Seismic Base Isolation of the Nunoa Capital Building, the Tallest Base Isolated Residential Building in the Americas

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ABSTRACT

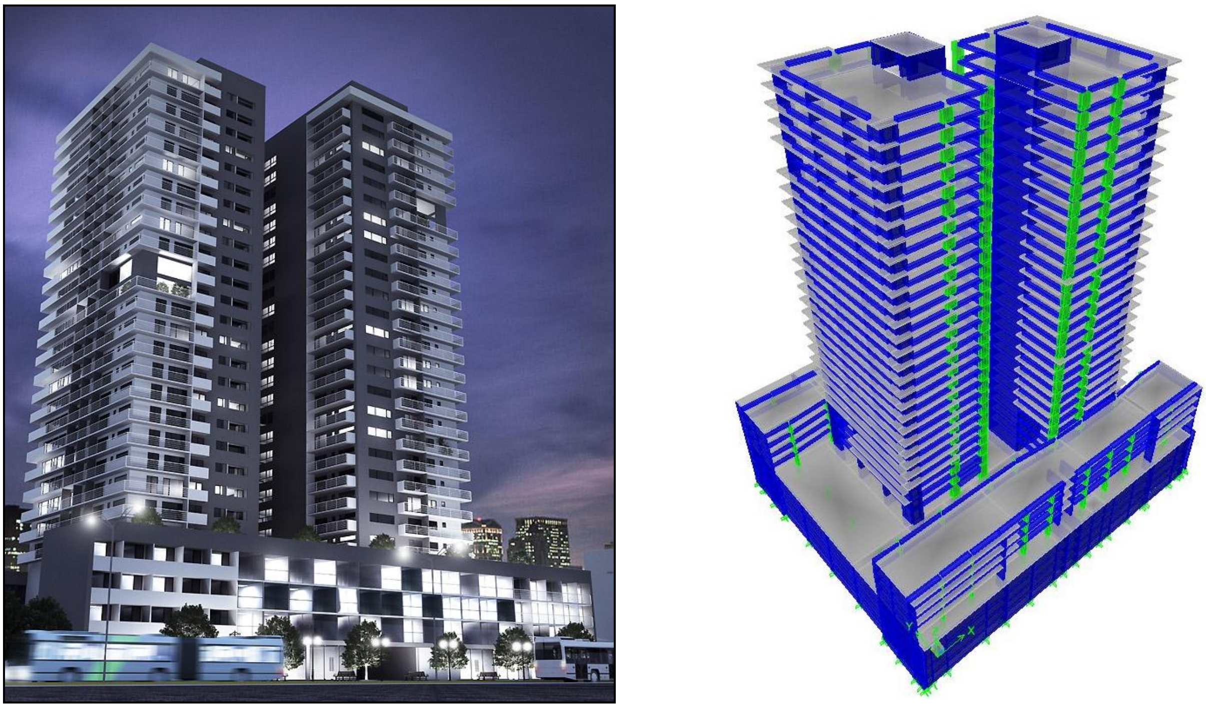
The 2010 Mw8.8 Maule Chile earthquake caused more than 30 billion dollars in direct losses. The indirect losses remained unknown, but it is estimated they exceeded by far the direct losses. In response to the extensive nonstructural damage in residential facilities and the generalized business operation disruption, investors and stakeholders triggered an increased demand for the use of seismic protection technologies such as seismic base isolation and energy dissipation systems. This paper describes the selection process for a seismic protection system for the Nunoa Capital Building towers, from Armas Enterprises, the tallest isolated residential building in the Americas. A comparison of the seismic performance of these 33-story (42,600 m^2) towers when protected with viscous walls and a lead rubber bearing base isolation system is presented. The challenges for the designing engineering team and the main design criteria that resulted in the use of 24 large diameter seismic isolators are presented. It is shown that the use of base isolation in this building caused the base overturning moment to increase in comparison to an equal base shear building designed without isolation. Moreover, the requirement of the Chilean isolation code that requests considering a minimum design base shear equal to 5% of the seismic weight, led in this case to design forces almost two times greater than the elastic seismic demand. This code requirement imposed the structural design team two enormous additional challenges: to limit the superstructure interstory drift below 0.25% (code requirement), and to avoid tension and excessive compression forces in the isolation devices, for the design and maximum

considered earthquakes, respectively. Interstory drift limit was achieved by using an outrigger system in a non-optimal location due to architectural constraints. Tension forces on the isolators were avoided by interconnecting the slabs of the two towers at the underground levels and at the foundation mat. The use of seismic isolation in this building prompted the use of seismic isolation technologies in high rise structures in Chile.

