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SYSTEM IDENTIFICATION OF AN ISOLATED STRUCTURE USING EARTHQUAKE RECORDS

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ABSTRACT

The growing interest of investors and stakeholders in protecting building contents and operation beyond the goals of current seismic design codes is generating an increased demand for the use of nonconventional seismic protection systems such as base isolation and energy dissipation devices. This paper focuses on the application of seismic identification techniques on earthquake records obtained in the first Chilean Base Isolated Building. Seismic responses recorded in this structure located in the capital city of Chile, range in Magnitude from 4 to 8.8. This last earthquake corresponds to the sixth largest event recorded in the world. Due to the strong nonlinear elastic response of the base isolation system, classical linear identification methods present poor results. Several procedures to identify the response are presented, including window the response data and nonlinear identification. Basic modal parameters like natural frequency and modal damping ratios are obtained as a function of response amplitude indicating the great benefit of the isolation system and the relevance of developing appropriate identification techniques for this type of highly nonlinear systems. Building isolation system performance is reflected in modal parameters variations observed in the analysis.

KEYWORDS

Structural health monitoring, nonlinear identification, earthquakes, base isolation.

INTRODUCTION

After the 27th February, 2010 Chilean earthquake, a renewed and increasing interest in additional seismic protection seismic in buildings was born in investors, stakeholders and structural engineering. Because of this several systems, like energy dissipating devices and base isolation, have been incorporated in new constructed structures. In high seismic countries, like Chile, recurrent damage due to low level earthquake in non-structural components is common. In severe earthquakes the structural and nonstructural is extremely large. As an example the Magnitude 8.8 event caused economic losses exceeding US$30 billion. A high percentage of these losses resulted from damage to nonstructural systems and components, such as architectural elements, building contents, and electric and mechanical equipment (Miranda *et al.* 2012). The increase use of these systems that are inherently nonlinear poses a challenge to Structural Health Monitoring. In this article we present some examples of isolated structures, one of them has been instrumented for more than 20 years. The main characteristics of the isolated structures are presented together with the models and procedures to identify their nonlinear characteristics.

EXPERIENCE OF BASE ISOLATED BUILDINGS IN CHILE

Almost two decades before the last great Chile earthquake, as part of the research projects developed at the University of Chile by Sarrazin & Moroni, a low cost isolated system was implemented in 1992 as an attempt to introduce this technology in developing countries. The Andalucia Condominium project was a pilot project conformed by one isolated building and a similar but conventional (fixed base) building (Moroni *et al.* 1998). In addition to the Andalucía Building, there are several pioneer Chilean projects, the Marga-Marga Bridge, developed by G+V Engineers in 1998 and the Military Hospital developed by Hoehmann Stagno & Associates in 2001 (Boroschek *et al.* 2012). The Chilean standard for the seismic design isolated buildings is the

NCh2745.Of2003 (INN 2003) which is partially based on the ASCE/SEI 7-98 provisions, adapted to the local seismicity conditions and to more stringent analysis and testing requirements. We present three examples of base isolated structures.

Torre del Sol Building

The Torre del Sol building, Figure 1, corresponds to a residential building with 19,000 m^2 and 19 stories (1 level for the isolation system, 2 underground parking levels, 14 residential floors, 1 level for equipment and 1 for the engine room (Bonelli Associates, ALCORP). A typical floor covers 53x12.9 m. The total height of the building, including the undergrounds, is 49.6 m. The seismic weight of the structure supported by the isolation system is 15,500 Tonf. According to the geotechnical report, the foundation soil corresponds to a stiff soil with average shear wave velocity in the first 30 m beneath the surface of 598 m/s. For this project, lead rubber bearings were chosen for a number of reasons, among which are: these systems have been extensively tested in the laboratory and their behavior has been thoroughly demonstrated in real seismic events; the ease in predicting and modeling their behavior for the purpose of analysis; the stability of the devices against environmental agents; and the independence of device's performance from the loading rate (RBA-DIS).

