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RESEARCH AND APPLICATIONS ON BASE ISOLATION AND PASSIVE ENERGY DISSIPATION GOING ON IN CHILE

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ABSTRACT

This paper contains the research activities and applications going on in Chile on base isolation and passive energy dissipation. In particular the following items are covered:
- Analysis of records obtained in three seismically isolated structures located in the central zone of Chile. These structures have been instrumented with local strong motion network.
- Experimental results from prototype damper testing as well as study of potential application of damper systems in industrial buildings.
- Code development for the design of base isolated structures.
- New projects that are being constructed or in a design phase.
- New available testing facilities.

1. INTRODUCTION

At the Department of Civil Engineering of the University of Chile, a team has been working on seismic isolation and passive energy dissipation for many years. As result of their research a four-story building resting on seismic isolators has been built in Santiago; a local strong motion network has been installed on it and on a similar building but with conventional foundation, in order to compare their seismic behavior. This team also participated in the design and testing of the isolators for Marga-Marga Bridge in Viña del Mar, and in the installation of strong motion network on that bridge and on the Santiago Metro's Line 5. Others works include testing of high damping rubber dissipative devices, improvement of testing facilities, feasibility studies on the use of dissipative devices on industrial buildings and draft preparation of seismic design codes for isolated structures.

External effects of these research efforts can be seen in the number of new projects that include base isolation or passive energy dissipation devices, specially bridges, that are being constructed or in the design phase. Conicyt has funded a new facility installed at the Catholic University of Chile for testing bearings and dampers.

2. ANALYSIS OF RECORDS

Three seismically isolated structures located in the central zone of Chile 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.

Different methods have been used to determine the dynamic characteristics of the structures and to reproduce the recorded information. Computer structural models have been developed to correlate observed and model frequencies.

2.1 Comunidad Andalucía Building

The Comunidad Andalucía buildings described by Moroni et al 1998, correspond to a low cost housing project. They are instrumented since 1992; the most important event up to date, magnitude 5.9, occurred on February 1996. The effectiveness of the isolation system increases with the intensity of the motion, however some amplification exists in the vertical direction. The largest amplification in the latter direction occurs between the ground and the first floor, so it can be presumed that it is due mainly to the isolation system.

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 1 shows the frequency range for different records as a function of the Arias Intensity of the ground motion for the NS direction. The nonlinear behavior of the response is clear: larger input energy of the earthquake is related to lower frequency of the building. 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 1. Frequency Range for Different Record](image)

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%.

2.2 Marga-Marga Bridge

The Marga-Marga Bridge as described by Boroschek et al 1997, is composed of 4 continuous steel beam structure, 383 m long, supported every 50 m by seven concrete single-column bents.

Ambient vibration 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 at this level of motion.

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.

2.3 Santiago Metro-Train

The Line 5 of the Santiago Metro-Train, as described by Valdebenito 1999, 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 posttioned 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.

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. 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 is shown in Table 1.
Table 1. Modal Frequencies Hz, Metro-Train

<table>
<thead>
<tr>
<th>Mode</th>
<th>29/07/98 Earthquake</th>
<th>Microvibration</th>
<th>Direction</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Minimum</td>
<td>Maximum</td>
<td>With Train</td>
</tr>
<tr>
<td>1</td>
<td>1.9</td>
<td>2.59</td>
<td>2.34</td>
</tr>
<tr>
<td>2</td>
<td>1.71</td>
<td>2.44</td>
<td>2.10</td>
</tr>
<tr>
<td>3</td>
<td>2.78</td>
<td>3.12</td>
<td>2.76-2.86</td>
</tr>
<tr>
<td>4</td>
<td>3.22</td>
<td>3.51</td>
<td>3.69</td>
</tr>
<tr>
<td>5</td>
<td>3.61</td>
<td>4.05</td>
<td>3.59</td>
</tr>
<tr>
<td>6</td>
<td>3.36</td>
<td>3.95</td>
<td></td>
</tr>
<tr>
<td>7</td>
<td>4.1</td>
<td>4.33</td>
<td>4.05-4.25</td>
</tr>
<tr>
<td>8</td>
<td>4.69</td>
<td>5.12</td>
<td></td>
</tr>
</tbody>
</table>

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 increases as the earthquake intensity increases, especially in the transverse direction. Figure 2 shows peak acceleration ratios between sensors located at the ground and above and over the pads in the longitudinal (sensor 1,8,4), transversal (sensor 3,9,6) and vertical (sensor 2,7,5) direction.

