ANALYSIS OF SEISMIC RECORDS OBTAINED IN ISOLATED STRUCTURES

Ofelia MORONI[1], Mauricio SARRAZIN[2], Rubén BOROSCHEK[3], Rubén VALDEBENITO[4] And David ROMO[5]

SUMMARY

The central zone of Chile is located in a high seismic risk area with the occurrence of magnitude 7 earthquakes every 10 years and magnitude 8 events every ninety years. Three seismically isolated structures located in this zone are instrumented with strong motion networks. These structures correspond to a 4 story confined masonry building in Santiago, a 383 m long bridge in Viña del Mar, and an elevated section of the Santiago Metro-Train.

Spectral analysis techniques are carried out for various recorded structural accelerations. Power spectral density functions, transfer functions, cross-spectra, coherence functions and phase angles are computed. Natural frequencies of the systems are identified based on the spectral analysis. Computer structural models are developed to correlate observed and model frequencies.

It can be seen from the data that, although the intensities of the motions were small, the isolation systems were effective in reducing the horizontal peak accelerations in the structures. For larger motions, the effectiveness of the isolation should increase given the nonlinearity of the force-displacement relationship of the isolation systems. However, for these small events, amplification has been observed in the vertical direction. It remains to be seen how if this behavior will occur for larger earthquakes.

INTRODUCTION

The central zone of Chile is located in a high seismic risk area with the occurrence of magnitude 7 earthquakes every 10 years and magnitude 8 events every ninety years. Three seismically isolated structures located in this zone are instrumented with strong motion networks. The earliest instruments were installed in 1992 in a 4 story confined masonry building in Santiago. The other structures correspond to a 383 m long bridge in Viña del Mar and an elevated section of the Santiago Metro-Train.

The isolated building and a conventional twin, standing nearby, are instrumented with four digital accelerometers (12 channels in total) located, respectively, on the ground at one side of the building, at the bottom slab of the building, at the roof of the isolated building and at the roof of the conventional building. Several small earthquakes have occurred at the site, with peak ground accelerations ranging from 0,65 to 58,72 cm/s^2 with dominant frequencies between 2 and 20 Hz and vertical peak ground accelerations ranging from 0.55 to 40.67 cm/s^2.

On the bridge, a strong motion network of 9 uniaxial forced balance accelerometers and 4 triaxial accelerometers has been installed. The sensors record absolute accelerations in an analog way, with a nominal dynamic range at 135 dB, between 0.01 and 50 Hz, and at 145 dB, between 0.01 to 20 Hz. Due to these characteristics it is possible to record ambient vibrations and strong motions with the same sensors. Since its installation in August 1998, eleven seismic records have been obtained from Magnitude 4.3 to 6.0 events; however given that the epicenters have been located far from the bridge, the level of shaking has been quite low.

[1] *Dept. of Civil Engineering, University of Chile, Santiago, Chile, e-mail: mmoroni@cec.uchile.cl*
[2] *Dept. of Civil Engineering, University of Chile, Santiago, Chile.*
[3] *Dept. of Civil Engineering, University of Chile, Santiago, Chile.*
[4] *Dept. of Civil Engineering, University of Chile, Santiago, Chile.*
[5] *Dept. of Civil Engineering, University of Chile, Santiago, Chile.*

The Line 5 of the Santiago Metro-Train has most of its extension elevated at an average height of 8 meters above the street level. It is composed of postensioned reinforced concrete simply supported beams, 27 to 36m long resting on neoprene bearings. A strong motion network of 12 channels has been installed last year on one bay of the structure. Fourteen moderate events have been recorded up to date.

Different methods were used to determine the dynamic characteristics of the structures and to reproduce the recorded information. Non-parametric system identification in the frequency and time domain were performed for the more severe records obtained for each structure. The Fourier spectra and transfer functions of the earthquake responses were determined from the characteristic peaks of the Fourier amplitude. Both the amplitude and the phase were considered in these identification processes. The parametric analysis includes a three dimensional model of each structure.