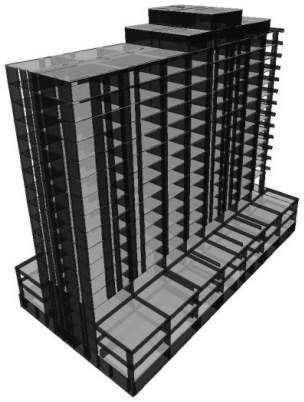

Figure 1. 3-D Numerical model

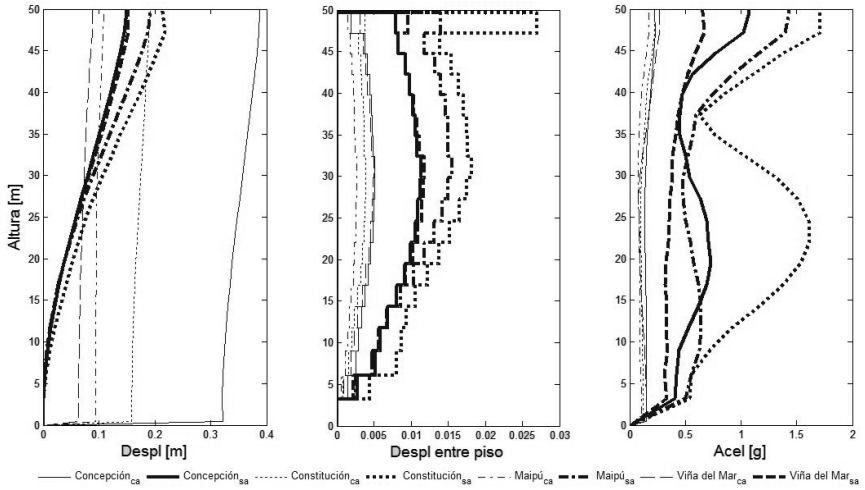

Figure 2. Seismic response of Torre del Sol building with and without base isolation

The seismic isolator prototypes and all production isolators were subjected to an exhaustive test series combining compression and shear in order to validate the properties assumed for the design and to verify the stability of those isolators against extreme seismic loads. In the design of the isolation system, a series of nonlinear dynamic analyses were carried out taking into consideration 7 seismic records compatible with the demands associated with the maximum considered earthquake according to the Chilean national standard for the seismic zone and ground conditions of the structure's site location. The analysis showed that the use of isolation allows for achieving a reduction of 88% in the average base shear of the structure. The design of the superstructure was made taking into consideration the maximum shear that could be transmitted through the isolation system. The demand of displacement is accommodated by the deformation of the isolators and not by the deformation of concrete walls. Figure 2 shows a comparison of the distribution of floor displacement, interstory drifts, and absolute floor accelerations of the base-isolated and fixed-base structures. The values shown represent the mean values obtained for the 7 records considered.

Claro Data Center Building

The second interesting and recent application of seismic isolation is the Claro Data Center shown in Figure 4. This building host one of the largest high-tech facilities in Chile. The total area of the building is approximately 35,000 m^2, distributed in 2 levels and a mechanical floor. The architectural, structural and isolation system designs considered the stringent requirements of ANSI/TIA-942 (TIA 2005) for Tier level 4. According to TIA's requirements, the infrastructure shall be fault-tolerant, with electrical power storage and distribution facilities with an expected availability beyond 99.995%. In order to protect the operation of the building following extreme seismic events, 340 lead rubber isolators (DIS), are used. The seismic responses estimated using the signals recorded during the M_w 8.8 2010 Chile earthquake is shown in Figure 5.

