



SEISMIC RESPONSE OF ISOLATED STRUCTURES SUBJECTED TO M_w 8.8 CHILE EARTHQUAKE OF FEBRUARY 27, 2010

Ruben BOROSCHEK¹, Rodrigo RETAMALES² and Antonio AGUILAR³

SUMMARY

The growing interest of investors and stakeholders in protecting building contents and operation beyond the goals of current Chilean 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 seismic responses recorded in an instrumented isolated building during the M_w 8.8 earthquake that struck the central south region of Chile on February 27, 2010. Furthermore, the effects of using seismic isolation devices in a 20-story residential building and in a high tech facility are presented. The main observations, conclusions and implications for design derived from the recorded data are discussed. The critical issues related to seismic isolation design and implementation, along with its effects on the building response and economical costs are described.

INTRODUCTION

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 couple of similar but conventional (fixed base) buildings, as described by Moroni [1]. The base isolated and one of the fixed base buildings were instrumented. This paper presents an analysis of the seismic response of the isolated building recorded during the M_w 8.8 Chile earthquake of February 27, 2010. In addition, this paper describes the basic design requirements of the Chilean Base Isolated Building Design Code and some recent progress on seismic isolation applications.

CURRENT TRENDS ON SEISMIC BASE ISOLATION OF STRUCTURES

The Maule earthquake of February 27, 2010 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, detailed by Miranda [2]. These losses, in addition to the public's perception regarding the earthquake's impact on its properties, has encouraged investors and real estate developers to incorporate seismic protection systems into buildings in order to achieve a performance target that exceeds the standards of current national codes. Seismic isolation is among those seismic protection systems that have experienced the most growth in demand. This type of system has been used extensively, both nationally and internationally, to protect critical and strategic infrastructure, structures of high heritage value, and structures with high-value

¹ University of Chile, Department of Civil Engineering, Chair Structural, Construction and Geotechnical Areas.

² Civil Engineer University of Chile, Leader Seismic Isolation and Energy Dissipation Ruben Boroschek & Associates.

³ Civil Engineer University of Chile, Leader Structural Health Monitoring Ruben Boroschek & Associates.



content, among which include hospitals, datacenters, museums, government buildings and emergency response structures, among others. However, most existing seismic base isolation systems in Chile are found in buildings of relatively low height. Notable Chilean projects include Andalucia Building, developed by Moroni and Sarrazin in 1992; the Marga-Marga Bridge developed by G+V Engineers in 1998; and the Military Hospital developed by Hoehmann Stagno & Associates in 2001. This paper describes the effects of implementing seismic isolation technology in two new special buildings.

National Design Code Characteristics

The Chilean standard for the seismic design isolated buildings, NCh2745.Of2003 (INN 2003) [3], is partially based on the ASCE/SEI 7-98 provisions, adapted to the local seismicity conditions and to more stringent analysis and testing requirements. Currently, the standardization committee's work is aiming at ease and promoting the use of this type of seismic protection technologies. The future version of the code will allow for treating reinforced concrete moment frames as intermediate instead of special moment frames (based on the intrinsic ductility capacity of intermediate moment frames and the low inelastic demands expected on isolated structures), and using a response modification factor of 1.5 for the design of the substructure (based on local design practices and the overstrength resulting from using current design methods), among others.

Torre del Sol Condominium

The first building, shown in Figure 1, corresponds to the Torre del Sol building, a residential building currently under construction in Copiapo. This project was developed by Inmobiliaria Santo Domingo Ltda. and built by Constructora ALCORP S.A. The structural design of the building was developed by Patricio Bonelli & Asociados Ltda. The design of the seismic isolation system, conducted in accordance with the provisions of NCh2745.Of2003, was developed by Ruben Boroschek & Asociados Ltda. (www.rbasoc.cl). The isolation system was manufactured and tested by Dynamic Isolation Systems Inc. (www.dis-inc.com), the world leader in seismic isolation systems that has isolated nearly 400 structures worldwide, including a dozen high-rises up to 33 stories tall. The excellent seismic performance of buildings with seismic isolation technology has been demonstrated during earthquakes in Northridge (EEUU, 1994), Kobe (Japan, 1995), Maule (Chile, 2010), and recently in Tuhoku (Japan, 2011). A detailed description of the performance of seismically isolated buildings affected by the earthquake in Tuhoku, Japan can be found in Saito [4].

