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# Seismic Base Isolation of the Nunoa Capital Building

## Nunoa首都大厦的隔震设计

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## Abstract

The 2010 Mw 8.8 Maule Chile earthquake caused more than 30 billion US dollars in direct losses. The indirect losses remain unknown, but it is estimated that they far exceed the direct losses. Although the standard's seismic philosophy goal for this earthquake, collapse prevention, was largely achieved (less than 0.05% of the buildings collapsed), an important percentage of the buildings suffered disruptions and required extensive and unlikely repairs (and in a few cases demolition) contrary to the principles of sustainable development. In response to this, investors 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, the tallest isolated residential building in the Americas. A comparison of the seismic performance of this 33-story building (42,600 m² built) when protected with viscous walls and a lead rubber bearing base isolation system is presented.



**Keywords: Tall Building Seismic Isolation, Outriggers, Passive Dampers**



## 摘要

2010 年智利8.8 级Maule地震造成的直接损失超过 300 亿美元。间接损失仍然未知，但据估计远远超过直接损失。虽然这次地震的标准抗震理念的目标，预防倒塌在很大程度上已经达到 (不到 0.05%的建筑物倒塌)， 但是一些重要的建筑物遭受破坏，需要大量和不大可能完成的修复 (很少情况需要拆除)， 违反可持续发展的原则。为此，投资方加大了对抗震技术的使用要求，比如基础隔震，耗能系统等。本文介绍了美洲最高隔震住宅楼Nunoa Capital Building的隔震系统的选择过程。这栋33层 建筑 (建筑面积42,600 平方米) 分别就采用粘性墙保护和橡胶支座基础隔震系统保护的抗震性能做了比较。



关键词: 高层建筑隔震，外伸桁架， 被动阻尼器


## Introduction

The Nunoa Capital Building (see Figure 1), with a total constructed area of 42,600 m², consists of two identical towers, both rising 29 floors (75 m) 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 is comprised of 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 supported by 24 natural rubber isolators, 16 of which are lead rubber bearings. The isolators are on footings connected by beams. The isolated towers and the peripheral structures are separated by a 50 cm isolation gap which is significantly

## 介绍

Nunoa首都大厦 (见图 1)， 总建筑面积42,600 平方米， 包括两个相同的塔楼， 都是地上29 层 (75 米)的住宅。在塔楼附近，有栋较小4层办公建筑。所有建筑共用地下室。建筑的抗震性能体系包括有一个偏心钢筋混凝土核心筒和外围框架以及角落的L 形墙。楼板采用后张预应力板。这两座塔楼都使用了共同的隔震系统，它们在地下室部分相连接，坐落在由 24个叠层橡胶隔震支座支撑2米厚楼板上，其中的16个是铅橡胶支座。隔震器在底部由横梁连接。隔震塔楼和外围结构之间有50 厘米的间距，这间距远大于智利隔震规范NCh2745. Of2003 [1] 的要求。此举的目的是尽量减少隔震结构与相邻结构之间产生相互影响的可能性 (见图 2) 。

对Nunoa 首都大厦采用各种耗能装置的情况进行了分析比较。这些包括粘性阻尼器、 粘性阻尼墙、 粘弹性墙壁以及调质阻尼器与粘滞阻尼器的组合体。项目第一阶段，由于对外立面的建筑影响，粘性阻

larger than the requirement of the Chilean isolation code, NCh2745. of 2003 [1]. The objective of this was to minimize the probability of impact between the isolated structure and the adjacent structures (see Figure 2).

For the Nunoa Capital Building, the use of various energy dissipation devices was evaluated. These included viscous dampers, viscous wall dampers, viscoelastic walls, and tuned mass dampers in combination with viscous dampers. The use of viscous dampers was eliminated during the first stages of the project, due to their architectural effect on the façades. For the same reason, the use of tuned mass dampers was also eliminated, 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. These are shown schematically in Figure 3. In addition, the feasibility of using a seismic isolation system in two configurations was evaluated: below each tower on the first floor and at the base of the entire structure below the lowest 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.

The preliminary analyses completed indicated that the costs of implementing the energy dissipaters in partitions and coupling walls was between 2 and 2.5 million US dollars, while the cost of the isolation systems required for isolating the towers individually or together was between 700,000 and 1.2 million US dollars. In light of the economical and technical analyses performed, it was quite evident that the most appropriate option for the structure was the use of seismic isolation systems.

Amongst the existing seismic isolation systems, the team of specialists decided to use a combination of natural rubber and lead rubber bearings, because of the stability and predictability of their 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.