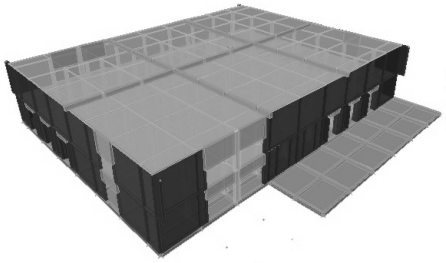

Figure 4. 3-D Numerical model of Claro Data Center

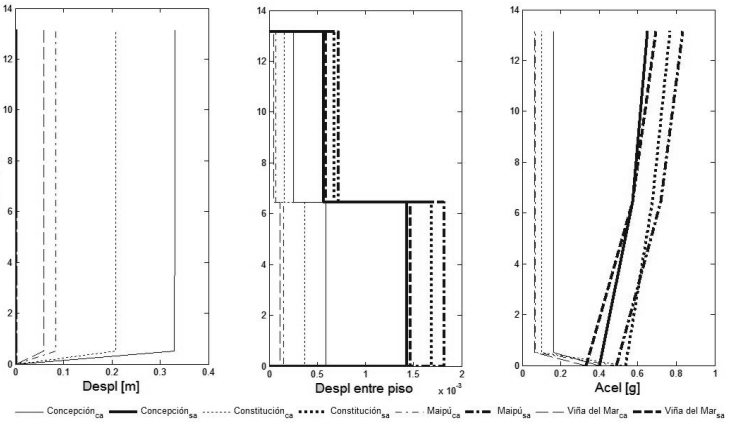

Figure 5. Seismic response of the Claro Data Center with and without base isolation

The Andalucia Condominium

The Andalucia Condominium is the first seismic base isolation structure in Chile. It is a 4 story concrete and masonry structure for low income families. The building rest on top of eight high damping rubber bearing located at ground level, Figure 6. To understand the behavior of the structure two identical building, the base isolated and one of fixed base building were instrumented with an accelerometer array.

The instrumentation consists of triaxial accelerometers with common triggering. The recording system has a 12 bit analog to digital converter so it is limited to earthquake events only. Figure 6 shows the position of accelerometers C, L and F. Signals registered from C and F sensors are presented in this paper, Table 1. Sampling frequency is 200 samples per second.

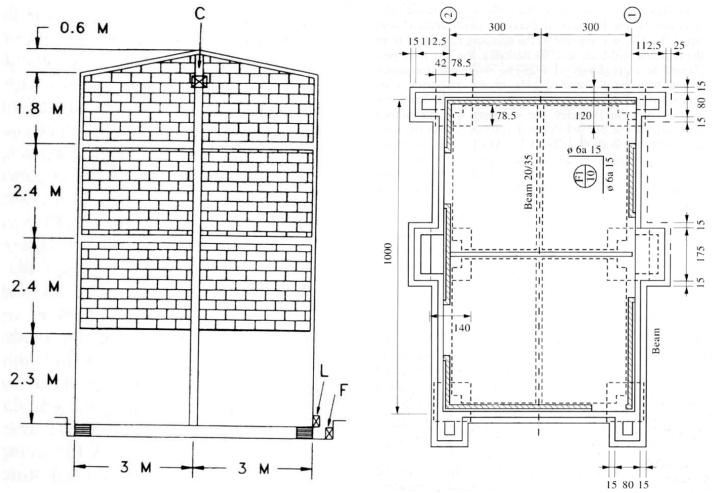

Figure 6. Andalucia Community Building. Global dimensions and isolator layout

Table 1. Sensors direction

Channel	Sens	Comp.	Level
1	F	EW	Ground
2		NS	
3		UD	
4	C	EW	4th
5		NS	
6		UD	

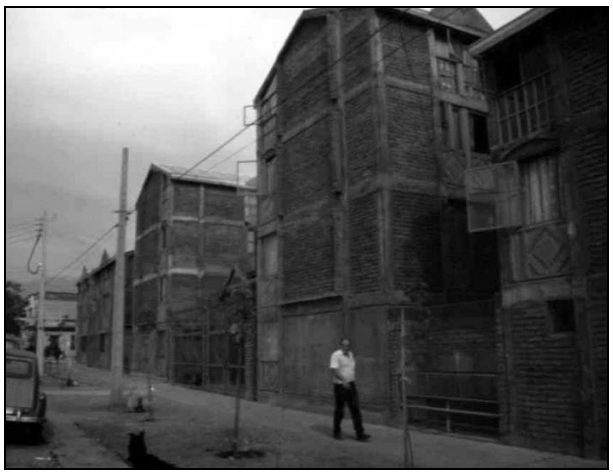

Figure 7. Andalucía Condominium buildings

Figure 8. Elastomeric Isolators

MODELS AND IDENTIFICATION METHODS

As in typical structure there is a need to validate the structural response of isolated systems as well as the device behavior. The basic characteristic of elastomeric isolators is its strong nonlinear elastic behavior. Also they are strongly influence by temperature, aging and vertical pressure. Recording system are the traditional ones but a close look at the isolation interface shall be considered. The basic system shall consider accelerometers below and on top of the isolation system and throughout the structure and displacement meters at the interface. In order to correlate the parameters with environmental variable a meteorological station is recommended.