The Torre del Sol building, which will span approximately 19,000 m² 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), will be the tallest residential building with seismic isolation in Latin America. 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. The number of isolators required was determined largely by the geometry of the building. In order to provide stability to the structure and to counteract the tension of the isolators, 60x197 cm beams needed to be installed underneath the parking area on the second underground floor. While tensile forces up to 2G (where G is the shear module for a shear strain of 100%) are permitted by the design codes, the test for tensile loading can be conducted in a limited number of

laboratories worldwide. Seismic isolators were arranged at the ends of beams and, generally, at the ends of all bearing walls. As an alternative to using large size beams under the parking area, the use of Cross Linear Bearings (CLB) was also evaluated, but was discarded due to cost. In other cases, CLB's may be the only alternative for implementing seismic base isolation systems in high-rise buildings. Another alternative was tested to withstand incidental tension; tensile cables were used within the isolators by Kasalanati [5] in order to exert pre-compression and to prevent tension. However, this alternative was also discarded due to the length of the cables required to prevent yield during the movement of the isolation interface. In order to reduce costs, the 45 seismic isolators used in the structure are all identical. The maximum compressive force on the isolators (considered for design and for testing) is 820 Tonf, while the tensile force (or minimum compression load) is 0 Tonf.

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. The tests were performed at Dynamic Isolation Systems' laboratories in McCarran, Nevada. 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. A wide variety of seed records were selected in a manner that allows analysis of the different seismic events to which the structure could be exposed during its lifetime. 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. In this case, the critical area of the structure, where the demand of seismic deformations is most concentrated, corresponds to the isolation interface; therefore, it was not necessary to include the confinement according to ACI318. The demand of displacement is accommodated by the deformation of the isolators and not by the rotation of sections of concrete walls. Figure 2 shows comparisons 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.

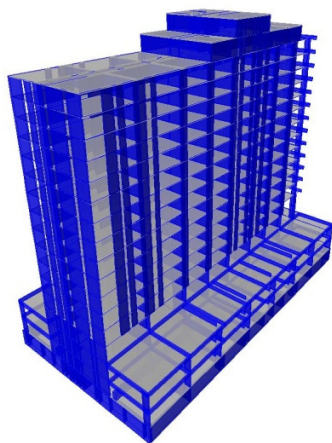


Figure 1. 3-D View analysis model

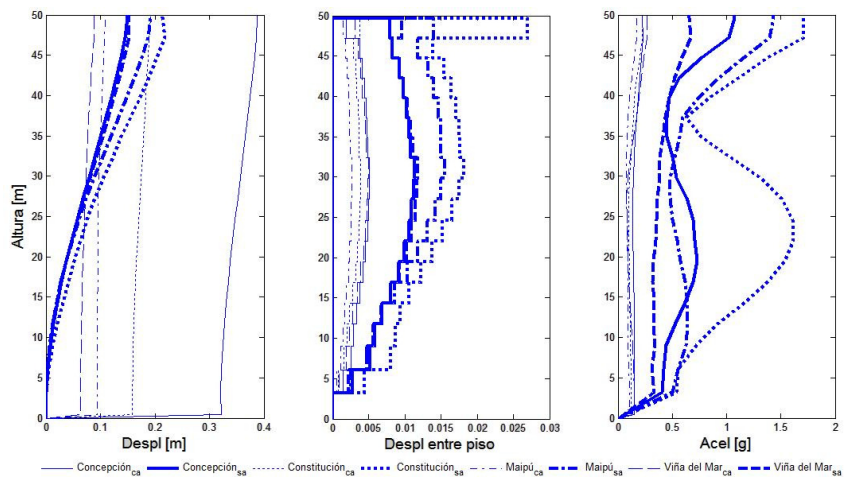


Figure 2. Comparison seismic response Torre del Sol building with and without base isolation

A critical component in the implementation of seismic isolation systems is their initial cost. While their superior structural response during severe seismic events is evident, the decision to invest in protection is

generally difficult. As reference, the cost of each seismic isolator for a project of this size is around US\$8000-US\$9000. Additional costs include those of the V60x197 cm beams required for the stabilization of the structure and the costs of excavations required to accommodate the height of the isolators and the aforementioned beams. Moreover, other considerations include the costs the flexible elements required for basic installations and other distributed systems that cross the isolation joint. However, the initial investment costs are offset undeniably by the reduced likelihood of damage or collapse of the structure. Moreover, the implementation of isolation systems may allow a reduction in the amount of structural walls in a building, given that, as described in previous sections, the elastic interstory drifts in the isolated structure does not exceed the 0.002 of the story height. Similarly, the use of isolation could allow for eliminating boundary elements due to the low demand of relative deformation between the roof and the isolation level. Additionally, this last measure would allow a reduction in the thickness of the walls. These measures, coupled with the minimum-shear design of a structure, could offset the costs of investment in seismic protection.

