DYNAMIC BEHAVIOR UNDER STRONG MOTIONS AND STRUCTURAL DESIGN PROCEDURES OF SEISMICALLY ISOLATED BUILDINGS

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ABSTRACT

This paper reviews the dynamic behavior under strong motions and structural design procedures of seismically isolated buildings. Chile and Japan are known as countries very prone to earthquakes and have a long history of earthquake damages and countermeasures. Seismic isolation system is a key issue to mitigate structural disasters against strong shaking, thus it is necessary to understand the characteristics of ground motions and the behaviors of seismically isolated buildings to enhance the seismic safety against earthquakes.

In Chile, after last 2010 earthquake, seismically isolated buildings have begun to be accepted into our society in order to keep structural safety and functionality, and to ensure that, base isolation system should be examined continuously to verify the validity of structural design.

Firstly, this paper introduces the observed behavior of seismically isolated buildings during last major earthquakes and advantages of this type of systems. Secondly, summarizes the strong motion observation of a target building and dynamic behavior of target building based on observed records. Finally, structural design of isolation systems is presented through a fundamental procedure.

Keywords: Seismically isolated building, Strong motion, Design procedure.

1. INTRODUCTION

Chile is a land of earthquakes, it is located in the Ring of Fire, an area with highest seismic activity. The earthquakes occur due to release of mechanical stress of the subduction movement of Nazca Plate below the South American Plate on the Chile-Peru Trench, which is an active area of the Ring of Fire.

Along its history large earthquakes have shaken hundreds of times the territory, including the largest earthquake in the world in 1960 with magnitude of 9.5Mw. The last important earthquake (with major destructive tsunami) is also the 6th largest Earthquake in the World (USGS Web site, 2012) and it occurred off the coast of Chile's Maule Region on February 27, 2010 (hereinafter called 2010 Chile earthquake). Its epicenter was at 35.909S, 72.733W offshore from Maule, it has a magnitude of 8.8Mw and the rupture area (parallel to the coast) had about 550km of length, 100km of width and 50km of depth (Pararas-Carayannis, 2010).

The development of seismic isolation in Chile was started with the first seismically isolated building as part of a project of University of Chile in 1992, where this technology was first introduced in the country. With the development of the technology in Chile, great deal of effort was put into the project to establish Chilean seismic codes in 2003 to fulfill the minimum requirements for seismically isolated buildings under Chilean seismic characteristics. After the publication of the seismic isolation code, the number of seismically isolated projects has increased. But, it was not until the occurrence of

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2010 Chile earthquake when the technology acquired national connotation due to its good performance in different types of structures. After the earthquake several hospitals could not be operated due to structural and non-structural damages. The mitigation of non-structural damages was having the same importance to enable the building due to its repair cost. Thus, we have learnt from the last earthquake it is important to avoid the interruption of operation in at least essential buildings.

At present, Chile has approx. 50 seismically isolated structures and the seismic isolation is used not only for private structures such as buildings, but also for bridges, harbors and gas plants, now the technology is starting to spread to public structures such as hospitals and public housings.

This study aims to achieve two main objectives: Contribute to permanent effort of decrease the seismic vulnerability and to protect the human life; contribute effectively on Chilean practice of seismic isolation design by learning and comparing with actual knowledge and procedures used in Japan.

The contents of this study are summarized as: Chapter 2 presents seismic behavior of seismically isolated buildings in Chile and Japan during last major earthquakes; Chapter 3 studies the behavior of a target seismically isolated building based on observed records; Chapter 4 discusses simulation analysis of target building trying to achieve same response through an analytical method; Chapter 5 presents a fundamental procedure of seismically isolated building design; Chapter 6 corresponds to final conclusions.

2. SEISMIC BEHAVIOR OF SEISMICALLY ISOLATED BUILDINGS IN CHILE AND JAPAN DURING LAST MAJOR EARTHQUAKES

2.1. Seismic Behavior Observation after 2010 Chile Earthquake

At the moment of the Mw 8.8 2010 Chile earthquake only one seismically isolated building was instrumented, which is the first isolated building in Chile built in 1992 in Santiago, called Andalucia Community Building. This building is part of a research project developed by Sarrazin & Moroni at University of Chile.

The superstructure corresponds to 4-story confined masonry building, with the first level of reinforced concrete and equipped with 8 high damping rubber isolators. The floor area is 6m x 10m with a total height of 9.5m.

From the structure several records have been obtained, and the most important one is the 2010 Chile earthquake record and an aftershock recorded on March 11 with a magnitude 7.0Mw (317km from the epicenter). The peak accelerations are summarized in Table 1.

Table 1. Peak accelerations on Andalucia Community Building.