## Seismic Hazard Evaluation

Chile is one of the countries with the highest seismic activity in the world, primarily due to the subduction process of the Nazca Plate below the South American continent. This subduction process gives rise 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

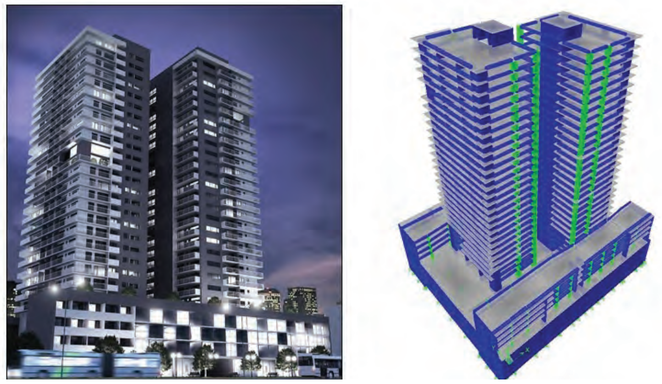

Figure 1. Nunoa Capital Building - Isometric View (left) and Analytical Model (right)
图1. Nunoa Capital Building三维效果图 (左) 和分析模型 (右)

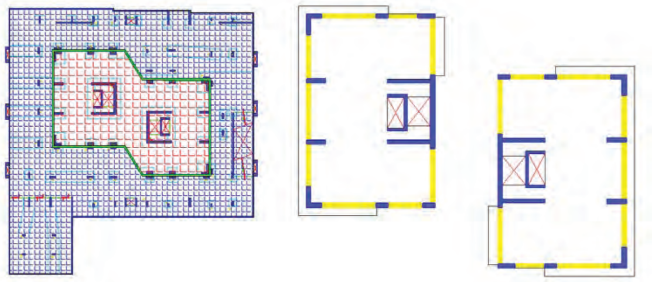

Figure 2. Typical Floor Plan - Fixed base underground (in blue), seismically isolated sector (in red), isolation gap (in green) and typical layout isolated towers (right)
图2. 高层建筑中标准平面图-地下室固定区 (蓝色)，隔震区 (红色)，隔震间隙 (绿色) 和隔震楼典型布局 (右)。

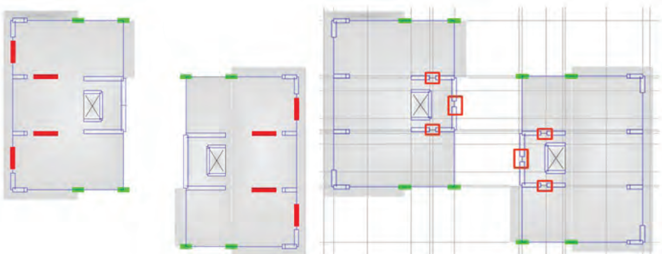

Figure 3. Energy dissipation alternatives. - Viscous walls in partitions (in red) and viscous walls coupling concrete walls (in red boxes)
图3. 耗能方案-间隔墙用粘性墙 (红色) 和粘性耦合混凝土墙。

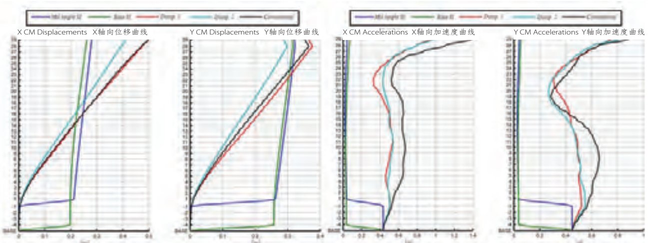

Figure 4 . Comparison of the elastic seismic response of a conventional structure and the structure with base isolation and energy dissipators
图4. 传统结构和基础隔震结构的弹性地震反应及能源耗散对比