COMUNIDAD ANDALUCIA BUILDING

The Comunidad Andalucía buildings correspond to a low cost housing project. Each structure weighs approximately 1630 kN and measures 10 by 6 m in plan. While the first floor is structured with reinforced concrete walls, the upper three are of confined masonry. All floors have a 10 cm thick reinforced concrete slab. The isolated building is supported on eight high damping rubber isolators who rest on foot foundations. The foot foundations are connected between them with reinforced concrete beams.

The bearings, 31.5 cm in diameter and 32 cm high, were composed of 34 layers of 6.7 mm thick rubber and 33 2mm steel shims. The design criteria sought to produce a shift in the fundamental period of the building from 0.1s for the rigid foundation, to near 2.0 s for the isolated case. The bearings had to resist vertical loads of 350 kN and accept lateral displacements of 20 cm. The bearings were made in a local factory and subjected to a set of standard tests at the University of Chile and the University of California, Berkeley [Sarrazin et al, 1993]. The rubber shear modulus varied from 3.2 MPa for small strains to 0.65 MPa for 50% shear strain, while the average equivalent damping ratio was 15 %.

A number of research activities were planned to study the behavior of the isolated building and compare it to its conventional twin. Such tests included measurements of ambient vibrations, static load testing, dynamic pull back testing and monitoring of response to seismic activity, [Moroni et al, 1998]

The most important event up to date, magnitude 5.9, occurred on February 1996. Figure 1 shows the ratio of the maximum peak acceleration on the roofs of the conventional and isolated buildings as a function of the maximum peak ground acceleration for different earthquakes. Each point represents the average ratio of the two horizontal directions. Clearly, the effectiveness of the isolation system increases with the intensity of the motion. The same figure shows the ratio between the roof acceleration at the isolated building and the ground acceleration. The results suggest a trend toward a reduction as the intensity of the earthquake increases.

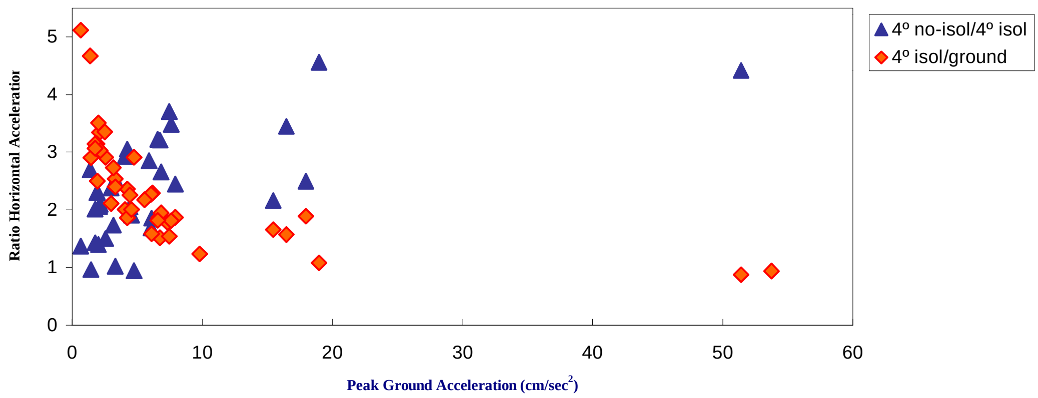

Figure 1. Ratio of Horizontal Peak Acceleration

Figure 2 shows similar ratios as before but for peak vertical accelerations. In this case there is amplification between the ground and the first floor and the roof of the buildings for all the records. The largest amplification occurs between the ground and the first floor, so it can be presumed that it is due mainly to the isolation system.