Claro Chile Data Center

The second interesting and recent application of seismic isolation is the Claro Data Center shown in Figure 4. This building will host one of the largest high-tech facilities in Chile. The total area of the building is approximately 35,000 m², distributed in 2 levels and a mechanical floor. The architectural, structural and isolation system designs, carried out by +Arquitectos and Gatica Jimenez Engineers and Ruben Boroschek and Associates, respectively, considered the stringent requirements of ANSI/TIA-942 [6] 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, also manufactured and tested by Dynamic Isolation Systems, are used under the structure. The seismic responses estimated using the signals recorded during the M_w 8.8 2010 Chile earthquake is shown in Figure 5.

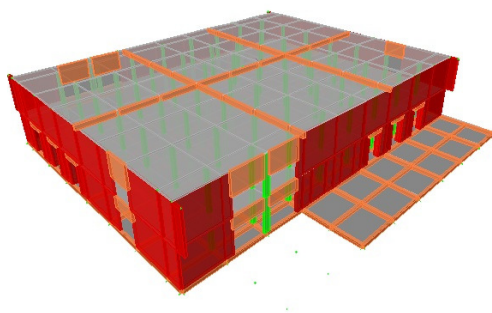


Figure 4. 3-D View Analysis Model
Claro Data Center

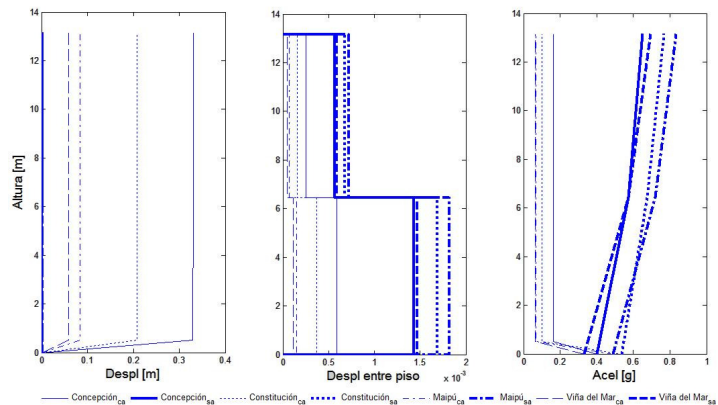


Figure 5. Comparison seismic response Claro Data Center with and without base isolation

RESPONSE OF AN INSTRUMENTED BUILDING DURING THE FEBRUARY 27, 2010 CHILE EARTHQUAKE

The instrumented building, shown in Figure 1, is a 4-story confined masonry structure mounted on top of 8 high damping rubber isolators. The first level considers reinforced concrete shear walls. The building has a typical floor plan of 10x6 m. The weight of the structure is 1630 kN. The typical slab thickness is 10

cm. The structure's roof is a wooden truss. The isolators are 31.5 cm in diameter and 32 cm in height. The thickness of the rubber layers is 6.7 mm. Four of these isolators are located in the corners of the building and two at each long side of the plan. The isolators have a vertical load capacity of 350 kN at 20 cm lateral displacement (100% strain). An extensive test series was carried out at the University of California, Berkeley [1]. The isolated building's design period is 2 seconds.

Instrumentation

The instrumentation consists of an array of triaxial accelerometers with common triggering. Given that each sensor has a 12 bit analog to digital converter, the recording system 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. Sampling frequency is 200 samples per second. Measures directions of accelerometer channels are shown in Table 1.

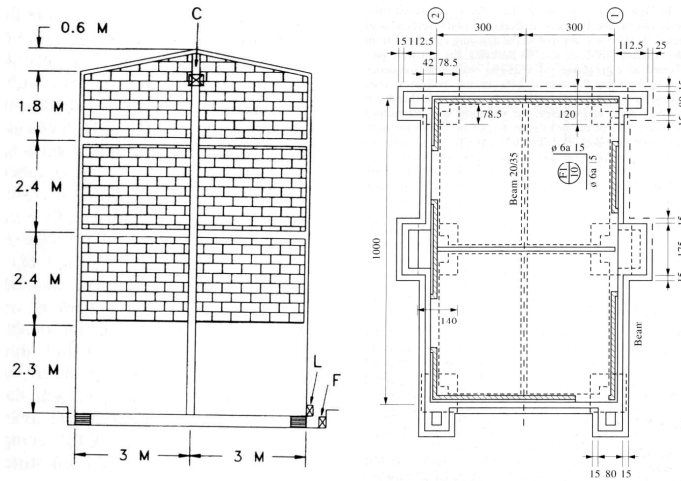


Figure 6. Andalusia Community Building. Global dimensions and isolators layout.

