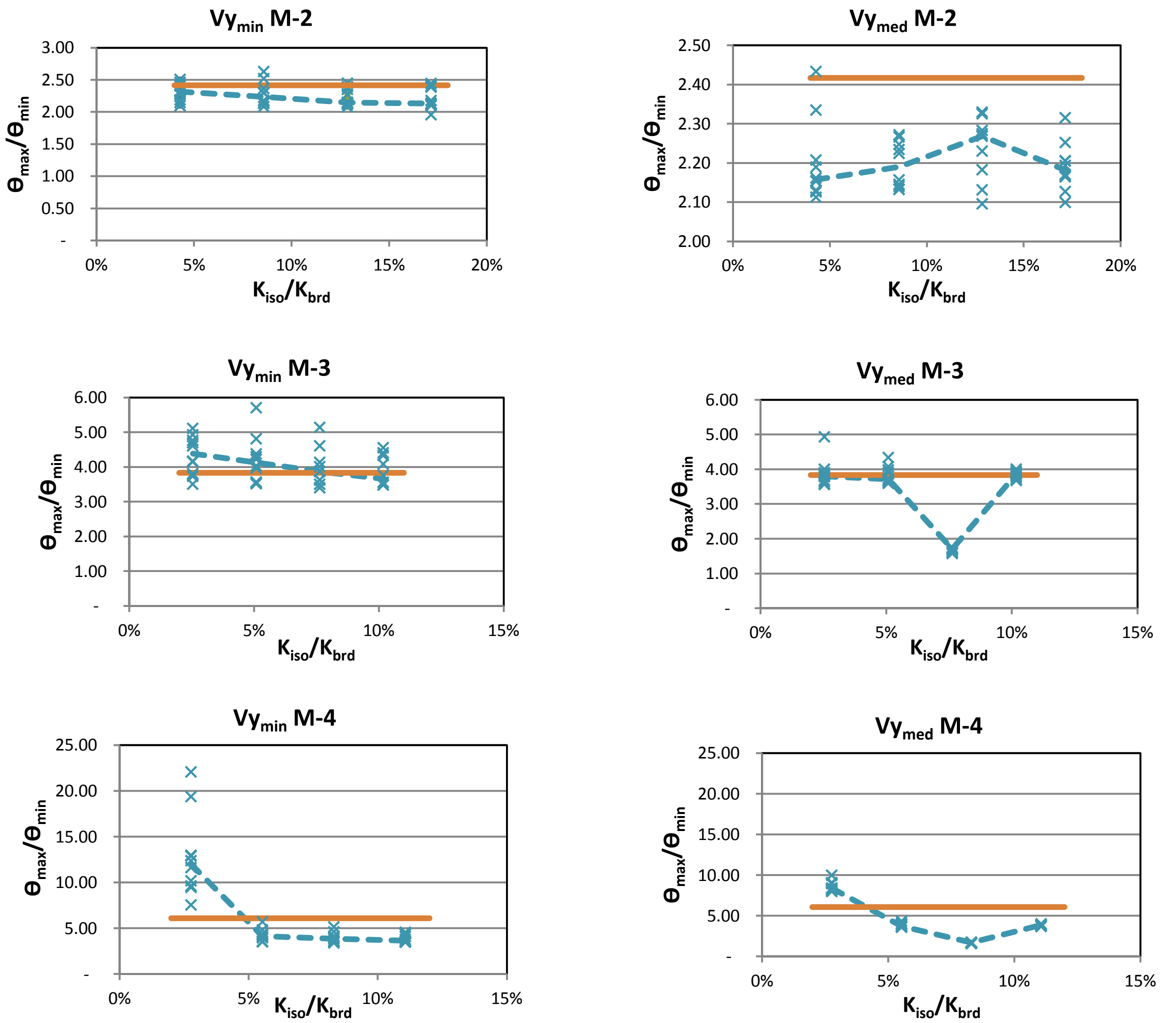


Figure 4. Ratios of the maximum to the minimum drift demands on piers in the transverse direction of the bridge models M-1 to M-4

It is interesting to note that in some of the isolated cases the drift ratio grows instead of being reduced.

This specially presented for low stiffness ratios (below 5%). It is also notable the small influence of the isolation on the drift ratio when the minimum isolator yield strength is employed. It is clear from figure 4 that the medium isolator yield strength should be used if we look for more regular behavior among the piers, and the isolation system shows more contribution in the seismic response when the increase of the central pier length (M-3 and M-4). According to these graphs the drift ratio between two piers can be reduced up to three times the drift ratio on traditional bearings.