Keywords: *base isolation, tall buildings, outrigger, seismic hazard*

1 INTRODUCTION

The Nunoa Capital Building (Figure 1), with a total constructed surface area of 42,600m^2, consists of two identical towers, both rising 29 floors (75m) above ground level and both intended for residential use. Around the towers, there are smaller 4 story buildings destined for offices. All the buildings share a common underground level, consisting of four floors. The seismic-resistant system of the building consists of an eccentric reinforced concrete core, as well as a perimeter frame and L-shaped walls in the corners. The floor system corresponds to post-tensioned slabs. A common seismic isolation system has been implemented for the two towers, which are connected at the underground level, and rest on top of a 2 m thick slab resting above 24 natural rubber isolators, 16 of which correspond to lead rubber bearings. Under the isolators, footings connected with beams were placed. Between the isolated towers and the peripheral structures, a 50 cm isolation gap is considered, significantly larger than the requirement of the Chilean isolation code, NCh2745.Of2003 [1], with the objective of minimizing the probability of impact between the isolated structure and the adjacent structures (Figure 2).

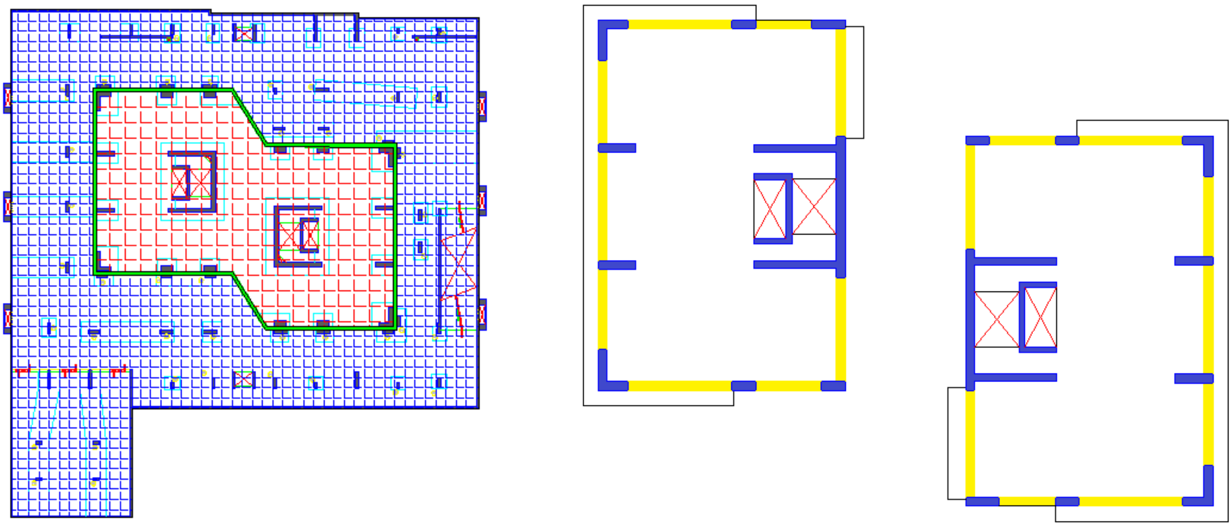


a) Isometric View (Courtesy Armas Enterprises) *b) Analysis Model*

Figure 1. Nunoa Capital Building

For the Nunoa Capital Building, the use of various energy dissipation devices was evaluated, amongst which were: viscous dampers, viscous walls, viscoelastic walls, and tuned mass dampers in combination with viscous dampers. The use of viscous dampers was discarded during the first stages of the project, due to the need to intervene the facades. For the same reason, the use of tuned mass dampers was also discarded, as they would only be used in combination with viscous dampers distributed along height. Consequently, in one of the first stages of the project, the use of viscous walls in nonstructural partitions was evaluated, as well as viscous walls coupled with the walls of the elevator shaft, as illustrated schematically in Figure 3. In addition, the feasibility of using a seismic isolation system in two possible locations was evaluated: below each tower on the first floor, and at the base of the entire structure below the last underground level.