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 1 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. 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. The strong non-linearity at low levels of deformation is reduced for large amplitude responses.
Figure 2. Peak Acceleration Ratio
3. APPLICATION OF DAMPER SYSTEMS TO INDUSTRIAL BUILDINGS

In Chile, the usual practice in the design of steel industrial building is to satisfy seismic strength and drift requirements through the inclusion of a strong bracing system. Columns represent the vertical load bearing system. The potential application of an energy dissipation bracing system for the seismic protection of two existing steel industrial buildings: a molybdenite roaster plant and a copper concentrate storage building has been studied.

3.1 Design of Damper Devices

Physical characteristics of the rubber were evaluated by testing shear test samples for sinusoidal excitations at prescribed strains and frequencies. The shear modulus and the equivalent damping ratio are dependent of shear strain amplitudes and less affected by the frequency variation. The loss factor ($\eta = 0.12$–0.14) is rather small compared to different materials used by other authors, [Sooong, 1997].

A prototype cylindrical device consisting of two concentric pipes filled with high damping rubber was designed and tested [Vera, 1999]. The damper device and its connections to the test machine are shown in Figure 3. The outer diameter is 12.7 cm and 3 mm thick, the inner diameter is 7.62 cm and 6 mm thick, and the rubber is 22.4 mm thick.

![Damper Testing Device](image)

Figure 3. Damper Testing Device.

The damper was subjected to dynamic tests at different frequencies and shear strains. The target damper stiffness was 6.3 ton/cm for 50% shear strain deformation. The target stiffness has been attained, with little margin of error. The rubber equivalent damping ratio was 9% for 50% shear strain, while the damper equivalent damping ratio was about 6.4-7% for the same shear strains.

3.2 Application to selected buildings

Based on the experimental results, damper devices were designed to be included in two industrial buildings. As an initial trial the dampers were modeled as viscous. Others mathematical models are being tested. The modal strain energy method was employed to predict the added damping to the structures.

Iterations involving the rubber storage modulus and loss factor were done in order to adjust the target frequency and the estimated damping ratio. The resulting story drift should...
represent a damper's deformation of about 50%, in order to insure a rubber shear modulus almost constant. If this does not occur either the rubber thickness or the device length must be modified. The brace elements must resist the seismic forces; if not, new sections should be provided. Finally, the invariance in the mode shapes must be verified.

The first building to be studied was a six-story steel structure [Loyola, 1999]. It weighs 11000 kN (including 25% live loads), measures 30 by 30 m in plan and is 23.5 m height. It has 6 equally spaced moment resisting frames every 6 m, in each direction. The four exterior frames are braced. All connections are bolted. The brace floor system is different in every story due to functional conditions. A reinforced concrete silo 6.55 m diameter and 20 m height is located on a hollow space but is completely disconnected from the main building. Story heights from bottom to top are 4.57, 3.30, 3.12, 2.95, 3.89 and 5.67 m.

The original structure was designed to resist a base shear force of 6000 kN mostly by means of the first floor bracing system. A finite element model with 4300 DOF and the SAP2000 computer program were used in the analysis. Table 2 shows the calculated frequencies for the original, bare and damped structure.

