Principles and Practices of Seismic Isolated Buildings

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Submitted to the
Institute of Graduate Studies and Research
in partial of the requirements for the Degree of

Master of Science
in
Civil Engineering

Eastern Mediterranean University
July 2009
Gazimagusa, North Cyprus

Approval of the Institute of Graduate Studies and Research

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ABSTRACT

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Earthquake design philosophy based on capacity, directs the following two unpleasant states:

1. The situation that continues to increase the elastic strength and stiffness; in fact this is not economical and also cause higher floor accelerations.
2. The situation that limits the elastic strength and increasing ductility by detailing; indeed this approach is the acceptance of non-repairable structural damages.

Base isolation is a different approach than the mentioned ones. It is based on the concept, which reducing the seismic demands rather than increasing the earthquake resistance capacity of the structure. On the other hand, application of base isolators to the structure reduce elastic base shear by shifting period of the structure and provide better performing structure that will remain essentially elastic during large earthquakes.

However, in this thesis, general information about seismic isolated structures such as type of isolators, world-wide applications, practical applications, properties, code requirements and different processes required for designing various seismic isolators are discussed. Then, 3 different buildings (3, 6 and 9 story) which were isolated by 3 various isolators (Lead Rubber Bearing, High Damping Rubber Bearing and Friction Pendulum System) were analyzed by applying dynamic response spectrum analysis, as a linear elastic analysis method, to evaluate the optimum one according to the seismic demands. Transmitted acceleration, maximum structural displacement and seismic coefficient for each building were shown in the different graphs. Furthermore, the 3 story optimum isolated building was compared with its conventional fixed base one in performance and material.

Based on obtained results, it could be inferred that Lead Rubber Bearings represent minimum transmitted acceleration and seismic coefficient among other types. Low effective stiffness and high damping which is represented by Lead Rubber Bearings are the most important factors for this minimization. Structural displacement is minimized by Friction Pendulum Systems due to the high friction of coefficient which they produce. In addition, in rubber bearings transmitted acceleration and structural displacement is affected by damping of isolation system. Furthermore, in the comparison process of base isolated building with its conventional fixed base one, it is concluded that application of the base isolators to the structure increase cost of the building around 5.8 % of total cost.

Keywords: Base isolation, Isolator, Cost, Earthquake, Strengthening.

ÖZET

Sismik Taban Yalitimli Binalarda Temel Prensip ve Uygulamalar

Kapasiteye dayanan deprem tasarim felsefesi bizi asagidaki iki kötü seçime yönlendirmektedir:

1. Elastik dayanimi sürekli olarak artirmak; Bu yaklasim ekonomik degildir ve yüksek kat ivmelerine sebep olmaktadir.
2. Elastik dayanimi sinirlandirmak ve detaylandirarak düktiliteyi artirmak; Bu yaklasim ise ileride binada tamir edilemeyecek yapisal hasarlarin kabülü sayilir.

Sismik taban izolasyonu yukarida belirtilenlerden farkli bir yaklasimdir. Yapinin deprem direnç kapasitesini artirmak yerine sismik talepleri azaltmaya yönelik bir yaklasimi temel alir. Diger taraftan, yapiya taban yalitiminin uygulanmasi yapinin periyodunu kaydirarak elastik taban kesme kuvvetini azaltir. Bununla beraber büyük depremlerde esasen elastik davranisi koruyan daha iyi bir yapi performansi saglanmis olur.

Bu tez çalismada izolasyon sistemleri, dünyadaki uygulamalari, pratik applikasyonu, özellikleri, yönetmelik esaslari ve degisik izolatörlerin tasarimi gibi konularda detayli bilgi verilmistir. Daha sonra ise degisik kat yüksekliklerine sahip (3, 6 ve 9), 3 ayri binaya 3 farki tip taban izolatörü (kursun çekirdek mesnet sistemi, yüksek sönümlü dogal kauçuk mesnet sistemi ve sürtünmeli sarkaç sistemi) uygulanarak response spektrum analizi yapilmistir. Farkli grafiklerde maksimum kat ivmeleri, maksimum deplasman ve taban kesme katsayilari karsilastirilmistir. Son olarak 3 katli ankastre ve izolatörlü yapilar performans ve malzemeye dayali olarak karsilastirilmistir.