Several models have been considered for the identification of elastomeric isolated structures. Some of these models are presented below.

Kelvin Voight Model and Maxwell Model

The viscoelastic model can be represented by damping and stiffness connected in parallel or in series. The equation of motion for the Kelvin Voight model, parallel system, is presented in Eq. 1. If the model is represented with this parameters in series, that is the Maxwell model, the equation of motion is given by Eq. 2.

$$C\dot{u} + K u = f \tag{1}$$

$$K\dot{u} = \frac{K}{C} \times f + \dot{f} \tag{2}$$

Where C=damping, K= stiffness, f=Excitation and u=displacement. Both correspond to the most basic representations of a viscoelastic isolator. An extension of these models corresponds to a *linear model of three parameters* (Soto, 2000). In this case the viscoelastic model can be represented by a damper and a spring connected in series with an additional spring connected in parallel. The equation of motion in this case is given by Eq. 3:

$$K_2 \left[u + \left\{ C \frac{\left[K_1 + K_2\right]}{K_1 K_2} \right\} \dot{u} \right] = f + \frac{C}{K_1} \dot{f} \tag{3}$$

This is only a composition of Eqs 1 and 2, considering the influence of the stiffness distribution over the isolation system.

Non-Linear Elastic Model

The non-linear elastic model assumes that both, damping and stiffness have a non-linear behavior. In this case the stiffness is represented by a bi-linear behavior, where stiffness changes for a characteristic displacement, and that both are connected in parallel. In this case the force associated to the non-linear stiffness is Eq. 4:

$$F_s = K_1 u + \left[\frac{K_2 - K_1}{3u_0^2} \right] \times u^3 \tag{4}$$

And the force associated with the non-linear damping is Eq. 5:

$$C_a = C_1 \dot{u} + \left[\frac{C_2 - C_1}{3\dot{u}_0^2} \right] \times \dot{u}^3 \tag{5}$$

Where u_0 =characteristic displacement and $\dot{u}_0$ =characteristic velocity

Considering Eqs 4 and 5, the equation of motion is given by:

$$C_1 \dot{u} + \left(\frac{C_2 - C_1}{3\dot{u}_0^2} \right) \dot{u}^3 + k_1 u + \left(\frac{K_2 - K_1}{3u_0^2} \right) u^3 = f \tag{6}$$

Bouc Wen Models

This model is constructed considering a complex function of the stiffness component as a restorative force of the isolator. The equation of motion for a SDOF system is given by the Eq. 7:

$$C\dot{u} + q = f \tag{7}$$

In which C=Damping Matrix, u =Displacement, q = Restoring force and f =Excitations. The restoring force is expressed as:

$$q = \alpha K + (1-\alpha) K Z \tag{8}$$

Where Z can be obtained from a differential equation that considers the time derivative of Z an its modulus, as given by Eq. 9:

$$\dot{Z} = A\dot{u} - \beta \dot{u} |Z|^n + \gamma |\dot{u}| |Z|^{n-1} Z \tag{9}$$

Biaxial Hysteretic Restoring Force Model

In order to consider the influence between the orthogonal directions in the isolator response, a modified Bouc Wen model is used (Park *et al.* 1986). The equation of motion of a 2-DOF system subject to two-dimensional excitations are given by Eq. 10:

$$[M]\begin{Bmatrix} \ddot{u}_x \\ \ddot{u}_y \end{Bmatrix} + [C]\begin{Bmatrix} \dot{u}_x \\ \dot{u}_y \end{Bmatrix} + \begin{Bmatrix} q_x \\ q_y \end{Bmatrix} = \begin{Bmatrix} f_x \\ f_y \end{Bmatrix} \tag{10}$$

In which [M] = Mass Matrix, [C] = Damping Matrix, u_x, u_y = Orthogonal displacements, q_x, q_y = Restoring forces and f_x, f_y = Orthogonal excitations. The restoring force vector is expressed as shown in Eq. 11:

$$\begin{Bmatrix} q_x \\ q_y \end{Bmatrix} = \alpha [K] \begin{Bmatrix} u_x \\ u_y \end{Bmatrix} + (1-\alpha)[K] \begin{Bmatrix} Z_x \\ Z_y \end{Bmatrix} \tag{11}$$