<table>
<thead>
<tr>
<th>Mode</th>
<th>Original</th>
<th>Bare</th>
<th>Damped</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>2.16</td>
<td>0.91</td>
<td>1.56</td>
</tr>
<tr>
<td>2</td>
<td>2.18</td>
<td>0.95</td>
<td>1.59</td>
</tr>
<tr>
<td>3</td>
<td>2.48</td>
<td>1.24</td>
<td>2.25</td>
</tr>
</tbody>
</table>

The second building [Retamales, 1999], weighs 24770 kN, 1690 kN representing the structure dead load and 23080 kN the weight of 7 silos suspended from the structure at 20 m above the ground. The silos are filled with copper concentrate or limestone. Its overall dimensions are 24 x 15 m in plan and 20.6 m in height. Story heights from bottom to top are 4.08, 2.22, 3.75, 3.77, 3.38 and 3.38 m. Four moment resisting frames in the longitudinal direction and 5 equally spaced in the transverse direction constitute the load bearing structure. All exterior frames and one of the interiors in the transverse direction are braced.

A finite element model that included the silos stiffness was used for the analysis. Table 3 shows the calculated frequencies for the original, bare and damped structure.

<table>
<thead>
<tr>
<th>Mode</th>
<th>Original</th>
<th>Bare</th>
<th>Damped</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1.78</td>
<td>0.38</td>
<td>0.88</td>
</tr>
<tr>
<td>2</td>
<td>1.85</td>
<td>0.44</td>
<td>1.17</td>
</tr>
<tr>
<td>3</td>
<td>2.63</td>
<td>0.58</td>
<td>1.69</td>
</tr>
</tbody>
</table>

3.3 Seismic behavior

In order to verify the improvement in seismic behavior, the buildings were subjected to the Antofagasta 1995 record (Magnitude 7.8). This record was chosen because it was the closest to the buildings locations registered up to the date and because it represents similar site soil
conditions. Time-history analyses were carried out for the original, bare and damped structures. Both horizontal components were applied simultaneously. Maximum peak ground acceleration is 0.28g and 0.22g in the E-W and N-S direction, respectively. Base shear, floor acceleration, lateral displacement, story drift and element forces were calculated for all cases. Table 4 shows the base shear as percentage of the total weight for both buildings.

<table>
<thead>
<tr>
<th>Structure Type</th>
<th>Building 1</th>
<th>Building 2</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$C_x$</td>
<td>$C_y$</td>
</tr>
<tr>
<td>Original</td>
<td>0.36</td>
<td>0.46</td>
</tr>
<tr>
<td>Bare</td>
<td>0.17</td>
<td>0.19</td>
</tr>
<tr>
<td>Damped</td>
<td>0.22</td>
<td>0.29</td>
</tr>
</tbody>
</table>

4. CODE DEVELOPMENT

4.1 Seismic design of industrial buildings NCh2369.c97

The NCh2369.c97 [INN, 1997] includes guidelines for the design of structures with base isolation or with energy dissipation systems. It establishes that this type of structures must be analyzed and design according to UBC 97, but local displacement response spectra must be determined consistent with the seismic forces prescribed in the same code.

The mathematical model of the structure must represent the mass and rigidity distribution of it, assuring an adequate calculation of the main characteristics of its dynamic response. A tridimensional model of the superstructure that takes into account the vertical displacement of the isolators must be used for those structures that are sensible to those movements as are those with friction bearings or with doweled bearings. The modal spectral analysis can be used only if a validated linear equivalent form could model the device. The force-deformation relationships considered in the analysis must be properly based and justified by laboratory tests.

The response spectra acceleration given by eq. 1 depends explicitly on the structural damping ratio $\xi$. Others parameters are the importance factor $I$, the effective ground acceleration $A_g$, which depends on the seismic zone, the response modification factor $R$ and $T$ and $n$ that depends on the soil conditions.

$$S_s = IC_g$$

$$C = \frac{2.75A_g}{gR} \left( \frac{T}{T} \right)^8 \left( \frac{0.05}{\xi} \right)^{0.4}$$  \hspace{1cm} (1)

Unlike the conventional structures there is not a minimum base shear requirement while the drift requirement are only applied to the superstructure.