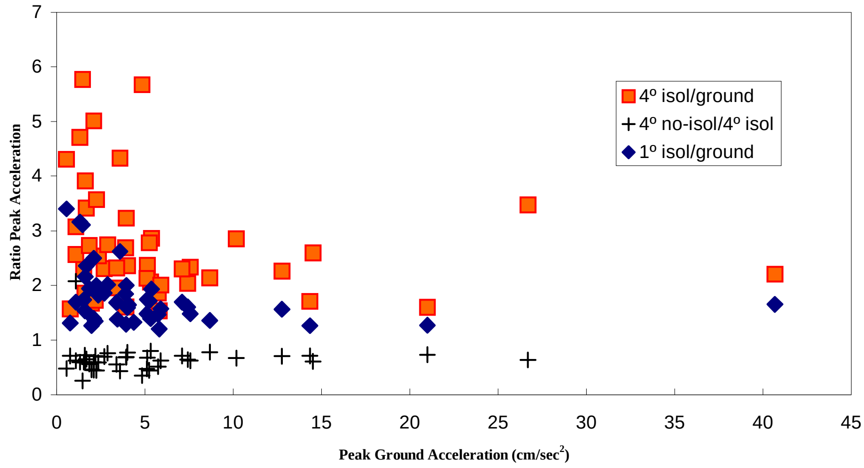

Figure 2. Ratio of Peak Vertical Acceleration

Fourier spectra and transfer function of the earthquakes responses were evaluated and the fundamental frequencies were determined as the characteristic peaks on the amplitude spectra. Both the amplitude and the phase were considered in the identification process. Figure 3 shows the frequency range for different records as a function of the Arias Intensity of the ground motion for the EW direction. The nonlinear behavior of the response is clear: larger input energy of the earthquake is related to lower frequency of the building. The values obtained for very low intensity earthquakes agree well with the microtremors. For larger earthquake motion the frequencies are lower, but still far away from the target value of 0.5 Hz considered in the design phase. The predominant vertical frequency for ambient vibration as well as for small events is about 15 Hz.

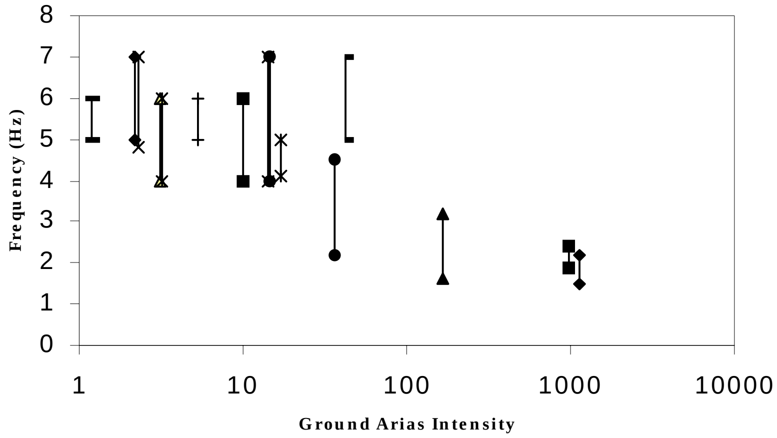

Figure 3. Frequency Range for Different Record

A simple equivalent linear SDOF model with viscous damping, with dynamic properties determined using the modal identification method proposed by [Beck, 1978] can reproduce the recorded behavior. The agreement observed between the registered and the predicted responses for horizontal as well as vertical directions are quite good [Riveros, 1998], [Rojas, 1998]; equivalent critical damping ratio ranges between 12.5 and 16 % for the horizontal analysis and between 4 to 5 % for the vertical analysis. In the case of ambient vibrations, the approximate equivalent critical damping ratio obtained from the power density spectrum was 2-3 %.

A second model consisted of a three-dimensional member-by-member definition of the isolated superstructure and an equivalent linear model with viscous damping to represent the individual isolators. Complete time-history responses were obtained using the 3D-BASIS-TABS, [Reinhorn et al, 1994]. The bearing stiffness properties and the equivalent damping ratio were selected to match the fundamental periods, peak acceleration and Fourier spectrum amplitude of the recorded responses. Depending on the records analyzed, the bearing stiffness range between 34.6 kN/cm to 44 kN/cm in the longitudinal direction and between 25.7 kN/cm to 41 kN/cm in the transverse direction. These values were above those assumed in the design phase (2.16 kN/cm) and represent negligible deformation of the isolators. Equivalent damping ratios between 8.5 % and 13.5 % were used.

Superstructure modal damping was set at 2 % in each mode. However, none of these models can represent the vertical and rocking motions that are present in the recorded response. A finite element model is required to represent the last effect.