Where $[K]$=initial stiffness matrix, α =post yielding stiffness ratio and Z_x, Z_y = Hysteretic dimensionless components of the restoring force. In this case, Z_x and Z_y are described by the following differential equations, Eqs 12 and 13.

$$\dot{Z}_x = A\dot{u}_x - \beta |\dot{u}_x Z_x| Z_x - \gamma \dot{u}_x Z_x^2 - \beta |\dot{u}_y Z_y| Z_x - \gamma \dot{u}_y Z_x Z_y \tag{12}$$

$$\dot{Z}_y = A\dot{u}_y - \beta |\dot{u}_y Z_y| Z_y - \gamma \dot{u}_y Z_y^2 - \beta |\dot{u}_x Z_x| Z_y - \gamma \dot{u}_x Z_x Z_y \tag{13}$$

Where A, β *and* γ are dimensionless constants.

Multiple Shear Spring Models

A bidirectional model is also constructed from several shear trilinear springs arranged in a circle (Wada, 1985, Furukawa, 2005). Each spring has its own specific characteristic of initial stiffness, secondary stiffness, and yield displacement.

$$\begin{aligned} \Delta x_{ew} &= x_{ew}(t) - x_{ew}(t-1) \\ \Delta x_{ns} &= x_{ns}(t) - x_{ns}(t-1) \end{aligned} \tag{14}$$

Where Δx_{ns} and Δx_{ew} are equal to NS and EW displacement at time *t*. The total restoring force can be evaluated by the equations system Eqs15 and 16:

$$\begin{bmatrix}\Delta R_{ew}\\ \Delta R_{ns}\end{bmatrix}=\sum_{i=1}^{N}k_i'(t)\begin{bmatrix}\cos^2\theta_i & \cos\theta_i\square\text{in}\theta_i\\ \cos\theta_i\square\text{in}\theta_i & \sin^2\theta_i\end{bmatrix}\begin{bmatrix}\Delta x_{ew}\\ \Delta x_{ns}\end{bmatrix} \tag{15}$$

$$\vartheta_i=\frac{\pi}{N}(i-1)?(i=1,2\cdots,N)$$

$$Uy'=\frac{\sum_{i=1}^{N}\cos^2(\vartheta_i)}{\sum_{i=1}^{N}\left|\cos(\vartheta_i)\right|}Uy?k'=\frac{1}{\sum_{i=1}^{N}\left|\cos^2(\vartheta_i)\right|}k \tag{16}$$

Here, ΔR_{ew} and ΔR_{ns} are equals to increments of restoring forces along the EW and NS directions, $k_i'(t)=$ stiffness of *i*-th spring at time *t*, *N* is the number of shear springs, and ϑ_i is the angle between the EW axis and the *i*-th spring, $Uy'=$ yield displacement and $k'=$ each shear spring

IDENTIFICATION ISOLATION BEARING WITH NON- LINEAR BEHAVIOR

Common system identification methods that consider Linear Time-Invariant systems have been apply to traditional and base isolated structures despite its nonlinear response. This section presents different analysis methods that have been used for isolation systems.

Stewart and other (Stewart *et al.* 1999) described the seismic response and the system identification procedures used for four buildings in Los Angeles Area subjected to several earthquake events. The earthquake events produce accelerations lower than 0.21 g. For the identification, they used basic frequency response function techniques, like peak peaking, and two parametric techniques proposed by Safak:

- Cumulative error method (CEM): The transfer function is estimated by minimizing cumulative error for the entire history (Safak, 1991)
- Recursive prediction error method (RPEM): The transfer function is determined by recursively minimizing error for each time step, using time windows (Safak, 1989a and Safak, 1989b)

The transfer function is given by Eq. 17:

$$H(q)=\frac{B(q)}{A(q)} \tag{17}$$

Because the systems dynamic are always contaminated by noise, it includes two lineal filters representing the systems dynamic and noise dynamic. The signal and the noise are represented by Eqs 18, where $y_s(t)=$ equation for the signal $y_n(t)=$ equation for the noise, and combined as presented in Eq. 19. Then, considering a time *k* delay between input and output, from Eqs 20 and 21, an estimator could be constructed as in Eq. 22 that allow to define an error objective function (Eq. 24), and depending of a set of model parameters denoted by ϑ in Eq. 23, which are the polynomials coefficients of the estimator.