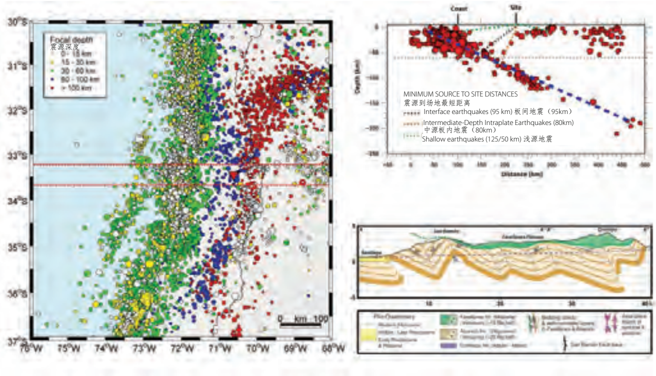

Figure 5. Seismic Hazard Study - Epicentral distribution of the earthquakes around the site between the years 1973 and 2012. The location of the site is indicated with a red triangle (left) - Hypocentral distribution of the earthquakes in EW profile, with a width of 50 km, centered on the site (upper right) Trace of the San Ramon Fault and schematic of geological-structural profiles of the area (bottom right)
图5. 地震危险性研究-1973年到2012年项目场地周围的地震震中分布。场所的位置用一个红三角表示 (左) -在EW方向地震震源分布，50公里的宽度，以项目场地为中心 (右上方) San Ramon 断层走向，以及该区域 (右下角) 的地质结构概况。

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 structure, 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 site specific design spectrum, according to the site's geographical location, geotechnical and geomorphic characteristics. To characterize the different seismic sources, earthquakes with a magnitude greater than 4 (M > 4) and an epicenter within a radius of less than 300 km from the site were considered. This allows the estimation of distances from the site to different seismogenic sources. The possible influence of shallow sources associated with the San Ramon Fault, located approximately 8.5 km from the site, was also evaluated (see Figure 5). Even though there is uncertainty about its activity, accelerations were estimated as a benchmark for analyzing its possible influence on the overall seismic hazard of the site.

Figure 6 shows the spectral accelerations of the different seismic sources that could affect the site and a comparison of the resulting acceleration spectra for two distinct earthquakes occurring at the San Ramon Fault.

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

## Seismic Isolation System

The seismic isolators of the building are located under the ends of the walls and under the columns (see Figure 7). The project will use 24 natural rubber isolators, manufactured by Dynamic Isolation Systems. The devices are made of rubber with a strain capacity over 600%, developed by Dynamic Isolation Systems, whose long term properties are extremely stable. The design process considered an extensive database of experimental results obtained for similar isolators, subjected to comparable vertical loads, and subjected to even larger displacement levels. Of the 24 devices, 16 of them have a lead core

尼器的使用就被淘汰了。。出于相同的原因，调质阻尼器的使用也被淘汰了，因为他们只能用于与沿高度分布的粘滞阻尼器的结合。因此，在项目的第一阶段，非结构性区域粘性墙的使用已经被评估，以及粘性墙与电梯井墙的结合。这些概要如图 3 中所示。此外，也研究了使用隔震系统的两种可行性: 即每座塔楼的第一层和整个建筑物地下室最底层。

图 4 显示的是四个分析案例提供的地震响应的对比。利用和智利隔震规频谱一致的历史地震记录所进行的初步比较已完成，

已完成的初步分析表明在间隔墙和耦合墙安装耗能装置的费用在两百万到两百五十万美元之间，同时塔楼单独或一起所需要的隔震系统的费用在 70 万和 120 万美元之间。根据已完成的经济和技术分析，很明显，对于本项目最适合的选择是隔震系统。

在现有的隔振系统中，由于其稳定性和其属性具有可预测性，专家团队决定使用天然橡胶和铅芯橡胶支座的组合。。其他的隔振系统，如摩擦隔震器，由于在建模预测其在地震运动期间隔震器的摩擦变化具有难度，以及考虑竖向地震影响方面的因素而被放弃了。同样，高阻尼橡胶隔震器因为只能提供低水平阻尼以及在强震动期间难于预测它们的行为也被放弃了。

由于固定基础结构的周期为 2 秒左右，据估计隔震结构的周期大约为 5 到 6 秒，因此，根据当前的智利隔震规范，需要研究特定场地地震危险性。

## 地震危险性分析

智利是世界上地震多发国之一，主要原因是在南美大陆板块下面的纳斯卡板块的潜移过程。这个潜移过程会引起不同类型的地震，分为以下几种: 不同板块地震 (发生在纳斯卡板块和南美板块之间的接触地带)，中等深度和大深度内部板块地震 (发生在纳斯卡板块的内部)，和浅层的内部板块地震 (发生在南美洲板块大陆地壳)。从十六世纪末到现在，在智利境内，平均每 8 至 10 年就有一次高震级的地震。

考虑到这一项目的具体特点，如结构、高度、刚度、基础隔震的结合和目前国家抗震设计标准中的要求，有必要进行基于项目场地的地震危险性研究。这项研究的目的是根据场地的地理位置、地质地貌特来建立一个特定场地的设计谱，为考虑不同震源特点，考虑了地震震级大于 4 (M > 4)以及 震中少于 300 公里的半径范围。这样就可以估算出从项目场地到不同的震源的距离。位于该项目场地大约8.5公里的San Ramon断层可能发生的浅表地震的影响也已被评估 (见图 5) 。即使它的活动存在不确定性，估算的加速度可用作分析项目场地的总体地震危险可能影响的基准。