In general, the cantilever behavior in longitudinal direction and the frame type behavior in transverse direction of the piers, produce different parameters of the isolation system. In order to have more uniform seismic response in the substructure, the best ratio between the isolator stiffness and the bridge stiffness in transverse direction was in the range of 6% to 12%. This percentage must be increased to the range of 18% to 28% in the longitudinal direction of the bridge. A complete discussion of the results of all models is presented in Jara, et al. (2012a). The applicability of the analyses was also evaluated by the study of a real isolated bridge described in the following section.

3. SEISMIC RESPONSE OF ISOLATED BRIDGE

In order of evaluating the applicability of the results of the analytical models studied, it was analyzed the response of a real isolated bridge located close to the Pacific Coast in Mexico. The bridge is a five simple supported 105 meters long span with a total length of 525 m. Each span of the bridge superstructure is divided in 17 panels with a length of 6 m each, approximately. The 12 m width deck has been made of light-gage steel deck cover with a concrete slab depth of 18 cm. The slab is supported on girders, spaced at 1.5 m, which off-loads to floor beams with triangular cross-sections and spaced at 6 m. The girders are connected to two Camel Back type steel trusses braced at the top, with a maximum height of 6.5 m. Shear transverse keys 70 cm depth are located at the ends of the cap beam. Fig. 5 shows a front and a longitudinal view of the bridge.

Figure 5. Front (left) and longitudinal (right) view of the isolated Infiernillo II Bridge

Non-prismatic abutments and four wall type reinforced concrete piers support the superstructure. Piers are hollow box shape sections with plan dimensions of 8.5 x 3.5 m, 15 m height, thickness of 40 and 60 cm in longitudinal and transverse direction respectively, and a hammerhead cap on the top (Fig. 6). Piers are supported on two circular reinforced concrete hollow cylinders with diameter of 8.5 (small piers) and 10 m (long piers). In both ends of the cylinders a solid reinforced concrete cap was built to make possible a fixed connection between them and the foundation piles in the bottom, and with the bridge piers on the top. The cylinders height is in the range of 21 m to 46 m. for a total piers' height in the range of 46 m (pier 5) to 70 m (pier 4).

The bridge was projected with isolator bearings of the sliding multi-rotational type. There are four devices over each pier and two devices on each abutment of the bridge. The model of the isolation system is a bi-linear hysteretic model with the elastic stiffness, yield strength, yield displacement and post-yielding strength presented in Jara, et al. (2011) and Jara, et al. (2012b). It should be noted the high elastic stiffness exhibited by the device and the small post-yielding stiffness value (about 1% of the elastic stiffness). Other conducted studies related to the seismic behaviour of this bridge can be found in Jara, et al. (2011), Jara, et al. (2012b) and Varum, et al. (2009).

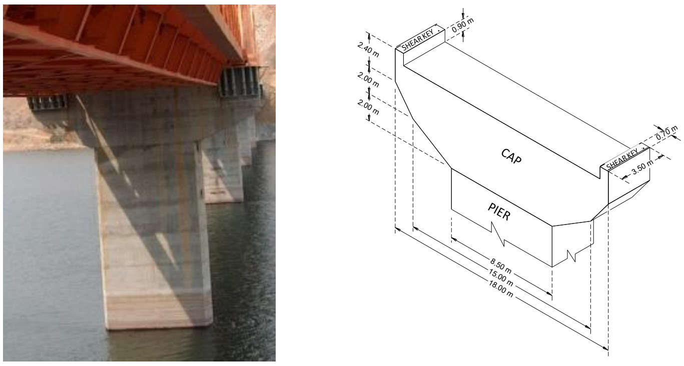


Figure 6. Reinforced Concrete Piers and Hammerhead cap of the Infiernillo II Bridge

After determining the dynamic characteristics of the bridge, it was decided to assess its seismic response with the properties of the original isolation system and with other two device models. These additional models varied the strength device parameters obtained with the results of the previous sections. The new isolation properties are shown in table 4.

Table 4. Stiffness and strength parameters of the new isolation system for the Infiernillo II bridge

Isolation type	Isolation location	$K_{elastic}$ (kg/mm)	$K_{post\text{-}elastic}$ (kg/mm)	$K_{iso/brd}$	V_y (Kg)	Δ_y (mm)
Equal properties	All piers	812.2	81.2	38%	31386	38.6
Different properties	Long Piers	812.2	81.2	38%	31386	38.6
	Short piers	812.2	81.2	38%	40000	49.3

The maximum seismic response is slightly improved with the new isolation properties of table 4 when the bridge is subjected to the 1000 years earthquake (fig. 7). This figure displays the drift and the bending moment demands in transverse direction of the bridge. The seismic analyzes were conducted for only four of the seismic records previously presented.