Figure 4 shows a comparison of the seismic responses obtained for the four analyzed cases. This preliminary comparison was completed considering historical earthquake records that are compatible with the spectrum of the Chilean isolation code.

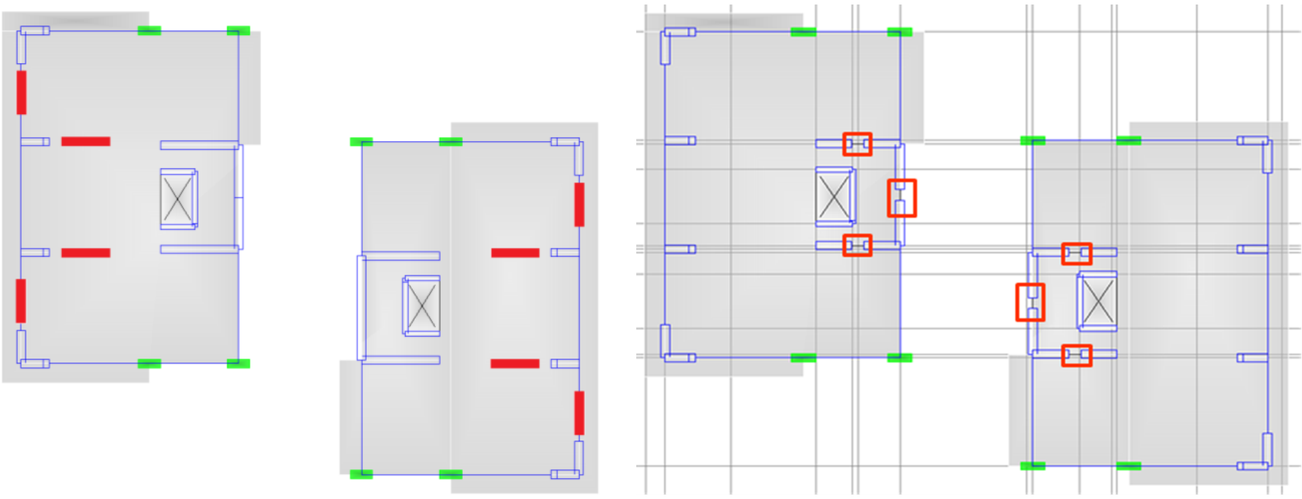


a) Fixed base underground (in blue), seismically isolated sector (in red), and isolation gap (in green)