$$y_s(t)=\frac{B(q)}{A(q)F(q)}\quad y_n(t)=\frac{C(q)}{A(q)D(q)} \tag{18}$$

$$y(t)=y_s(t)+y_n(t) \tag{19}$$

$$A(q)y(t)=q^{-k}\frac{B(q)}{F(q)}x(t)+\frac{C(q)}{D(q)}e(t) \tag{20}$$

$$y(t)=\left[1-\frac{D(q)A(q)}{C(q)}\right]y(t)+q^{-k}\frac{D(q)B(q)}{C(q)F(q)}x(t)+e(t) \tag{21}$$

$$E\left[y(t)\right]=\hat{y}(t,\vartheta)=\left[1-\frac{D(q)A(q)}{C(q)}\right]y(t)+q^{-k}\frac{D(q)B(q)}{C(q)F(q)}x(t)+e(t) \tag{22}$$

$$\vartheta=(a_1,\ldots,a_{na},b_1,\ldots,b_{nb},c_1,\ldots,c_{nc},d_1,\ldots,d_{nd},f_1,\ldots,f_{nf})^T \tag{23}$$

$$\varepsilon(t,\vartheta)=y(t)-\hat{y}(t,\vartheta) \tag{24}$$

The least squares method is used as criteria for measuring errors. The total error in time *t* includes a weighting factor, as is observed in Eqs 25 and 26.

$$V(t,\vartheta)=\frac{1}{2}\gamma(t)\sum_{s=1}^{t}\beta(t,s)\varepsilon^{2}(s,\vartheta)=1 \tag{25}$$

$$\gamma(t)\sum_{s=1}^{t}\beta(t,s)=1 \tag{26}$$

Where $\beta(t,s)$ corresponds to the weighting factor and $\gamma(t)$ is the normalization factor. The model that minimizes the total estimation error is given by Eqs 17 to 30.

$$\frac{d}{d\vartheta}V(t,\vartheta)=0 \tag{27}$$

$$\vartheta(t)=\vartheta(t-1)+\gamma(t)R^{-1}(t)\psi(t)\varepsilon(t) \tag{28}$$

$$R(t)=R(t-1)+\gamma(t)\left[\psi(t)\psi(t)^{T}-R(t-1)\right] \tag{29}$$

$$\psi(t)=\frac{d\varepsilon(t,\vartheta)}{d\vartheta}=\frac{dy(t,\vartheta)}{d\vartheta} \tag{30}$$

Yoshimoto *et al.* (2005) used the *Multivariable Output Error State Space (MOESP*) method in a base isolated building under small shaking levels. MOESP is a method that obtains state space matrices from input and output data using Hankel and Toeplitz matrices and time windows analysis. The model is described by Eq. 31. This system is solved for each time window determining the modal properties of the system.

$$\begin{aligned} x_{k+1} &= Ax_k + Bu_k \\ y_k &= C_k x_k + Du_k \end{aligned} \tag{31}$$

Furukawa *et al.* (2005) use the *Prediction Error Method* and a nonlinear model derived by Wada. This method consists in considering a parameter vector (ϑ) in the restoring force. The system discrete-time state space is given by Eq. 42:

$$\begin{aligned} x_{k+1} &= A_d(\vartheta,t)x_k + B_d(\vartheta,t)u_k + w_k \\ y_k &= C(\vartheta,t)x_k + v_k \end{aligned} \tag{42}$$

Where $\mathbf{x_k}$ is the state vector at k, y_k the Output of the system at k, w_t the Gaussian process noise at k, v_t = Gaussian measurement noise k, and A_d, B_d = Matrices, and C_d = observation process noise matrix. Introducing an estimator for the Gaussian noise and measurement depending of a Kalman gain, the Eq. 42 is transformed to Eq. 43.

$$\begin{aligned} \hat{x}_{k+1} &= A_d(\vartheta,t)x_k + B_d(\vartheta,t)u_k + K\left[y_k - C_d\hat{x}_k\right] \\ \hat{y}_k &= C_d\hat{x}_k \end{aligned} \tag{43}$$