Table 1. Sensors direction

Channel	Sensor	Direction	Level
1	F	EW	Ground
2		NS	
3		UD	
4	C	EW	4 th
5		NS	
6		UD	

Method of Analysis

To identify the dynamic properties of the building an equivalent parametric viscoelastic model is used and adjusted using the MOESP (Multivariable Output-Error State Space) system identification method, developed by Yoshimoto [7]. In this method the state space equilibrium and observation equations are given by:

$$\{z_{k+1}\} = [A]\{z_k\} + [B]\{u_k\} \quad (1)$$

$$\{y_k\} = [C]\{z_k\} + [D]\{u_k\} \quad (2)$$

Considering that every step $\{z_k\}$ can be expressed as a function of previous values, the relation with $\{y_k\}$ can be written as:

$$\begin{Bmatrix} \{y_k\} \\ \{y_{k+1}\} \\ \vdots \\ \{y_{k+s-1}\} \end{Bmatrix} = [O_s]\{z_k\} + [T_s] \begin{Bmatrix} \{u_k\} \\ \{u_{k+1}\} \\ \vdots \\ \{u_{k+s-1}\} \end{Bmatrix} \quad (3)$$



where $[O_s]_{ls \times n}$ is the extended observability matrix with order “s” and $[T_s]_{ls \times ms}$ is a Toeplitz matrix. Given that $[O_s]$ depends on $[A]$ it is possible to obtain from this matrix the natural properties of the structure.

Seismic Response of the Structure and Identification Process

Several records have been obtained in this building. 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. A strong aftershock was registered in March of the same year. Both cases are studied in following subsections.

The M_w 8.8 February 27th, 2010 Chilean Earthquake

Figures 7 and 8 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. The high frequency content of the horizontal signals is considerably reduced when the acceleration reaches its peak value. This does not happen for the vertical signal.

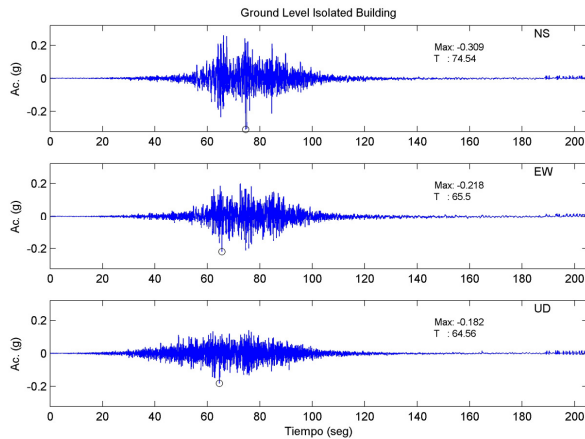


Figure 7. Base acceleration records Andaluca building

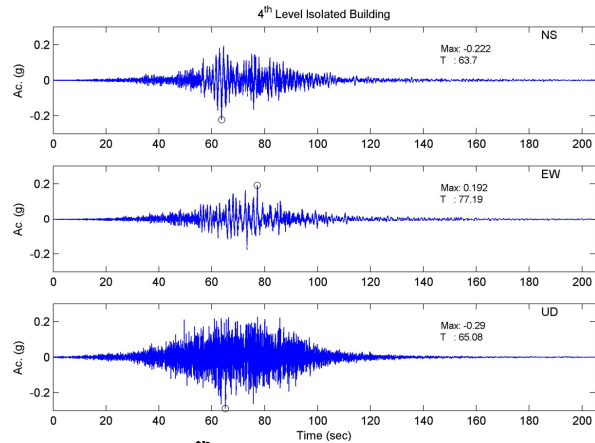


Figure 8. 4th level acceleration records Andaluca building

The identification process was performed considering the time frame from 30 to 118 sec. 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.