b) Typical layout isolated towers

Figure 2. Typical Floor Plan

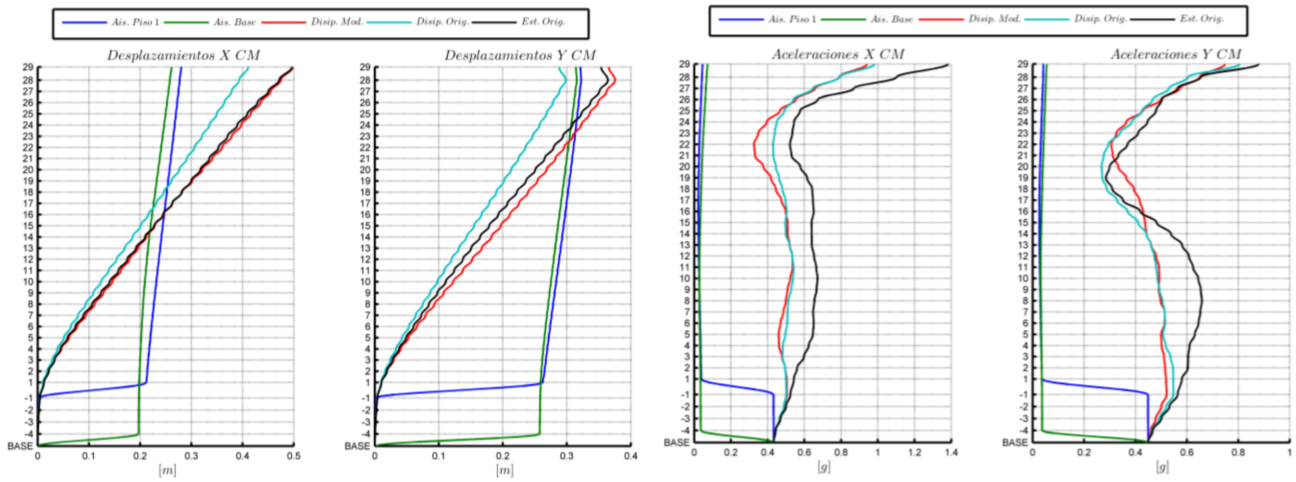
The preliminary analyses completed indicated that the costs of implementing the energy dissipators in partitions and coupling walls fluctuated between 2 and 2.5 million dollars, while the cost of the isolation systems required for isolating the towers individually or together fluctuated between 0.7 and 1.2 million dollars. In light of the economical and technical analyses performed, it was quite evident that the most appropriate alternative for the structure was the use of seismic isolation systems.



a) Viscous walls in partitions (in red)

b) Viscous walls coupling concrete walls (in red boxes)

Figure 3. Energy dissipation alternatives



a) Displacements *b) Absolute accelerations*

Figure 4. Comparison elastic seismic response conventional structure and structure with base isolation and energy dissipators

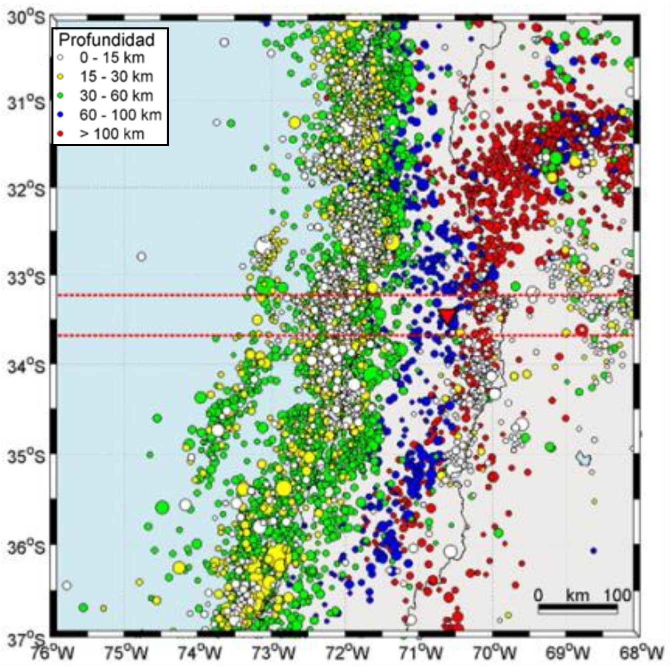
Amongst the existing seismic isolation systems, the team of specialists decided to use a combination of isolators composed by natural rubber bearings and lead rubber bearings, because of the stability and predictability of its properties. Other isolation systems, such as frictional isolators, were discarded due to the difficulties they present for predicting and modeling the variation of the friction in the isolator during seismic movements, given the vertical effects of earthquakes. Similarly, the use of high damping rubber isolators was dismissed for the low level of damping they provide, and for the difficulties in predicting their behavior during severe strong motions.