Elde edilen sonuçlardan yola çikarak kursun çekirdek mesnet sistemi digerleri arasinda minimum ivme ve sismik katsayiyi vermistir. Yüksek viskoz sönüm ve düsük rijitlik özellikleri bunu saglayan en önemli faktörlerdir. Yapisal deplasman sürtünmeli sarkaç sisteminde yüksek sürtünme katsayisina bagli olarak minimize edilmistir. Buna ilaveten, kauçuk mesnet sistemlerinde ivme ve yapisal deplasmanin izolasyon sisteminin viskoz sönümden etkilendigi söylenebilir. Sonuç olarak ankastre ve izolatörlü yapilar arasinda yapilan karsilastirmaya dayali olarak temel izolatörlü yapilarin toplam maliyeti 5.8% artirdigi gözlenmistir.

Anahtar Kelimeler: Taban izolasyonu, Izolator, Maliyet, Deprem, Güçlendirme.

ACKNOWLEDGEMENT

I take this opportunity to appreciate of my parents and my brothers (Reza and Omid) for their emotional support and encourage me from the start until the accomplishment of this manuscript; you have been my source of strength and love. It wouldn't have been this bearable if I didn't have the four of you in my life.

I would like to express my deepest appreciate to Asst. Prof. Dr. Giray Özay, my academic advisor and supervisor for guiding me and his emotional support all along my thesis program. I am equally grateful to all the staff of the Department of Civil Engineering especially Asst. Prof. Dr. Huriye Bilsel (Chairwoman), Prof. Dr. Ali Günyakti (my advisor), Asst. Prof. Dr. Erdinç Soyer, Asst. Prof. Dr. Serhan Sensoy, Asst. Prof. Dr. Mürüde Çelikag and Assoc. Prof. Dr. Özgür Eren.

I want to express my gratitude to my colleagues and my friends Amir Hedayat who kindly shared his knowledge and experiences with me, Milad Gorban Ebrahimi, Saman Esfandiarpour, Pedram Hessameddin, Saeed Kamkar, Arash Frazam, Amir Khanlou, Temuçin Yardimci, Mana Behnam, Obina Onuaguluchi, Anoosheh Iravani and a host of other friends.

Finally, I would like to extend my deepest appreciation to these people who have helped me in one way or another to finish the task at hand, this thesis manuscript.

Dedicated to my parents (Hossein & Mahin)

and my brothers (Reza & Omid)

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A	Cross-section area
A_b	Bounded area of the rubber
A_g	Gross area of bearing
? ?	Area of hysteric loop
A_{pl}	Area of the lead core in LRB
A_r	Reduced area of rubber bearing
A_{sf}	Minimum area based on shear failure
A_0	Cross-section area base on axial stress
A_1	Cross-section area based on shear strain
B	Numerical coefficient related to effective damping of isolation system
B	Overall plan dimension of bearing
B_b	Bounded plane dimension of the isolator
B_M	numerical coefficient related to the effective damping of isolation system at maximum displacement
b	The shortest plan dimension
C_A	Seismic coefficient in Design Base Earthquake
C_{AM}	Seismic coefficient in Maximum Capable Earthquake
C_{VD}	Seismic coefficient
C_{VM}	Seismic coefficient
? ?	Design displacement
? ? ?	Design displacement at the center of the rigidity of the isolation system
? ?	Maximum displacement at the center of rigidity of the isolation system
? ? ?	Maximum displacement at the center of the rigidity of the isolation system
D_{TD}	Total design displacement

D_{TM} Total maximum displacement

d The longest plan dimension

d_{pl} Core diameter in LRB

E Elastic modulus of rubber

$?_?$ Buckling modulus of rubber

E_c Effective compressive modulus of rubber bearing

$?_?$ Buckling modulus

e Eccentricity

$?_?$ Force in bearing at specified displacement

F_y Yield force of Lead

f Factor applied to elongation for load capacity (1/ factor of safety)

f_y Bending reinforce yield stress of reinforcement

f_{ys} Shear reinforce yield stress of reinforcement

$?_?^?$ Specified concrete compressive strength

$?_?$ Shear modulus of rubber at shear strain ?