MARGA-MARGA BRIDGE

The Marga-Marga Bridge is composed of 4 continuous steel beam structure, 383 m long, supported every 50 m by seven concrete single-column bents. Each beam is supported by nine high damping rubber bearings located at every pier and at the two abutments. The concrete hollow piers are 10 x 2 m and its height varies between 22 and 30 m; the five interior piers are founded on 1 m diameter concrete piles, 14 to 31 m long.

The bearings resting on the piers are 85 x 55 in plan, and those resting on the south and north abutments are 70 x 50 and 50 x 50 cm, respectively. All are 30 cm height, with 20.4 cm of rubber. The bearings design horizontal stiffness was 17.0, 12.7 and 9.2 kN/cm, respectively, and the design horizontal displacement was 15.6 cm. All the bearings were tested at Universidad de Chile, [Boroschek et al, 1997].

On the bridge, a strong motion network of 9 uniaxial forced balance accelerometers and 4 triaxial accelerometers has been installed. The sensors located in the structure are connected to an 18-channel central recording unit and those located in the free-field are connected to a 3 stand alone recording station

Microvibration measures were taken in various opportunities: before and after the bridge was opened to traffic, and during day and night periods. The first transverse mode has a frequency of 1.05 Hz and the first longitudinal mode has a frequency of 1.85 Hz. The structure shows coupling in the longitudinal and transverse direction. The longitudinal vibrations registered at the south end of the slab and at the abutment show negligible relative motion between the two points. That means that the sliding lateral supports of the bridge at that end are not properly working.

Two different effects can be seen at the isolation system. Vibrations caused by traffic on the slab are reduced at the support. Movements coming from the base of the piers are not transmitted to the slab. In this latter case the pier behaves as a cantilever.

Since the set up of the strong motion network in August 1998**,** twelve records have been obtained, as consequence of Magnitude 4.3 to 6.0 events; however, given that the epicenters have been located far from the bridge, the level of shaking has been quite low. Predominant response frequencies range from 1 to 1.7 Hz in the transverse direction and from 1.5 to 1.6 Hz in the longitudinal direction.

A 3-D computer model consisting in 2730 degrees of freedom was developed. To obtain frequencies and modal shapes similar to those measured, the following parameters were modified: the types of supports at the abutments, at the piers and at the piles level; the mechanical properties of the concrete; and the bearing stiffness properties. Table 1 shows a comparison between the recorded and predicted dynamic properties.

Table 1. Modal Frequencies, Marga-Marga Bridge

Mode	Experimental		Analitycal			
	Frequencies Hz	Direction	Frequencies Hz	Effective Mass Factors		
				X-Dir	Y-Dir	Z-Dir
1	1.05	Trans	1.063	0	51.485	0
2	1.18	Trans	1.397	0	0.021	0
3	1.5	Trans	2.186	0	4.778	0
4	1.62-1.67	Trans-Long				
5	1.85	Long-Tran-Vert	1.857	0.028	0	0.111
6	2.11	Long-Tran-Vert	2.108	60.167	0.001	0.001

SANTIAGO METRO-TRAIN

The Line 5 of the Santiago Metro-train has an elevated section, 5810 m long, supported on single-column piers, about 8 m height. The superstructure consists of simple supported U-shape beams, 27 to 36 m long, 1.8 m height, composed of a 30 cm postensioned reinforced concrete slab connected to the bottom flange of two precast beams. They rest on neoprene pads. These pads were designed not only for thermal expansion purpose but also for seismic isolation. The natural period of vibration of the structural system was increased due to the bearings from 0.68 s to 1 s, thus decreasing considerably the value of the design accelerations. The rectangular hollow concrete piers are 2.4 x 1.4 m, 30 cm thick, and are founded on hollow rectangular footing, 7 to 12 m deep. The soil condition at the site, as indicated by the soil boring-logs for bore holes located near the site, consists of layers of grave. Even though the superstructure is simple supported between adjacent piers, stiffness coupling across the slab joints between spans exists due to the stiffness of the rails.

The bearings are 30 x 60 cm in plan, 52 mm height with 40 mm of neoprene. The neoprene shear modulus determined by testing for 50 % shear strain was 1.1 MPa and the equivalent damping ratio for the same strain was 8 %.