图 6 显示的是可能影响项目场地的不同震源的频谱加速度以及发生在San Ramon断层的两次截然不同的地震和由此产生的加速频谱的对比。

从地震危险性研究的结果来看，确定了周期最大为7秒的结构设计用弹性加速度频谱。此外，根据隔震设计规范，同时与建议的设计频谱相匹配，生成了七对人工加速度记录。

## 隔震系统

隔震支座设置于墙端部和柱底部。该项目将使用由"动态隔离系统公司"制造的24个天然橡胶隔震支座 (见图7) ，。隔震支座橡胶应变量可达 600%以上，其长期性能非常稳定。设计过程参考了采用类似隔震支座试验结果的大量数据这些试验中所承受的垂直荷载和本项目接近而位移程度更大。这24个隔震支座中，其中 16

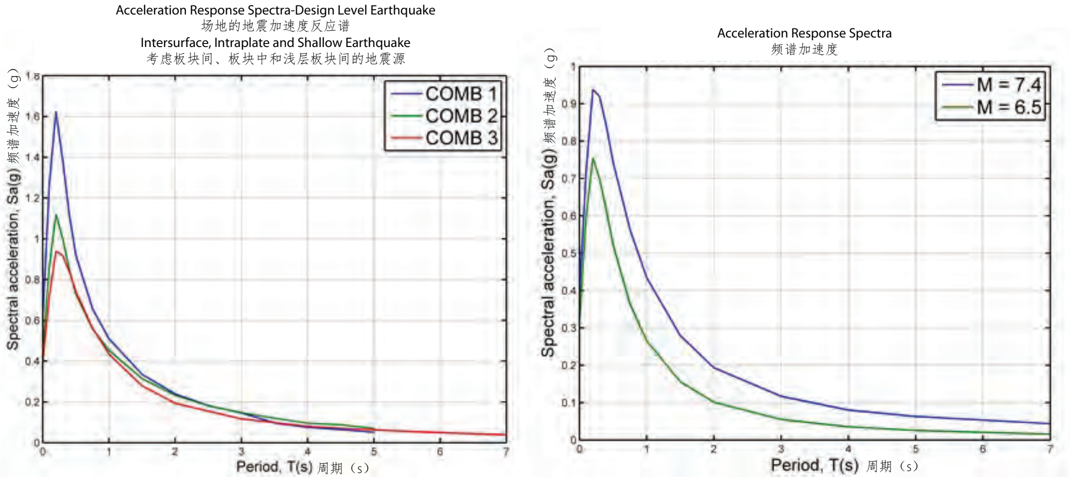


Figure 6 . Site spectra - Spectral acceleration considering inter-plate, intermediate and shallow intra-plate seismic sources (left) and comparison spectral accelerations for San Ramon Fault, considering magnitudes of M=6.5 and M=7.4 (right)
图6. 场地频谱，考虑"板块间", 板块中和浅层板块间地震源的频谱加速度 (左) 以及 San Ramon断层的频谱加速度的比较，考虑震级6.5到7.4 (右)

(LRB), while the 8 remaining do not have a lead core (RB). The largest diameter Type C (RB) isolators are 155 cm in diameter and have a load capacity of more than 40,000 kN and will be located in the most heavily loaded locations under the ends of the walls of the elevator shafts. There are 8 Type A LRB isolators, 115 cm in diameter, with a load capacity of more than 20,000 kN, and 8 Type B isolators, 135 cm in diameter, with a load capacity of more than 30,000 kN. The stiffest isolators are the Type B isolators and they are located at the perimeter to control the torsion of the structure. The seismic isolation system achieves an effective vibration period near to 5 seconds, and about 20% effective damping. Based on the results of the seismic analyses, reductions of shears, absolute accelerations and interstory drifts by 70% to 80% are obtained compared to the fixed base elastic response.

The seismic isolation system was subjected to an extensive test series carried out at Dynamic Isolation Systems' laboratories in McCarran, Nevada, and Eucentre laboratories in Pavia. Prototypes and all production isolators were subjected to an exhaustive test sequence of combined compression loads and shear displacements in order to validate the properties assumed for the design and to verify the stability of the isolators under extreme seismic loads.