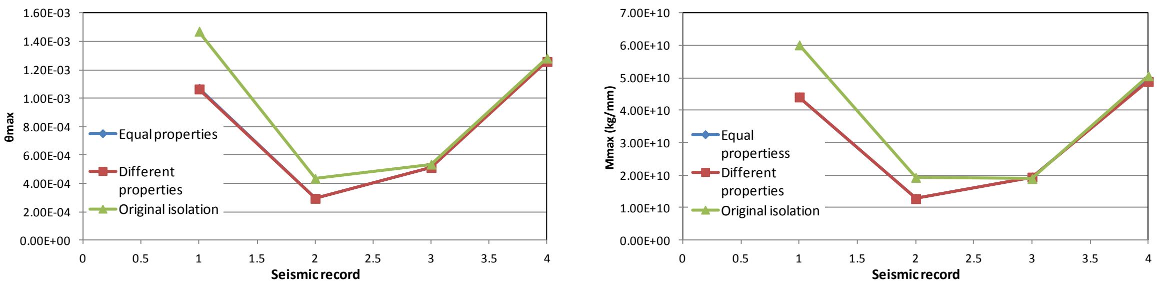
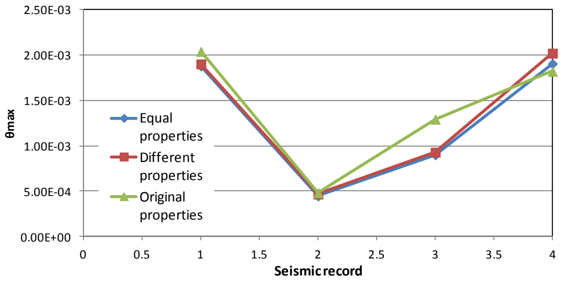


Figure 7. Drift and bending moment demands in piers for the seismic action in transverse direction

The proposed isolation system with equal properties produced similar results of the original isolation system in the bridge. However, the change of the isolation system properties in short and long length piers, reduced slightly the seismic demands.

There is not the same behavior in longitudinal direction, where the three models behave quite similar (fig 8). In this case, it was not presented any advantage of changing the isolation properties between the short and long length piers.

In order to better understand the seismic behavior presented, it was analyzed the pier properties of the bridge. In spite of the great difference among the piers length, the long piers have larger diameter than the short piers, appreciably reducing the lateral stiffness change among the columns. This must be the main reason of the similar results obtained with the isolation models analyzed.



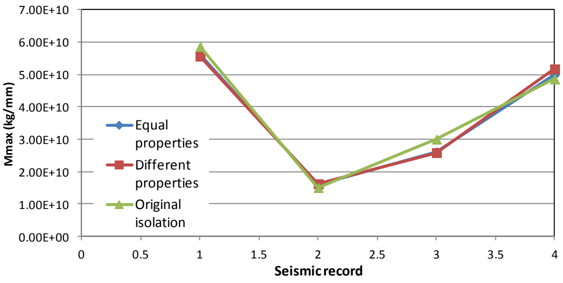


Figure 8. Drift and bending moment demands in piers for the seismic action in longitudinal direction

4. CONCLUSIONS

This paper presents the analysis of typical RC medium length isolated bridges with three stiffness irregularity distribution. The three types of bridge models selected cover a wide range of real RC medium length bridges. The paper aimed at determining the best strength and stiffness parameters of an isolation system to reduce the effect of the pier height irregularity. It was also analyzed an irregular real isolated bridge located close to the Pacific Coast in Mexico.

According to the results derived from the study, the more regular behavior was obtained with the use of isolators' stiffness in the range of 6% to 8% of the bridge stiffness in transverse direction and in the range of 16% to 23% in longitudinal direction of the bridge. The isolator yield strength in the range of 10% to 50% of the yield strength of the bridge seems to be appropriate for reducing the shear force concentrations in some piers. The isolator yield strength should be selected by considering the ductility demand and capacity on these elements. With these parameters of the isolation system, the rotation, moment and shear forces concentration demands traditionally presented on short piers of bridges supported on traditional laminated-neoprene bearings, can be dramatically reduced.

The suggested parameters for the isolation system can be used for a preliminary analysis of a new irregular isolated bridge or for an initial analysis of existent irregular bridge that must be retrofitted using an isolation system as an alternative for improving its future seismic behavior.

The real isolated bridge analyzed showed that the expected behavior can be slightly improved if the isolation system properties of the short and long length piers are changed. It was also determined that even though the great difference among the pier heights, the lateral stiffness of these elements are not quite dissimilar because of the different cross section diameter of the short and long piers.

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