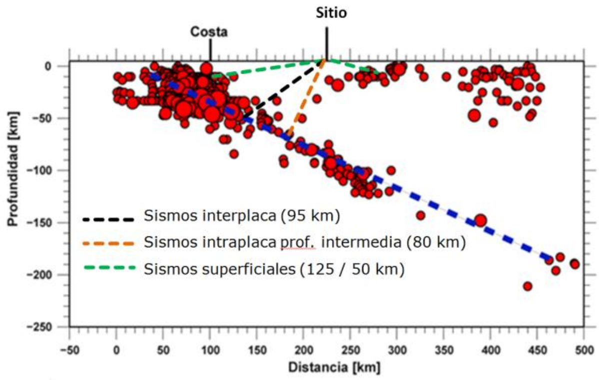
Due to the period of the fixed base structure, around 2 seconds, it was estimated that the period of the isolated structure would be around 5 to 6 seconds, and therefore, in accordance with the current Chilean isolation code, a site specific seismic hazard study was required.

2 SEISMIC HAZARD EVALUATION

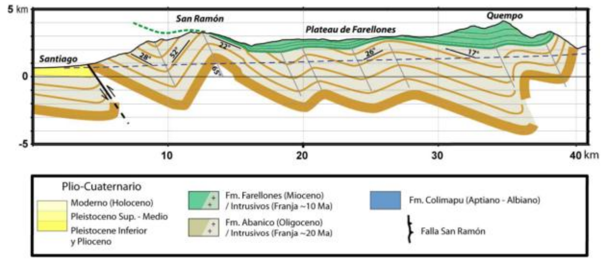
Chile is one of the countries with the highest seismic activity in the world, and the subduction process of the Nazca Plate below the South American continent is the primary cause. The aforementioned process gives origin to different types of earthquakes, which are classified under the following groups: inter-plate earthquakes (occurring in the contact zone between the Nazca Plate and South American Plate), intermediate depth and large depth intra-plate earthquakes (occurring within the interior of the Nazca Plate), and shallow intra-plate earthquakes (occurring in the continental crust of the South American Plate). From the late 16th century to the present, there has been a high-magnitude earthquake every 8 to 10 years on average, throughout the Chilean territory.

Taking into account the particular characteristics of this project, such as the structuring, height, rigidity, incorporation of base isolation, and also the requirements present in the national seismic design standards, it was necessary to perform a study based on the seismic hazards for the project site. The objective of this study was to establish a seismic design spectrum specific to the conditions of the site, according to its geographical location, and geotechnical and geomorphic characteristics. To characterize the different seismic sources, earthquakes with a magnitude greater than 4 ($M \geq 4$) and an epicenter within a radius of less than 300 km from the site (Figure 5a) were considered. This allows the estimation of distances from the site to different seismogenic sources (Figure 5b). On the other hand, the possible influence of shallow sources associated with the San Ramon Fault, located approximately 8.5 km from the site, was evaluated (Figure 5c),. Without having much certainty about its activity, accelerations were estimated as a benchmark for analyzing the possible influence of this source in the calculation of the overall seismic hazard in the site of interest.





b) Hypocentral distribution of the earthquakes in EW profile, with a width of 50 km, centered on the site



a) Epicentral distribution of the earthquakes around the studied site between the years 1973 and 2012. The location of the site is indicated with a red triangle

c) Trace of the San Ramon Fault and schematic of geological-structural profiles of the area (modified from Armijo et al. 2010)

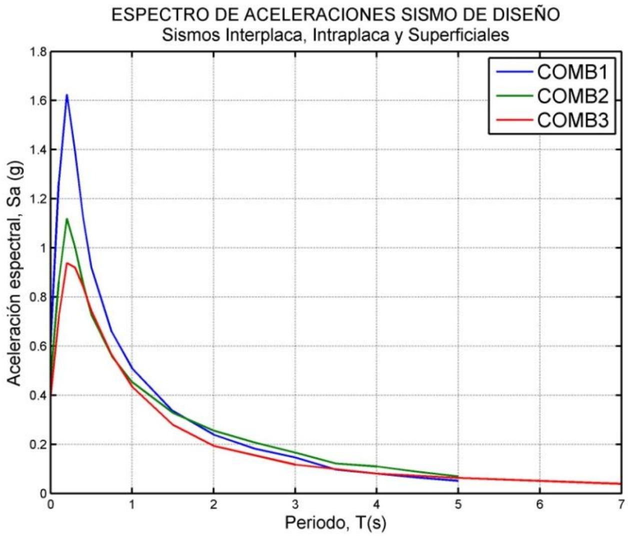
Figure 5. Seismic Hazard Study

Figure 6a shows the resulting spectral accelerations associated to the different seismic sources that could affect the site. In Figure 6b, a comparison of the resulting acceleration spectra for two distinct earthquakes occurring at the San Ramon Fault is presented.