In which $\hat{\mathbf{x}}_\mathbf{k}$ = state vector expectations at k, $\hat{\mathbf{y}}_\mathbf{k}$ = Output of the system expectations at k, and K = Kalman gain. As well as the last methods, a prediction error vector $\boldsymbol{\varepsilon}$ and prediction error matrix E are defined, as is shown in Eqs 44 and 45.

$$\varepsilon(k,\vartheta)=y_k-\hat{y}_k, E(\vartheta)=\begin{bmatrix}\varepsilon(1,\vartheta)\\ \varepsilon(2,\vartheta)\\ \vdots\\ \varepsilon(N,\vartheta)\end{bmatrix}_{N_{xny}} \tag{44}$$

$$J(\vartheta)=\det\left\{\frac{1}{N}\left[E(\vartheta)^{T}E(\vartheta)\right]\right\}\rightarrow\min \tag{45}$$

Then, the Gauss-Newton method is used to search for the best parameter, as is described in the process represented by Eqs 46 to 48.

$$\vartheta^{k+1}=\vartheta^{k}+\alpha\Delta\vartheta^{k} \tag{46}$$

$$\Delta\vartheta^{k}=\left[\Phi^{T}W\Phi\right]^{-1}\Phi^{T}(y-\hat{y}) \tag{47}$$

$$\Phi = \begin{bmatrix} \frac{\partial \hat{y}_1}{\partial \vartheta_1} & \cdots & \frac{\partial \hat{y}_1}{\partial \vartheta_d} \\ \vdots & \ddots & \vdots \\ \frac{\partial \hat{y}_N}{\partial \vartheta_1} & \cdots & \frac{\partial \hat{y}_N}{\partial \vartheta_d} \end{bmatrix} \tag{48}$$

where W=weighting matrix or the inverse of prediction-error covariance matrix.

A LINEAR ANALYSIS APPROACH TO THE ANALUCIA BUILDING

Several records have been obtained in the Andalucia Buildings. The largest seismic event recorded is the February 27th, 2010 Chile earthquake. Despite the M_w 8.8 magnitude, the earthquake did not cause any damage to the isolated structure or its contents. Figures 10 and 11 show the recorded data. The strong motion phase lasts about 60 seconds. The peak accelerations at ground level were 0.30g and 0.18g in the horizontal and vertical directions, respectively. The maximum horizontal acceleration recorded within the isolated building was 0.22g. In the vertical direction amplification was observed since the maximum acceleration recorded at the top building level was 0.29g.