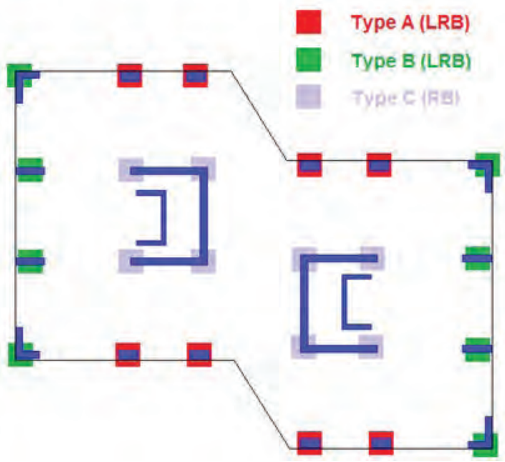


Figure 7 . Isolator Layout
图7. 隔震支座布局

个有铅芯 (LRB)，而其余的 8个没有铅芯 (RB)。最大的 C型 (RB) 隔震支座直径 为155 厘米，可承受 4 万KN的荷载将安置在荷载最大位置的电梯井道墙端部。有 8 个 A型 LRB隔震支座，直径为115 厘米，负载能力超过 20 000 kN 。而 8 个B 型隔震支座直径为135 厘米，负载能力超过 30000 kN。刚度最大的B 型隔震支座将安置于外围用于控制结构的扭转。隔震系统使有效的振动周期接近 5 秒，并产生了约 20%有效阻尼。根据地震分析的结果我们可以看到，与底部固定没有隔震支座的结构弹性响应相比，整个建筑的剪力，绝对加速度，层间位移分别下降了70%-80% 。

该隔震支座通过了"动态隔离系统公司"在内华达州麦卡伦以及在帕维亚的Eucentre实验室实施的一系列大量测试。原型和所有量产隔震支座经受了压缩荷载和剪切位移组合情况下大量详细试验，用以验证设计中假定的性能以及确认隔震支座在超大地震荷载作用下的稳定性。

## 可持续性和隔震

从可持续发展的角度来看，采用隔震系统使项目在社会、 经济和环境方面都有贡献。事实上，从社会角度来看，隔震系统可以减少地震期间建筑的振动量，大大增加建筑内居民的舒适度。根据一系列模拟分析结果，采用2010 年智利8.8级Maule地震期间所录得的 7组地震波，振动量的平均减少了 80%～90%。

关于使用隔震系统的经济效益，经分析传统结构在考虑最大的的地震级情况下 (无隔震系统)，观察到没有、轻微、 中等和大面积损害的概率大致分别是 4%、 36%、 54%和 6%，别。这些数据和当前全球范围内使用的设计规范中的要求目标是一致的。在采用隔震系统情况下，同样的分析表明观察到没有损坏和轻微损坏的概率分别是 94% 和 6%，可达到业主所期望的性能目标。同时，在这种情况下观察到更高水平损坏的概率为零。针对对位移敏感的非结构构件如间隔墙所进行的类似分析，表明传统结构观察到没有、轻微、 中等和大面积损坏的概率大致分别为 27%、 43%、 29 %和 1%。在隔震结构情况下，观察对位移敏感的非结构构件没有损坏的概率是 100%。对楼面加速度敏感的非结构构件情况下，传统结构显示没有损坏的概率和轻微、 中等、 大面积和完全损坏的概率分别是 1%、 12%、 56%、 27% 和 4%。在隔震结构情况下，对楼面加速度敏感的非结构构件无损坏和轻微损坏的概

### Sustainability and Seismic Isolation

From the sustainability point of view, the seismic isolation system incorporated contributes to the social, economical, and environmental aspects of the project. In fact, from the social point of view, the seismic protection system reduces the vibration doses acting on the building occupants during seismic events, increasing considerably the comfort of the residents. According to the results of a series of simulations, performed considering a set of 7 pairs of records recorded during the 2010 Mw 8.8 Maule Chile earthquake, the average reduction of vibration doses is in the order of 80 to 90%.