	2010 Chile earthquake		Aftershock on May 11	
Direction	**Basement Acc. cm/s^2**	**4^{th} level Acc. cm/s^2**	**Basement Acc. cm/s^2**	**4^{th} level Acc. cm/s^2**
Horizontal	303	218	34	36
Vertical	178	284	20	30

The seismically isolated building has shown an excellent seismic behavior without structural and non-structural damages even during the 2010 Chile earthquake; on the other hand, the fixed building suffered some light cracks and non-structural damages (Civil Engineering Department, University of Chile, 2012).

2.2. Seismic Behavior Observation after 2011 Tohoku Earthquake

This subchapter is based on an investigation made in June of 2011 for a team consisted by the members of NILIM (National Institute of Land, Infrastructure and Management) and BRI (Building Research Institute) (Saito, Iiba, et. al., 2012 & Saito, Okawa, & Kashima, 2012).

During the 2011 Tohoku earthquake, in spite of the strong shaking with JMA intensity 6 upper, the seismically isolated buildings showed excellent performance without any severe structural damage even to non-structural damages in superstructure. However, some damages to isolation dampers and expansion joints were observed.

Some damages in dampers were observed, for example many cracks were found in lead dampers. In addition, peeling off of paint was observed in U-shape steel dampers and residual deformation of steel was remained.

On the other hand, the one-story steel warehouse located in Miyagino-ward in Sendai City, near to the bay, was reached by tsunami. Building and the SI floor was submerged under the water taking 16 days to remove all water. Tsunami height was estimated in around 4m and external walls presented damages by the collision of floating debris. There was no harmful scratch or inflation of the rubber of HRB (high rubber bearing); however severe rust at the steel plates and bolts was observed.

3. BEHAVIOR OF SEISMICALLY ISOLATED BUILDING BASED ON OBSERVED RECORDS

The target is Tsukuba City Hall Building, hereinafter called Tsukuba Building, is located in Ibaraki Prefecture at 334km from the epicenter and 24km in focal depth in the 2011Tohoku earthquake. The isolation system consists of 20 Natural Rubber Bearings (NRB), 45 Lead Rubber Bearings (LRB) and 9 Natural Rubber Bearings with Steel Dampers.

The mainshock of 2011Tohoku earthquake was recorded in Tsukuba Building on March 11, 2011, at 14:46 (local time) with intensity 5.2 of JMA Seismic Scale. In comparison with the earthquake motion on the basement and the response on the first floor, the peak acceleration decreases from 326.9cm/s^2 to 91.7cm/s^2 (in Y direction). That in Z direction shows acceleration amplification where the peak acceleration on the sixth floor is twice that on basement (121.9cm/s^2). The superstructure suffered a small amplification from magnitude 4.6 to 4.8 in JMA scale.

This study uses records from 461 accelerograms measured effectively in Tsukuba Building, between February 05, 2011, and June 06, 2012, including the mainshock of the 2011 Tohoku earthquake. The results will be shown below according to analysis directions.

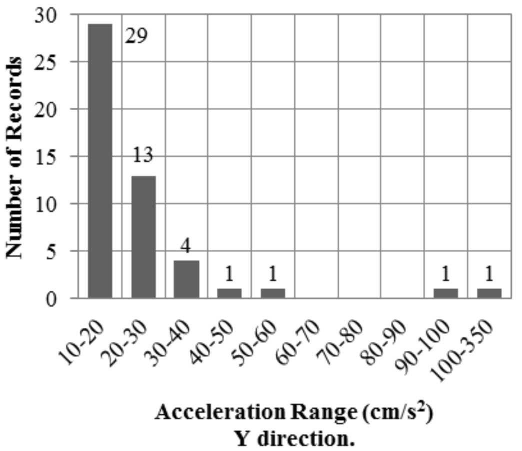

Figure 1. Distribution of basement acceleration ranges between 10cm/s^2 and 350cm/s^2

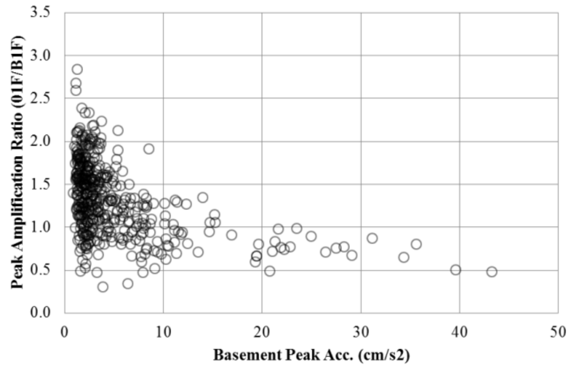

Figure 2. Basement peak acceleration less than 50cm/s^2 vs. ratio 01F/B1F.