? Ground acceleration

$?_?$ Height of the bearing free to buckle

h Total height of the individual bearing

I_m Moment of inertia

I Important factor

K Material constant

K_{Dmax} Maximum effective stiffness of the isolation system at the design displacement

K_{Dmin} Minimum effective stiffness of the isolation system at design displacement

$?_{???}$ Effective stiffness of isolation system

K_{em} Maximum effective stiffness of individual rubber isolator

$?_?$ Horizontal stiffness of isolation system

K_{Mmin} Minimum effective stiffness of the isolation system at maximum displacement

$?_?$ Lateral stiffness after yield of LRB

$?_{??}$ Vertical stiffness of layer i

K_u Elastic lateral stiffness

M Seismic mass

M_M Maximum Capable Earthquake response coefficient

N_a Near-source factor

N_v Near-source factor

n Number of rubber layers in rubber bearing

P Bonded perimeter of rubber bearing

$?_{??}^{?}$ Buckling load

$P_?$ Maximum rated vertical load

Q_D Characteristic strength of LRB

R Reduce of disk in FPS

R_f Fixed base lateral force coefficient

R_I Lateral force coefficient

$?_?$ Spectral acceleration at effective period $?_?$

$?_?$ Shape factor

T Structural isolated period

$?_?$ Effective period of isolation system

T_M Effective period at maximum displacement

T_r Total rubber thickness of rubber bearing

t_i	Layer rubber thickness of rubber bearing
t_{pl}	Load plate thickness of rubber bearing
t_{sc}	Cover thickness of rubber bearing
$?_{??}$	Thickness of internal shims in rubber bearing
V_b	Total design shear force on elements below isolation system
V_s	Total design shear force on elements above isolation system
?	Seismic weight
y	The distance between the center of the rigidity of the isolation system and the element of interest
Z	Seismic zone factor
?	Damping of the isolation system
$?_{??}$	Shear strain under earthquake load
$?_{??}$	Shear strain under compression
$?_{???}$	Design shear strain
$?_{??}$	Shear strain under rotation
?	Applied displacement
$?_{?}$	Maximum applied displacement
$?_{?}$	Maximum yield displacement
$?_{?}$	Vertical displacement of FPS
e_b	Elongation of rubber at break
e_c	Compressive strain of rubber bearing
e_{sc}	Shear strain due to the vertical load of rubber bearing
$?_{??}$	Shear strain due to the lateral load
$?_{??}$	Shear strain due to the rotation
e_u	Minimum elongation at break of rubber

?	The applied rotation
?	Displacement factor
s_c	Axial stress
s_y	Lead yield stress
µ	Friction coefficient in FPS

LIST OF ABBREVIATIONS

DBE Design Base Earthquake

DIS Dynamic Isolation System

EB Elastomeric Bearing

EPS Earthquake Protection System

F Friction damper

FEMA Federal Emergency Management Agency

FPS Friction Pendulum System

FSB Flat Slider Bearing

HDRB High Damping Rubber Bearing

LRB Lead Rubber Bearing

MCE Maximum Capable Earthquake

NRB Natural Rubber Bearing

PTFE Polytetrafluoroethylene

RC Reinforce Concrete

S Steel damper

SEP Seismic Energy Products

SHPO State Historic Preservation Office

UBC Uniform Building Code

CHAPTER 1

INTRODUCTION

1.1 Introduction

Every year lots of people die because of earthquakes. Usually, this type of disaster occurs due to the problems related with the buildings' performances. Especially the countries in the high seismic zones such as America, Japan, Turkey, and Iran are the significant cases which are under the danger. In these regions, structural engineers consider their own earthquake specifications when designing different structures so that they can survive after earthquake. In the design process, for all of the load cases, they encounter to meet a single basic equation:

Capacity > Demand

It is known that earthquakes happen and are uncontrollable. Therefore, the demand should be accepted and make sure that the capacity surpasses it. The internal forces in the structure depend on building mass and ground acceleration. When the ground acceleration increases, the strength of the building must be raised to reduce structural damages. However, it is not possible to increase the strength of the building as limitless. Usually in the high seismic zones (near fault) transmitted acceleration to the structure exceeds one or even two times ground acceleration. On the other hand, strength of a building to resist one g means that the building could resist gravity applied sideways. In that level of strength, designing a building is not easy and economical. Hence, codes allow engineers to use ductility factors for reducing earthquake forces. Ductility means, allowing the structural elements to deform beyond their elastic capacity. In this case, displacements increase with the small changes in forces (Figure 1.1).