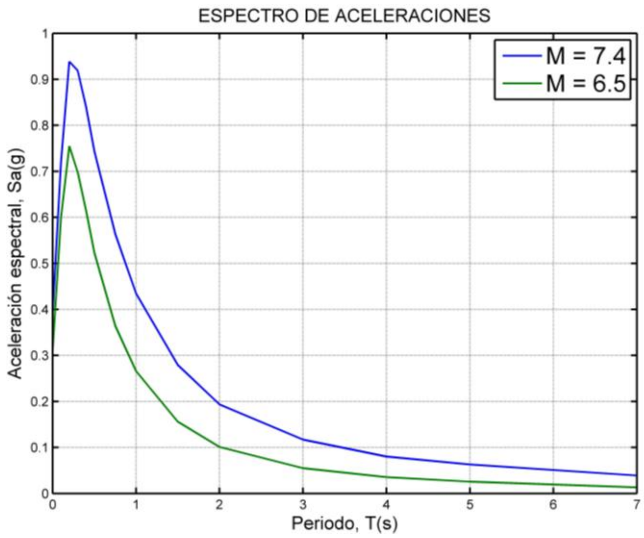
From the results of the seismic hazard study, the design elastic acceleration spectrum was determined, which has validity for structures with periods up to seven seconds. Additionally, and in accordance with the standards for seismic isolation design, seven pairs of artificial acceleration records, compatible with the proposed design spectrum, were generated.

3 SEISMIC ISOLATION SYSTEM

The seismic isolators of the building are located under the ends of the walls and under the columns. The project will use 24 natural rubber isolators, manufactured by Dynamic Isolation Systems (www.dis-inc.com). The devices are composed by rubber with a strain capacity over 600%, whose behavior is not altered in the long term. Of the 24 devices, 16 of them contain a lead core (LRB), while the 8 remaining do not have a lead core (RB). The isolators of a larger diameter, Type C (RB), 155 cm in diameter, have a capacity to support loads above 40,000 kN and will be located in the most loaded points, under the ends of the walls of the elevator shafts. As for the LRB type isolators, these are distributed in 8 Type A isolators, 115 cm in diameter, with a load capacity above 20,000 kN, and 8 Type B isolators, 135 cm in diameter, with a load capacity over 30,000 kN. The Type B isolators, the stiffest ones, will be located in the farthest points of the structural plan in order to control the torsion of the structure. The seismic isolation system allows for achieving an effective vibration period near to 5 seconds, and an effective damping in the order of 20%. Based on the results of the seismic analyses, reductions of shears, absolute accelerations and interstory drifts in the order of 70% to 80% are possible in comparison to its fixed-base simile.



a) Spectral acceleration that considers inter-plate as well as intermediate and shallow depth intra-plate seismic sources



b) Comparison of the spectral acceleration for the San Ramon Fault, considering magnitudes of M=6.5 and M=7.4

Figure 6. Site spectra

4 STRUCTURAL ANALYSIS

4.1 Analysis Methods and Models

Chilean isolation code allows for using either linear or nonlinear models to analyse and design the structure and the seismic isolation system. It also specifies two levels of seismic demand: the design basis earthquake and the maximum considered earthquake. The design basis earthquake (DBE) is used to analyse and design the whole structure except the seismic isolation system. On the other hand, the maximum considered earthquake (MCE) is used to analyse and design the seismic isolation system.