Figure 1 shows a bar graph focusing on the accelerations between 10cm/s^2 and 350cm^2. The total number of events in this range is 50 and this graph is showing that the number of events is decreasing according to increase of peak acceleration range. There are 411 events lower than 10cm/s^2.

Figure 2 presents the peak acceleration ratios (1F/B1F) with basement acceleration less than 50cm/s^2. When the basement accelerations are more than 10cm/s^2 the ratios are less than 1.0. The isolation effect is confirmed.

4. SIMULATION ANALYSIS OF SEISMICALLY ISOLATED BUILDING

The main objective of this chapter is to describe, with an analytical model, the behavior observed in Tsukuba building during 2011 Tohoku earthquake and to compare properties considered in the design versus properties observed during strong motions.

4.1. Fitting for characteristics of superstructure

The first step of this analysis is to make an analytical model to obtain the design period of the superstructure using 3% of damping factor, from this analysis the primary natural period of superstructure is 0.591 second for X direction and 0.598 second for Y direction.

Then the observed frequency domain of superstructure should be established, through the Fourier spectral ratio between sixth and first floors, using the average natural frequency of three small records (with JMA intensity of 2.0) to obtain the elastic stiffness of the superstructure in the range of small displacements. Fourier spectral ratio graphs between sixth and first floors indicating the response characteristics of superstructure and the peaks corresponding to the first natural frequency.

To reach the analytical case the same observed period will change the stiffness of superstructure simply using the equation: $T = 2\pi\sqrt{M_{\text{sup}}/k_{\text{sup}}}$.

The stiffness of each story of the superstructure is modified to 2.1 times the stiffness considered in the design process.

4.2. Fitting for first stiffness of seismically isolated system

Using the designer properties and the modified stiffness of superstructure done in the last section, is calculated the first natural period of in both analysis directions of the seismically isolated building. For X direction the period is 1.376 second and 1.366 second for the Y direction.

To calibrate the first stiffness of designer force-displacement total curve, it is chosen a big record. It has a large JMA intensity (greater than 4.0) that causes a displacement closest to yielding point of the system. The record is the second largest record of Tsukuba building, while the largest one is the mainshock of 2011 Tohoku earthquake.

Therefore, the periods of entire system should be modified from 1.376 second (X direction) and 1.366 second (Y direction) to an observed period of 1.0 second according to Fourier spectral ratio 06F/B1F and phase lag. By modification of first stiffness of isolator devices with keeping the values of Q2, δ2 and K3 of total curve, only the first stiffness of LRB and NRB with steel damper are modified. It means that the stiffnesses K1 and K2 of total curve are changed. The results are shown below.

Table 2. Modified properties of total curve of isolators and dampers.

Characteristics						
NRB	Number	K1 (kN/mm)				
	20	24.54				
LRB	Number	**K1 (kN/mm)**	Q1 (kN)	**K2 (kN/mm)**	Q2 (kN)	K3 (kN/mm)
	45	**1,218.98**	7,976	**87.29**	9518	54.44
Steel Damper	Number	**K1 (kN/mm)**	Q1 (kN)	K2 (kN/mm)	-	-
	9	**469.24**	3,070	2.30	-	-
Total Curve of Isolated Layer			Weight(kN) =		103,113.6	
NRB+LRB+ Steel damper	Number	**K1 (kN/mm)**	Q1 (kN)	**K2 (kN/mm)**	Q2 (kN)	K3 (kN/mm)
	74	**1,712.76**	11,207	**114.13**	13,224	82.20

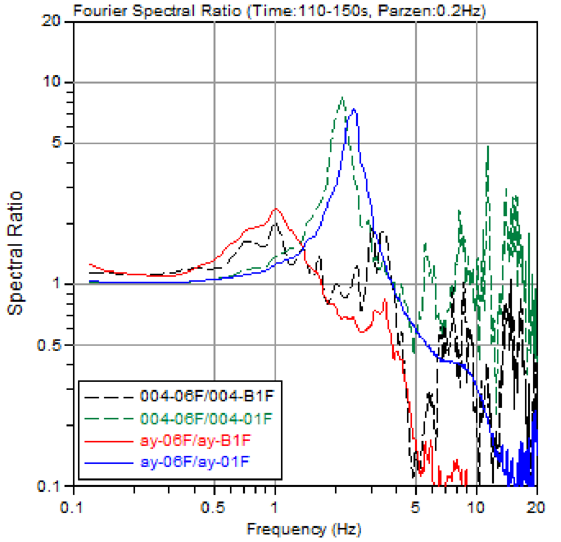

Figure 3. Fourier spectral ratio of observed record (dashed lines) and analytical case (solid lines).