A strong motion network of 3 uniaxial force balanced accelerometers and 3 triaxial accelerometers connected to a central record unit has been installed last year on one span of the structure, immediately to the south of Mirador Station. Figure 4 shows the locations of the different sensors.

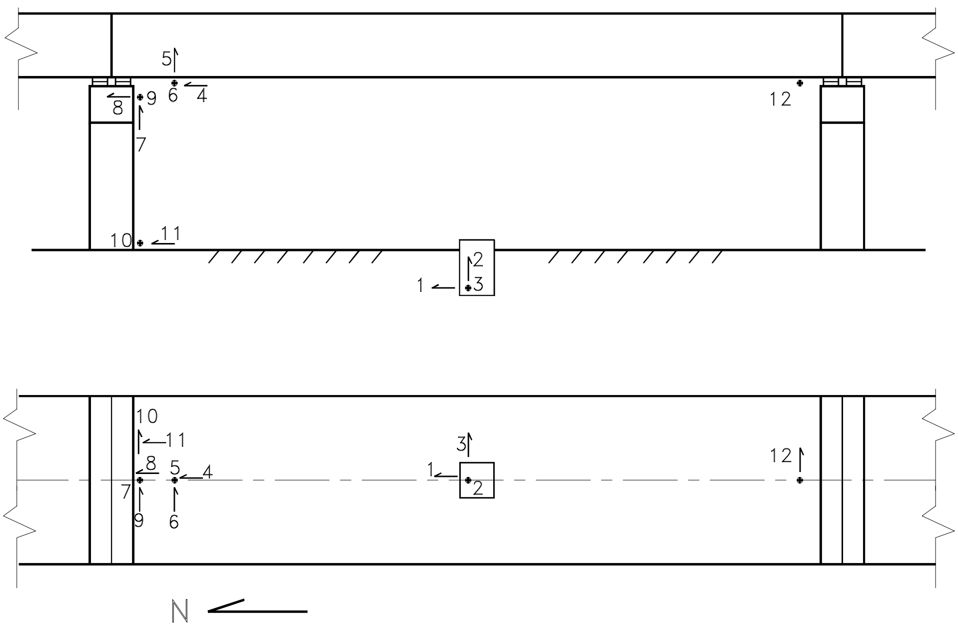

Figure 4. Strong Motion Network at the Santiago Metro, Chile

With this array, ambient vibrations have been recorded with and without the presence of the train. A clear shift in the structure predominant frequencies due to the presence of the train is observed. The variation of the natural frequencies identified by means of spectral analysis are shown in Table 2.

Fourteen moderate seismic events have been recorded up to date, with magnitude ranging from 4.2 to 6.2, being the largest peak ground acceleration 0.14 g . Despite this low level of excitation, there is appreciable motion amplification between the free field and the top of the column, but the effect of the isolation grows up as the earthquake intensity increases, especially in the transverse direction. Figure 5 shows the ratio between sensors located above and over the neoprene pads in the longitudinal (sensor 4,8), transversal (sensor 6,9) and vertical (sensor 5,7).

Table 2. Modal Frequencies Hz, Metro-Train

Mode	29/07/98 Earthquake		Microvibration		Direction
	Minimum	Maximum	With Train	Without Train	
1	1.9	2.59	2.34	2.44	Long- Vert
2	1.71	2.44	2.10	2.44	Trans
3	2.78	3.12	2.76-2.86	2.90	Vert
4	3.22	3.51		3.69	Long-Vert
5	3.61	4.05	3.59	3.86	Long-Vert
6	3.36	3.95		3.86	Trans
7	4.1	4.33	4.05-4.25	4.34	Trans
8	4.69	5.12		4.91	Trans