With regards to the economical benefit of using seismic isolation in the building, the analysis performed yielded that in the case of the conventional structure (without seismic isolation), the probabilities of observing no damage, and slight, moderate, and extensive damage are roughly 4, 36, 54 and 6%, respectively, for the maximum considered earthquake level. These values are compatible with the goals and requirements of current design codes used worldwide. In the case of the isolated structure, the same analysis showed that the probabilities of observing no damage and slight damage are 94 and 6%, respectively, allowing achieving a performance according to owners' expectations. The probabilities of observing higher levels of damage are zero in this case. A similar analysis performed for drift sensitive nonstructural contents, such as partition walls, showed that for the conventional structure, the probabilities of observing no damage, and slight, moderate, and extensive damage are roughly 27, 43, 29 and 1%, respectively. For the case of the isolated structure, the probability of observing no damage on drift sensitive nonstructural components is 100%. In the case of nonstructural components sensitive to floor accelerations, the conventional structure would exhibit probabilities of no damage, and slight, moderate, extensive, and complete damage are 1, 12, 56, 27 and 4%, respectively. In the case of the isolated structure, the probabilities of no damage and slight damage to nonstructural components sensitive to acceleration are 96 and 4%, respectively. All these reductions in the probabilities of damage to structural and nonstructural building contents have a direct impact on the reduction of the economical losses expected during extreme seismic events.

Finally, in relation to environmental issues, it shall be mentioned that the steels used by Dynamic Isolation Systems in anchor bolts, couplers, and shims, sole and masonry plates, are provided by manufacturers that provide steels containing 97% of recycled steel content. Fifteen percent of the recycled steel corresponds to post-industrial steel, while the 82% corresponds to post-consumer steel. The steels products are manufactured in an electric arc furnace process that melts the recovered scrap steel.

### Analysis Methods and Models

The Chilean isolation code allows for using either linear or nonlinear models to analyze 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 analyze and design the whole structure except the seismic isolation system. On the other hand, the maximum considered earthquake (MCE) is used to analyze 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

率分别为 96% 和 4%。在超大地震作用下，减少了对结构构件和非结构构件的损害概率，可直接减少经济损失。

最后，关于环境问题，应提到的是"动态隔离系统公司"使用的锚定螺栓、接驳器、垫片，基础和砌体板块是有再生钢含量 达97%的钢材制造商提供。15%的回收钢材来自工业界， 而82%来自大众消费。这些钢产品是在融化回收刚材的废钢电弧炉工艺制造的。

### 分析方法和模型

智利隔震规范允许使用线性或非线性模型来分析和设计结构以及隔震系统。它还指定了两种级别的地震需求: 设计基础地震和最大考虑地震。设计基准地震(DBE) 用来分析和设计除隔震系统外的整个结构。另一方面，最大考虑地震 (MCE) 被用来分析和设计隔震系统。

Nunoa 首都大厦使用了两种类型的分析。第一，使用了线性分析， 特别是是反应谱分析。在此分析中，隔震系统利用在设计位移水平时的等效割线刚度建模，使用了从地震危险性评估获得的一个特定场地的设计反应谱。该反应谱代表设计基准地震(DBE)所要求的水平，在周期大于3.0秒时通过减少系数1.99以考虑隔震器产生的"额外阻尼效果"。此初始估计系数后来通过非线性分析被核实。线性分析用于设计上部结构，基础和定义隔震系统。

然后实施了非线性时程分析。在此分析中，隔振震支座通过采用为两个剪切变形的耦合塑性性能来模拟成一个非线性的双轴滞回模型 [2]。这会导致隔震支座产生额外的可直接考虑的阻尼并提供系统的有效阻尼的估算。这种分析主要用于验证隔震系统的初步设计。使用了来自地震灾害性分析所得的7组地震加速度记录。这些地面加速度记录代表最大考虑地震 (MCE) 水平的需求。智利隔震规范要求所有的隔震器的参数如位移，轴力需要按照每组地震波最大响应之7组平均值来定。如后面所述，上部结构希望在设计基准地震(DBE)以及最大考虑地震 (MCE) 作用下均保持弹性，所以在ETABS模型中采用线弹性元素来模拟结构。

为设防地震水准的前八个振动周期如表1所示。X方向代表建筑物的长边，Y方向代表建筑物的短边。智利隔震规范允许考虑设计基底剪力为弹性要求除以响应修正系数R=2.0，但是不能少于按照地震带而定的建筑物抗震重量的某个最小百分比。本项目为5%。通常，最小基底剪力介于弹性和折减要求之间，因此有效折减系数R介乎1~2之间 (通过调整R来达到最小基底剪力)，无论如何，本项目已达到长周期，弹性要求小于最小基底剪力，因此使得上部结构要设计成比弹性要求更大，即表示有效折减系数R弹性要求，有效折减系数R，各方向最终设计剪力及其他主要设计参数列于表2。和底部固定结构相比，隔震结构X方向基底倾覆弯矩被放大了 (按弹性设计要求)，这显示了隔震结构在高阶振型的重要影响。 (见表3)