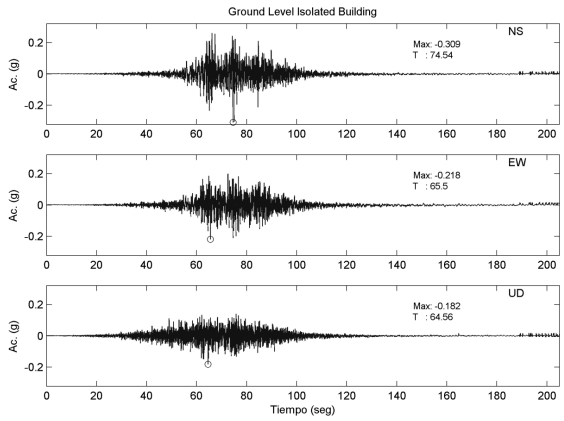

Figure 10. Base acceleration records Andalucia building

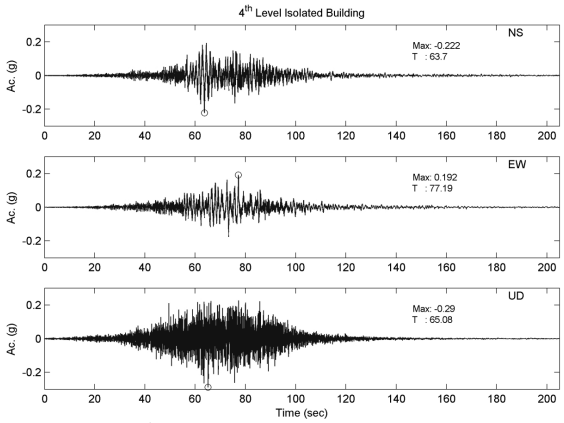

Figure 11. 4th level acceleration records Andalucia building

The window MOESP technique was used to identify the modal properties of the structure as an approach to the system identification. The results of the other non-linear identification method are an ongoing research and their are not presented in this paper. The MOESP identification process is a linear identification method. To considerer capture nonlinear behavior the records were divided in consecutive time windows. Three time windows for the analysis were selected, each one 8 sec long. This time intervals are from 30 to 38 sec, 70 to 78 sec, and 110 to 118 sec, respectively. This procedure allows the identification of variations of structure's modal properties between time windows selected, considering different accelerations amplitudes situations. Shorter time window did not produce better results. Figure 12 shows the high non-linearity of the dynamic system when the acceleration amplitude increase with the earthquake demand, this is, when the time frame of 70 to 78 sec is analyzed. This shows the difficulty when the identification system process when linear properties are considered in the analysis, even in a time frame processing of the data. The figure clearly shows the variations in poles definition, and a high variability even with a short time frame.

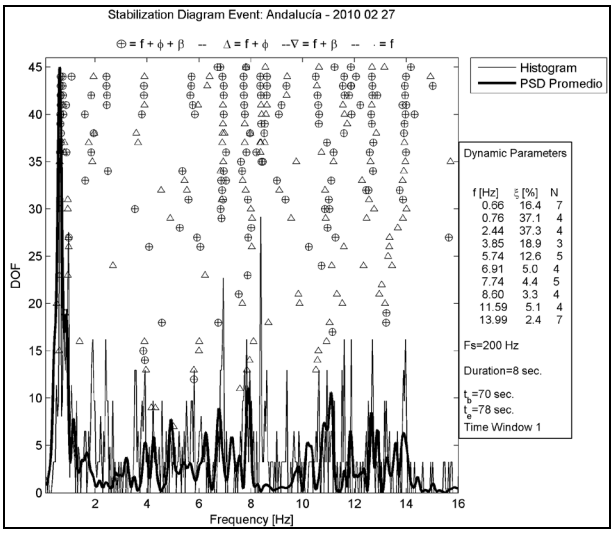

Figure 12. Stabilization Diagram. 70-78 sec Frame of Event

Table 2 shows the variations of the modal frequencies and damping ratios identified, for each time window of analysis. In Table 2 it can be observed that the first modal frequencies strongly depend on the amplitude of vibration. For example, the natural period at the beginning of the strong motion phase is 0.54 sec and gets longer during the second time window, between 70 sec and 78 sec, achieving a value of 1.61 sec. It is important to note that after the earthquake the structure recovered the dynamic properties it had before the earthquake. Modal damping ratios show that, at the same time that the natural frequencies changes damping ratios change too. In the case of the isolated structure, the high damping ratio is associated with the first modes.

Table 2. Identified Modal Properties: February 27th, 2010 Chile Earthquake Record in Andalucia Building

Modal Frequencies (Hz)			Modal Damping Ratios (%)		
30 sec – 38 sec	70 sec – 78	110 sec – 118	30 sec – 38 sec	70 sec – 78 sec	110 sec – 118
1.86	0.62	1.89	2.7	23.7	22.3
2.23	0.78	2.00	12.7	41.8	---
2.99	2.34	3.06	17.9	50.5	19.9
14.11	16.42	18.63	3.9	3.0	2.3

CONCLUSIONS

This case study of the Andalucía isolated building has demonstrated the benefits of using seismic isolation systems. The analysis shows the increase of firsts two periods when the isolation system reacts to the earthquake, as well as an increase of dissipation capacity. After the earthquake the isolators recover their original properties, demonstrating the effectiveness of these devices. The consideration of non-linear properties could give more assurance to the identification methods and will improve the adjustment of the structural response. Moreover, the assessment allows us to conclude that incorporating seismic protection systems in real estate can increase structural safety. The isolation system reduces maximum accelerations and deformations of the structure to 75 and 85%, respectively, with a consequent reduction in the probability of damage and economic losses during exceptionally severe earthquakes. Linear Time Invariant models are appropriate for isolated response only when the nonlinearity is small and time windows are selected for similar response amplitudes.

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