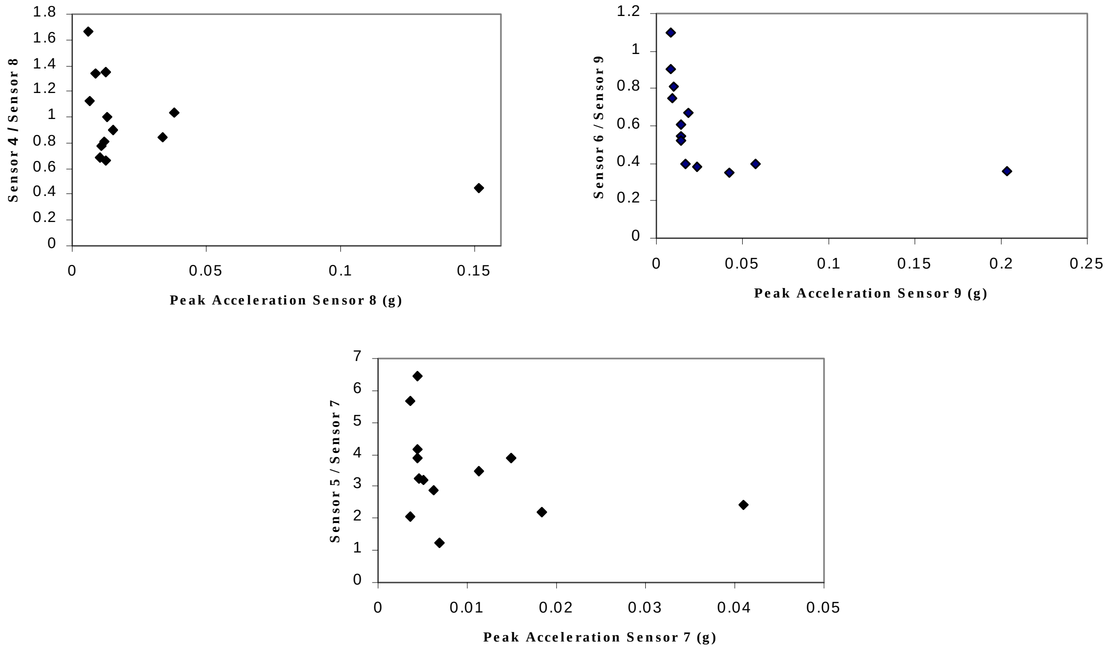

Figure 5. Peak Acceleration Ratio

As observed from the ambient excitations the structure has a distinct non linear behavior. The same behavior is observed for the low-level seismic events, so the natural frequencies vary depending on the excitation level. Table 2 contains the frequency range obtained for the 29/07/98 earthquake.

A 3-D equivalent linear computer model was prepared for three consecutive spans of the structure. The stiffness of the bearing pads, the foundation soil modulus and the concrete mechanical properties were modified to obtain similar dynamic characteristics as those derived from ambient vibration analysis. After that, the final model was subjected to the 29/07/98 record. In this latter case the damping of the structure was used to better fit the analytical with the recorded response. Figure 6 shows a comparison of the acceleration response spectra at sensor locations 6 (north beam), 12 (south beam), 9 (pier top) and 10 (pier base).