In the Nunoa Capital Building, both types of analyses were used. First, a linear analysis, specifically a response spectrum analysis, was performed. In this analysis, the isolators were modelled with their equivalent secant stiffness at the design displacement level. A site specific design spectrum obtained from the Seismic Hazard Evaluation was used. This spectrum represents the demand for the DBE level and was reduced by a factor of 1.8 for periods larger than 3.0 seconds to consider the extra damping provided by the seismic isolation system. This initial estimated factor is later verified through nonlinear analysis. This linear anaylisis was used to design the superstructure, the foundations and to predesign the seismic isolation system.

Then a nonlinear time history analysis was carried out. In this analysis the isolators were modelled with a nonlinear biaxial hysteretic model that has coupled plasticity properties for the two shear deformations [2]. This causes the extra damping due to the seismic isolation system to be directly considered and provides an estimation of the effective damping of the system. This analysis was used primarly to validate the predesign of the seismic isolation system.

Seven pairs of ground acceleration records obtained from the Seismic Hazard Evaluation were used. These ground acceleration records represent the demand for the MCE level. As allowed by the Chilean isolation code, all the parameters of interest such as displacements or axial loads on the isolators were obtained as the average of the peak response for each individual pair of records.

As explained later, the superstructure is expected to remain essentialy elastic for both the DBE and the MCE, which allows for modelling the building by using the linear elastic elements available in ETABS [2].

4.2 Global Behavior

The first eight vibration periods of the structure, for the design earthquake level, are shown in Table 1.The X direction refers to the long side and the Y direction to the short side of the structure.

Table 1 – Periods of the Structure for the design earthquake level

Mode -	Period (s)	X Mass Ratio (%)	Y MassRatio (%)	Rot. Mass Ratio (%)
1	5.344	98.25	0.56	0.04
2	5.260	0.56	98.61	0.00
3	4.826	0.04	0.00	98.65
4	2.128	0.00	0.00	0.05
5	1.554	0.03	0.53	0.00
6	1.414	1.10	0.01	0.01
7	1.397	0.01	0.00	0.68
8	1.129	0.00	0.28	0.00

The Chilean isolation code allows for considering the design base shear as the elastic demand divided by a response modification factor R = 2, but no less than a minimum percentage of the seismic weight that depends on the seismic zone, specifically 5% for this project.

Usually, the minimum base shear is between the elastic and the reduced demand, which causes the effective reduction factor R* (the adjusted R factor to achieve the minimum base shear) to be between 1 and 2. Nevertheless, in this project, due to the long periods achieved, the elastic demand is less than the minimum. This causes the superstructure to be designed for a seismic demand greater than the elastic one, which is equivalent to have an effective reduction factor R* less than 1. The elastic demands, effective reduction factors R* and final design shear for each direction, among other key design parameters, are shown in Table 2.

Table 2 – Elastic and Final Design Base Shear

	X Direction	Y Direction
Elastic Base Shear (kN)	13,368	13,734
Design Base Shear R=2 (kN)	6,684	6,867
Minimum Design Base Shear (kN)	20,346	20,346
Effective Reduction Factor R*	0.657	0.675
Final Design Base Shear (kN)	20,346	20,346

The base overturning moments in the isolated structure (for the final base design shear) in the X direction is amplified when compared to the fixed base structure solution. This shows an important influence of the higher modes in the isolated structure (table 3).

Table 3 – Contribution of each vibration mode to the base overturning moment

	Mode	Base Shear	Base Overturning Moment	% of Base Overturning Moment
	-	kN	kN-m	%
Isolated Structure	ALL MODES (CQC)	20,346	1,043,967	100.0
	MODE 1	19,955	837,546	80.2
	MODE 2	117	4,861	0.47
	MODE 3	11	376	0.04
	MODE 4	0	0	0.00
	MODE 5	61	10,334	0.99
	MODE 6	2,906	607,879	58.2
	MODE 7	30	6,257	0.60
	MODE 8	1	291	0.03
Fixed Base Structure	ALL MODES (CQC)	20,346	868,339	100.0
	MODE 1	13,950	835,313	96.2
	MODE 2	395	23,367	2.69
	MODE 3	43	2,529	0.29
	MODE 4	4	236	0.03
	MODE 5	13	587	0.07
	MODE 6	1	42	0.00
	MODE 7	8	74	0.01
	MODE 8	43	206	0.02

5 DESIGN CHALLENGES

As explained in a previous section, for this particular project there were two aspects that are not common for low rise isolated structures: a design base shear greater than the elastic demand, and an increased design base overturning moment. Those two aspects imposed two big challenges for the structure design: to control the interstory drift and to avoid tension forces on the isolators.