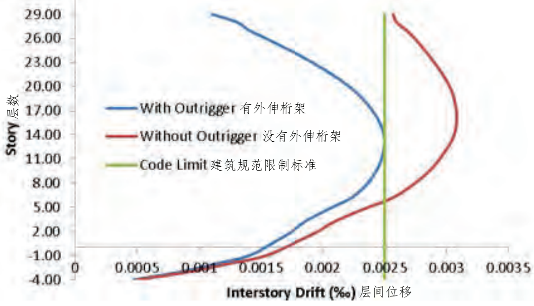


Figure 8. Interstory Drift for the DBE with and without the Outrigger
图8.有与没有外伸桁架的层间位移比较

performed. In this analysis, the isolators were modeled 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 1.99 for periods larger than 3.0 seconds to consider the extra damping provided by the seismic isolation system. This initial estimated factor was later verified through nonlinear analysis. This linear analysis was used to design the superstructure, the foundations and to define the seismic isolation system.

A nonlinear time history analysis was then carried out. In this analysis the isolators were modeled with a nonlinear biaxial hysteretic model that uses coupled plasticity properties for the two shear deformations [2]. This causes the extra damping of the seismic isolation system to be directly considered and provides an estimation of the effective damping of the system. This analysis was used primarily to validate the preliminary design 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 essentially elastic for both the DBE and the MCE, which allows for modeling the building by using the linear elastic elements available in ETABS [2].

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. 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. Notwithstanding this 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 demand, which is equivalent to have an effective reduction factor R* of less than 1. The elastic demands, effective reduction factors R*, final design shears for each direction, and other key design parameters, are shown in Table 2. 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).

## Design Challenges

As explained previously, for this particular project there were two characteristics 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 characteristics imposed two big challenges for the structure design: to control the interstory drift and to avoid tension forces on the isolators.

The Chilean isolation code limits the story drifts to be less than 0.25% for the DBE level. This is different to other codes such as the ASCE7 [3]

| Mode 模型 | Period (s) 周期 | Dir X Mass Ratio (%) X轴系数 | Dir Y Mass Ratio (%) Y轴系数 | Rot. Mass Ratio (%) 旋转系数 |
|---|---|---|---|---|
| 1 | 5.34 | 98.25 | 0.56 | 0.04 |
| 2 | 5.26 | 0.56 | 98.61 | 0 |
| 3 | 4.83 | 0.04 | 0 | 98.65 |
| 4 | 2.13 | 0 | 0 | 0.05 |
| 5 | 1.55 | 0.03 | 0.53 | 0 |
| 6 | 1.41 | 1.1 | 0.01 | 0.01 |
| 7 | 1.4 | 0.01 | 0 | 0.68 |
| 8 | 1.13 | 0 | 0.28 | 0 |

Table 1. Periods of the Structure for the design earthquake level
表1. 设计地震水平时结构的周期

| | X Direction | Y Direction |
|---|---|---|
| Elastic Base Shear (kN) 弹性基底剪力 (kN) | 13,368 | 13,734 |
| Design Base Shear R=2 (kN) 设计基底剪力[修正系数R=2] (kN) | 6,684 | 6,867 |
| Minimum Design Base Shear (kN) 最小设计基底剪力 (kN) | 20,346 | 20,346 |
| Effective Reduction Factor R* 有效折减系数R* | 0.657 | 0.675 |
| Final Design Base Shear (kN) 最终设计基底剪力 (kN) | 20,346 | 20,346 |

Table 2. Elastic and Final Design Base Shear
表2. 弹性及最终设计基底剪力

| | Mode 模型 | Base Shear 基底剪力 kN | Base Overturning Moment 基地倾覆力矩 kN-m | Base Overturning Moment 基地倾覆力矩贡献 % of combined (CQC) |
|---|---|---|---|---|
| Isolated Structure 隔震结构 | ALL MODES (CQC) | 20,346 | 1,043,967 | 100 |
| | MODE 1 | 19,955 | 837,546 | 80 |
| | MODE 2 | 117 | 4,861 | 0 |
| | MODE 3 | 11 | 376 | 0 |
| | MODE 4 | 0 | 0 | 0 |
| | MODE 5 | 61 | 10,334 | 1 |
| | MODE 6 | 2,906 | 607,879 | 58 |
| | MODE 7 | 30 | 6,257 | 1 |
| | MODE 8 | 1 | 291 | 0 |
| Fixed Base Structure 固定基础结构 | ALL MODES (CQC) | 20,346 | 868,339 | 100 |
| | MODE 1 | 13,950 | 835,313 | 96 |
| | MODE 2 | 395 | 23,367 | 3 |
| | MODE 3 | 43 | 2,529 | 0 |
| | MODE 4 | 4 | 236 | 0 |
| | MODE 5 | 13 | 587 | 0 |
| | MODE 6 | 1 | 42 | 0 |
| | MODE 7 | 8 | 74 | 0 |
| | MODE 8 | 43 | 206 | 0 |