4.2 Seismic Code for isolated bridges

A complete review of the available code for isolated structures was prepared by Fuenzalida, 1999, and considering the Chilean practice and local seismicity a set of guidelines has been
proposed for the design of seismic isolated bridges with elastomeric bearings. Only one level of seismic input is considered—a level of ground motion that has a 10% chance of being exceeded in a 50-year period; a linear behavior is expected in the structure with the exceptions of the isolation interface; time-history analysis is recommended and the testing requirements of AASHTO 1997 have been included.

5. NEW PROJECTS

Five bridges that include base isolation or dissipation devices are being constructed along the North-South Route. The bearing design and testing follows the AASHTO 1997 code although the seismic design forces has not been reduced due to the isolation. Figure 4 shows a view of Limari Bridge and one of its neoprene bearings.

Figure 4. Limari Bridge.
Amolanas Bridge (268 m long) will include four viscoelastic dampers connecting a steel box girder to the concrete abutments, while at Rio Maule Bridge (400 m long) at Constitucion, viscoelastic fluid damper have been considered between a concrete box girder and the south abutment.

With respect to buildings, the new hospital of the Catholic University includes high damping rubber bearings.

6. NEW TESTING FACILITIES

6.1 Shear testing machine

A small size, low cost, mechanical device for testing the rubber specimens in direct shear has been developed by Herrera, 1998. It is made of steel and uses a 1/3 HP DC motor to generate a sinusoidal displacement pattern. Its performance has been validated comparing the results obtained testing specimens of different ages of fabrication and origin, with those obtained using a MTS universal testing machine. The general appearance of the machine is showed in figure 5.

![Shear Testing Machine](image)

Figure 5. Shear Testing Machine

- The machine is able to apply control displacements that generates shear strain on the specimens, between 2 % and 200 %.
- The frequency of the sinusoidal displacement can be varied between 0.5 and 2 Hz.
- The machine can test rubbers with a shear modulus between 6 and 10 Kgf/cm² for 50 % of deformation.
- The values of the shear modulus and the damping ratio fit with an acceptable range of variation with those obtained with the MTS machine, as is shown in figure 6. The maximum differences occur in the damping ratio, particularly for the oldest specimens, reaching up to 20 %.
- The cost of the prototype is approximately US$1000.
Fig.6: Values of G and \( \beta \) obtained with the prototype and the MTS machine, set 1.

6.2 Catholic University testing facilities

A new facility for testing bearings and dampers has been recently inaugurated at the Catholic University of Chile. It includes a 100 ton MTS actuator with 100 cm displacement capability at a velocity of 20 cm/s and a 25 ton MTS actuator with 50 cm displacement capability at a velocity of 120 cm/s.

7. 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 into account in order to properly represent the structure. A general conclusion is that these latter types of structures even for relatively low level excitations respond 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 non-linearity of the force-displacement relationship of the isolation devices. However, amplification has been observed in the vertical direction. It remains to be seen how this behavior will occur for larger earthquakes.

The feasibility of including VE dampers in two industrial buildings has been studied. The modal strain energy method has been used to estimate the structural damping added by the dampers. The added damping depends mainly on the loss factor of the rubber $\eta$, and the frequency ratio of the structure without and with dampers. Its maximum value is $\eta/2$, therefore, for the rubber compound used in this study, that value is limited to 6-7%.

Although the amount of added damping was moderate, its effect was significant. When the buildings including dampers were subjected to the Antofagasta 1995 earthquake, the maximum base shear and peak accelerations were reduced as compared to the conventional braced structures, without increasing the lateral displacement or interstory drifts. These results show that the viscoelastic dampers can be used to effectively reduce the overall response of structures at a reasonable cost.

New bridges that include base isolation or energy dissipation devices are being constructed or in design phase; testing facilities have improved; code draft for seismic design of isolated structures are being discussed. All these facts create a favorable ambient to the development of new applications on seismic isolation or energy dissipation devices.

8. ACKNOWLEDGMENT

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