Regarding the story drifts, the Chilean isolation code limits it to be less than 0.25% for the DBE level. This is different to other codes such as the ASCE7 [3] because it is directly measured from the reduced (by the R* factor) DBE spectrum instead of multiplying the elastic response by Cd/I.

In order to control the story drifts, it was necessary to stiffen the building, which was a challenge because a tight architectural plan with almost no space for additional or thicker structure. This was achieved by using 2 outriggers per tower in the upper (mechanical) floor. Despite of this location not being the optimal, the outrigger’s benefits were enough to stiffen the structure and reduce the interstory drifts right under the code limits. This can be seen for the X direction in the Figure 7.

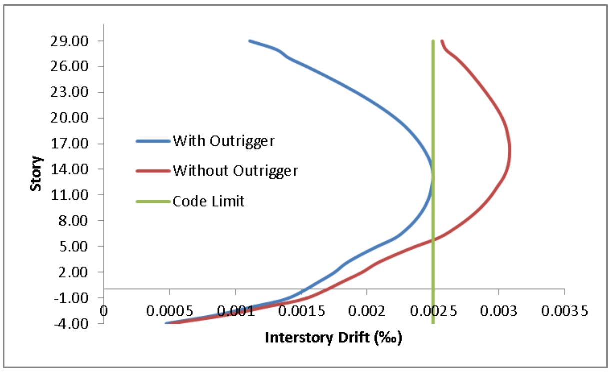


Figure 7 - Interstory Drift with and without the Outrigger

The second challenge was to avoid tension forces on the isolator. Although the isolators can resist small tension forces, the design team decided to eliminate tension forces on the isolators for the MCE level. This was achieved by connecting the slabs of the two towers at the underground levels and by the use of a 2m thick slab resting directly above the isolators. This thick slab stiffens the isolated interface so that the external isolator are coupled with the internal ones, leading to a greater lever arm which resists the overturning moment and reduces the magnitude of the seismic compression/tension forces. Final compression forces on the isolators for the MCE level are shown in Figure 8. To enhance the performance of this thick coupling slab, imbedded beams with close spaced stirrup and ties were designed.

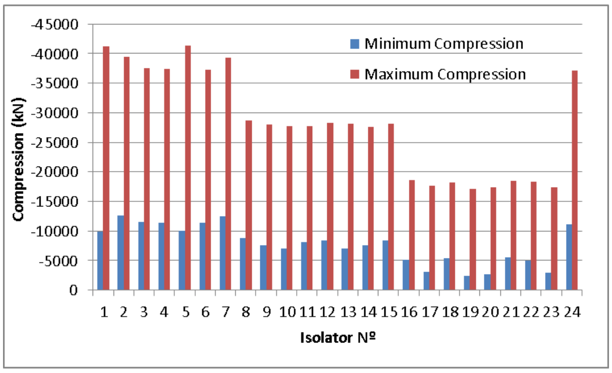


Figure 8 - Compression Forces on each Isolator for the MCE

6 CONCLUSIONS

The developed work demonstrates the technical feasibility of implementing seismic isolation systems in high rise buildings. The main design challenges included the use of large diameter isolators to resist large compression forces, the difficulties to achieve simultaneously a high vertical stiffness and lateral flexibility, avoiding tension forces on the isolators and controlling the interstory drifts to a level below the code limits. The feasibility of using a seismic isolation system in this 33 story structure is associated to the good geotechnical conditions at the site, the use of a relatively stiff superstructure, and the use of an outrigger system. The sizes of the structural elements were determined by the interstory drifts limits rather than the design forces. It has been proven that the use of seismic isolation systems can considerably reduce the seismic demands, even in tall buildings such as the Nunoa Capital Building.

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