As the peak of Fourier spectrum (3.65 at approx. 1Hz for entire building) is almost two times larger than the corresponding spectral ratio, the peaks in the modeling should be decreased applying an additional damping factor. The additional damping is calculated according to the maximum relative velocity within the isolated layer and was calculated to be approx. 0.10m/s.

Finally, Figure 3 shows a clear similarity among waveforms of observed record measurement on the building and the response of the building from the structural model with the record input. The behavior of superstructure is represented by green dashed line for observed record and blue solid line for the modeling case, those show almost same peak values but have a small gap between them, which is a peak of 8.62 versus 7.35 respectively. On the other hand, black dashed line and red solid line represent behavior of the entire system for the observed and modeling case respectively. In this situation both peaks appear are in a frequency of 1Hz but they have a small difference of amplitude, at a peak of 2.38 vs. 2.04 for observed one. However, this simulation reaches remarkable approximations.

5. STRUCTURAL DESIGN OF SEISMICALLY ISOLATED BUILDING

Two design procedures are adopted in Japan according to new regulations stipulated in Notifications 1461 and 2009 in October of 2000 from the Ministry of Land, Infrastructure, Transport and Tourism. The techniques of design are: time-history analysis method and equivalent linearization method (ELM) using a design spectrum. According to the regulation, seismically isolated building with height equal to or less than 60m, first or second site ground classification and seismic isolation at the base does not need authorization by Minister (Saito, Hiramatsu, & Kani, 2011). Is this structural design the technique of time-history analysis is selected.

A procedure of design of seismically isolated building is briefly explained below, in which there are some recommended values used in the practice. The adopted procedure is as follows:

(1) Target building input data: preliminary sections of elements, building location and a soil type.
(2) Calculation of superstructure using simplified method by pushover analysis with base shear coefficient between 0.15 and 0.20. It is recommended to use 0.18.
(3) Setting appropriate design of superstructure such as sections of structural elements and reinforcement.
(4) Static analysis of superstructure using long term loads such as dead (DL) and live loads (LL). Determine building weight (W) and vertical forces to isolators (NL) due to long term loads.
(5) Preset isolators determining minimum diameters in accordance with maximum axial stress. It is common to use minimum value of 10N/mm^2.
(6) Setting target performance of seismically isolated building:
    a) Target period (Tf) between 2 and 5 seconds. It is recommended to the period in 4 seconds.
    b) Ratio Qy/W $\leq$ 5.0% (Qy; yield horizontal force), whose recommended value should be between 1.5% and 3.0%.
    c) Determine maximum target displacement. It is recommended to set 40cm.
(7) Check distance between center of gravity and center of stiffness is less than 3%.
(8) Make dynamic analysis (time-history) on entire system (superstructure and seismic isolated layer). Equivalent MDOF system of superstructure is obtained through Pushover analysis.

(9) Check performance limitation of isolators using vertical forces due to long term loads and short term loads (seismic lateral forces) and check tensile strength less than 1N/mm2.
(10) Comparison between response and performance of entire system.
(11) Check design against wind forces.

6. CONCLUSIONS

From analysis of behavior of seismically isolated building based on observed records the following understanding are gained.
a) The input spectrum at the basement is dominant in short-periods, and predominant at the range of 0.2 to 0.3 second, while the predominant periods at the range of 3 to 4 seconds are remarkable in both directions for the velocity response spectrum during mainshock.
b) 461 accelerograms were measured in Tsukuba building, 99% of them have maximum horizontal acceleration less than 50cm/s^2, and was observed that for accelerations larger than 10cm/s^2 amplification of acceleration on first story dominates reduction.

Through simulation analysis dynamic characteristics of base isolated building were compared designer considerations versus real parameters obtained from to observed records.
a) After setting the first stiffness of entire system according to the first natural frequencies of Fourier spectrum analysis, it is possible to notice that the analytical stiffness of superstructure is approx. twice of the stiffness considered in the design procedure.
b) Simulation analysis reaches remarkable approximations through changing stiffness according to observed records.

A simple design procedure should be done to understand the seismic behavior to comprehend dynamic characteristics and to gain better understanding and to verify the validity of structural design.
a) It is recommendable to set lead rubber bearings in external perimeter of the arrangement and natural rubber bearing at the center to avoid torsional movement through their stiffnesses.
b) Lead rubber bearing has major influence for the calculation of the yield horizontal force and consequently it is important to set a determined value of Qy/W.
c) To calculate the period of seismically isolated building it was used the second stiffness of the total load-displacement curve of isolator arrangement, where natural rubber bearing has importance.

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