Table 3. Contribution of each vibration mode to the base overturning moment in X Direction
表3. 在X方向每个振型对基底倾覆力矩的贡献

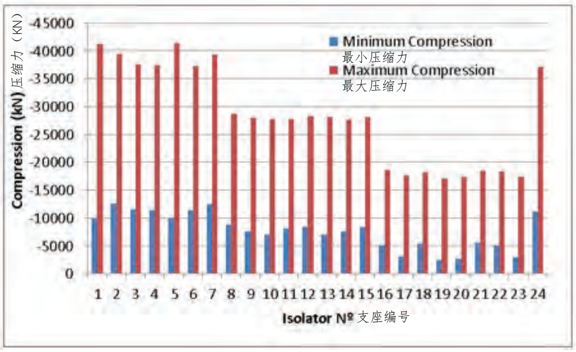


Figure 9 - Compression Forces on each Isolator for the MCE
图9-MCE地震级时每个隔震支座上的压缩力

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. Even though this location was not optimal, the outrigger's benefits were enough to stiffen the structure and reduce the interstory drifts to below the code limits. This can be seen for the X direction in Figure 8.

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 using a 2 m in thickness slab resting directly above the isolators. This thick slab stiffens the isolated interface so that the external isolators are coupled with the internal ones, leading to a greater lever arm that 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 9. To enhance the performance of this thick coupling slab, imbedded beams with closely spaced stirrups and ties were designed.

## Conclusions

The final design 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, and in consequence expecting less structural damage (elastic response of superstructure), nonstructural damage (reduced interstory drifts) and content losses (lower absolute accelerations) than its fixed base counterpart for an MCE level earthquake, leading to a sustainable design.

## 设计中的挑战

如前面所述，本项目有两个与普通低层隔震结构不同的特点: 一是基底剪力大于弹性要求; 二是底部倾覆力矩的加大设计，这两个特点为结构设计提出了两大挑战: 控制层间位移和避免隔震支座产生拉力。

智利隔震规范限制设计基准地震DBE 级别的层间位移要小于0.25%。这不同于其他规范，如 ASCE7 [3]，因为它是直接从减少DBE 频谱 (按R*因素 )测量而不是由Cd/I乘以弹性反应。

为控制层间位移，有必要增加刚度，这是一项挑战，因为非常紧凑的建筑布局几乎没有空间来增加或加厚结构。最终通过在每个塔楼设备层) 采用 2个外伸桁架来实现。即使这个位置不是最理想的，外伸桁架的好处是足以加固结构并控制层间位移控制在规范限值以下。如图 8 中的 X 方向所示。

第二个挑战是避免隔震支座产生拉力。虽然隔震支座可以抵抗小拉力，设计团队决定为MCE级别消除隔震支座的拉力。具体是通过连接两个塔楼在地下室部分的楼面板，以及直接在隔震支座上设置了2米厚楼板来实现。这个厚板加固了隔震支座界面以便外部隔震支座和内部隔震支座产生耦合效应，从而形成更大的力臂来抵抗倾覆力矩并减少地震压缩拉力。MCE级别的在隔震支座的最终压缩力如图9所示。采用了具有密集箍筋连接的埋入梁来来加强耦合厚板的性能。

## 结论

最终设计证实了在高层建筑实施隔震系统的技术可行性。主要设计挑战包括采用抵抗大压缩力的大直径隔震支座的使用、同时实现高竖向刚度和横向抗弯的困难，避免隔震支座的拉力，控制层间位移小于规范要求。在33层结构中使用隔震支座的可行性与项目场地良好的地质条件相关，使用相对刚度大的上部结构和伸臂桁架。结构构件的尺寸由层间位移确定而非设计力。隔震系统的使用已证明可以大大减少地震需求，甚至如Nunoa 首都大厦这样的高层建筑,，因此，在比MCE 级地震作用下，和底部固定结构相比，结构损伤更少 (因上部结构采用了弹性反应) 、非结构性损伤也更少 (因减少了层间位移) 和建筑物内部损失更少 (因更小的加速度响应)，达到具有可持续性的设计目标。

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