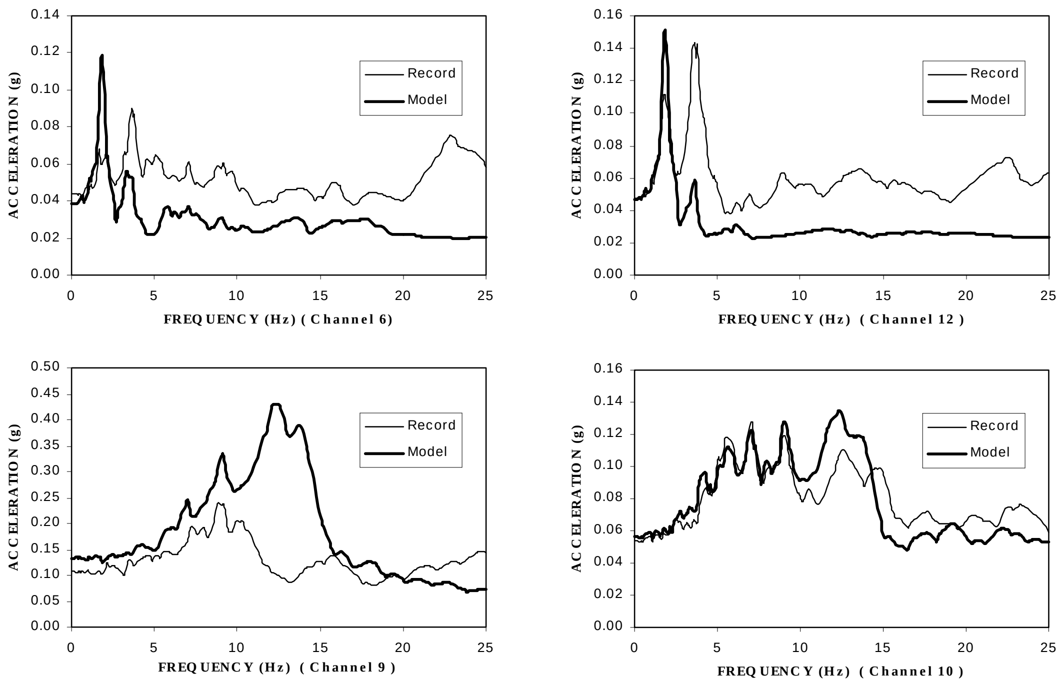

Figure 6. 5% Damped Acceleration Response Spectra

The analytical model captured the transverse fundamental mode response at the frequency of 1.81 Hz. Nevertheless the overall response match is not good due to:

- Structural response records are relatively low; parameters not included in typical models that try to predict strong motion response, like friction or interaction with secondary elements affect considerably the low motion response.
- The bearings exhibit a strong nonlinear behavior for small events. These can be expected from the derived experimental curve for the neoprene showed in figure 7. This strong nonlinearity at low levels of deformation is reduced for large amplitude responses.

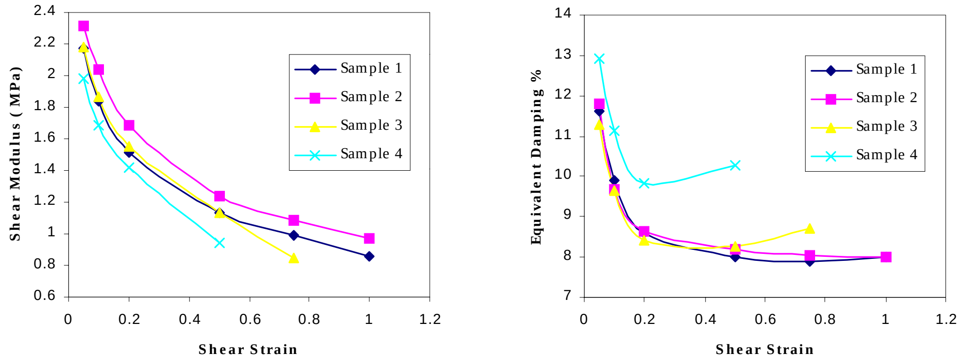

Figure 7. Neoprene Shear Modulus and Equivalent Damping Ratio.

CONCLUSIONS

Ambient vibrations and seismic induced vibrations recorded in three isolated structures of different types have been studied. The data obtained up to date provides valuable information for understanding the seismic response of the structure. Spectral analysis techniques were used in order to identify natural frequencies. Computer structural models were developed to correlate the observed and model frequencies.

For the isolated building, frequencies are identified quite precisely. On the contrary, for the bridge structures the process is rather difficult, because most of the modes are coupled and therefore, more elaborated techniques must be used. Something similar occurs with the computer models; the lateral response of the building can be represented even with a SDOF model, while for the bridge structures 3-D models are compulsory and several parameters must be taken in to account in order to properly represent the structure.

A general conclusion is that these latter types of structures even for relatively low level excitations responds in a non linear fashion limiting the applicability of linear models.

It can be seen from the data that, although the intensities of the motions were small, the isolation systems were effective in reducing the horizontal peak accelerations in the structures. For larger motions, the effectiveness of the isolation system should increase due to the nonlinearity of the force-displacement relationship of the isolation devices. However, amplification has been observed in the vertical direction. It remains to be seen how if this behavior will occur for larger earthquakes.

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