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 in order to increase the structure fundamental period, 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 Andalusia Building

Modal Frequencies (Hz)			Modal Damping Ratios (%)		
30 sec – 38	70 sec – 78	110 sec – 118	30 sec – 38	70 sec – 78	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

March 11th, Aftershock 2010 Chile Earthquake

The March 11th was one of the strongest aftershock after main earthquake. It generated peak ground acceleration in horizontal plane of 0.03g for both directions. In vertical direction, maximum ground acceleration registered was of 0.02g. The total duration of the earthquake was around 60 seconds. The identification process was performed considering the time frame from 20 to 70 sec. Three time windows were considered beginning from 20 to 30 sec, continuing from 40 to 50 sec, and finishing from 60 to 70 sec. The strong motion phase extends over 20 seconds from 30 to 50 sec. The aftershock records are shown in Figures 9 and 10.

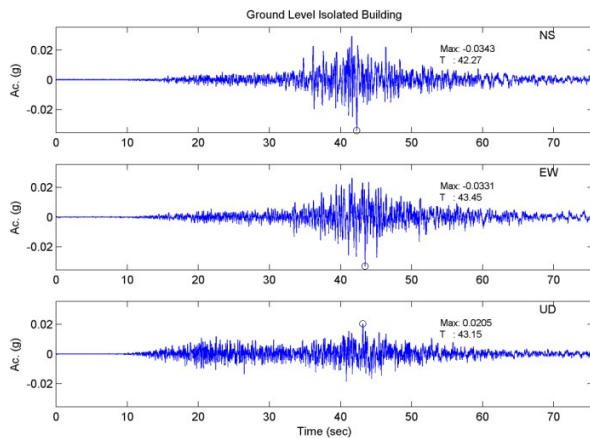


Figure 9. Base acceleration records Andalusia building

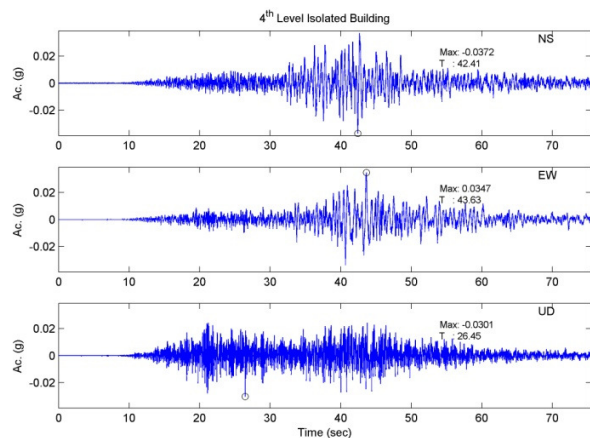


Figure 10. 4th level acceleration records Andalusia building

From the analysis and system identification process, it can be observed that the first two frequencies are related to the isolation system, as is presented in Table 3. This is reflected in the decrease of modal frequencies below 2 Hz in the second time window in comparison to the corresponding two frequencies of the first time window. The highest modal damping ratios are observed in the beginning and ending of the record, the first and third time windows, achieving over 20% of damping. In relation with the main earthquake analysis, the high nonlinearity imposed by the high seismic demand causes that first two mode to vary as a function of amplitude.

It is important to note that the decrease of the modal damping ratio during the strong motion and an increment of the natural periods. This shows that the isolation system effectively acts over the structural system increasing the flexibility while demand is the highest. In this case the linear equivalent model is not able to separate the two first modes. So values present the contribution of the first two translational modes. An appropriate nonlinear elastic model is now been work as part of the identification algorithm to be able to capture the high nonlinearity observed in isolated systems.



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Table 3. Identified Modal Properties: March 11th, 2010 Chile Earthquake Aftershock Record in Andaluca Building

Modal Frequencies (Hz)			Modal Damping Ratios (%)		
20 sec – 30	40 sec – 50	60 sec – 70	20 sec – 30	40 sec – 50	60 sec – 70
2.34*	1.51	2.07	21.6	16.6	22.6
2.62	1.65	2.12	24.1	14.3	30.9
18.26	14.05	10.54	4.1	17.0	2.1

CONCLUSIONS

This practical application has demonstrated the feasibility of using seismic isolation systems in a 19-floor building, contrary to the belief of professionals and of investors in the impossibility of incorporating seismic isolation in mid-rise buildings. 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. The costs of investing in protection may be partly offset by the potential cost reductions in the structure, by reducing design requirements for minimum and maximum shear and by eliminating the containment requirements for wall edges.

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