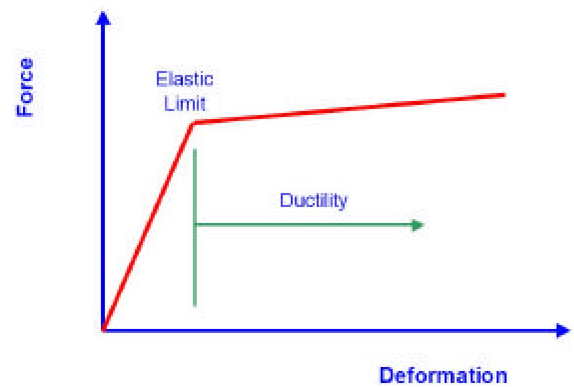


Figure 1.1: Force deflection curve of structure (T.E. Kelly, 2001).

The elastic limit is the load which its effects are not permanent. After removing the load, materials return to their initial properties. Once this elastic limit is exceeded, changes occur in material properties. These changes are permanent and non-reversible after removing the load. Generally ductility causes visible damages on the structures. In the case of the concrete structures, cracks form when concrete exceeds its elastic limit in tension (T.E. Kelly, 2001).

In fact, earthquake design philosophy based on capacity, directs the following two unpleasant states:

1. The situation that continues to increase the elastic strength and stiffness; in fact this is not economical and also cause higher floor accelerations.
2. The situation that limits the elastic strength and increasing ductility by detailing; indeed this approach is the acceptance of non-repairable structural damages.

Base isolation is a different approach from the mentioned ones. It is based on the concept, which reducing the seismic demands rather than increasing the earthquake resistance capacity of the structure. Therefore, the primary reason to use isolators is to reduce the earthquake forces (T.E. Kelly, 2001).

Most of the time, it is thought that the stronger connection between superstructure and foundation can protect building during earthquake. However, these connections can't reduce accelerations, shear forces and frequencies. Therefore they transmitted exactly to the superstructure. When earthquake happens, foundation may moves with ground shaking, superstructure deformed (this deformation is due to the inertia force that depends on buildings acceleration) and seismic waves are transmitted to the structure via these connections (Jacobs, 2008). From many years ago, civil engineers had used old methods to reduce level of damages during earthquake. However in the last decade remarkable progresses have been achieved. A medical doctor (1909) in England invented the first seismic isolator. He used fine sand and mica or talc under foundation to protect building during earthquake. After this English scientist, John Milne, who was the professor of mining engineering in Tokyo improved this concept. He used balls that put in the concave cast-iron plate. Finally, John Milne built the first isolated building over balls to test. However, he couldn't be successful. The structure had small displacement under wind load (Naeim and J.M. Kelly, 1999). Hence it is changed to the main concept for other engineers to discover new devices and improve their ideas.

Finally, for nearly four decades, seismic analysis engineers have been perfecting unusual and complex systems call base isolators. It is aimed to protect buildings from earthquake, which become a major feature for new and retrofit buildings. These base isolation systems save thousands of people's life, but unfortunately the public consciousness about the method is negligible (Jacobs, 2008).

1.2 Main Concepts of Seismic Isolation

Some main concepts of seismic isolation are:

1. To protect building from ground motions by shifting the period of the structure is shown at Figure 1.2. However, it is necessary to be noted that, the structural period should be selected carefully in order to prevent resonance phenomenon. (Tonekaboni Pour, 2005).

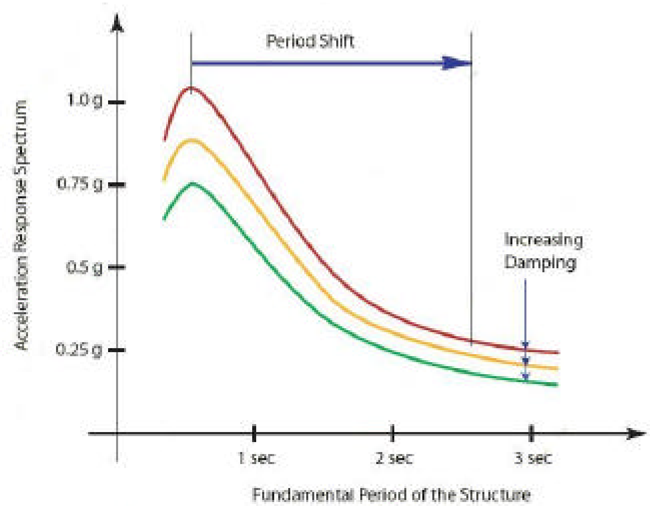


Figure 1.2: Shifting period of structure (Dynamic Isolation System [DIS], 2007).

2. To increase damping for reducing transmitted structural shear force and displacement (Figure 1.3).

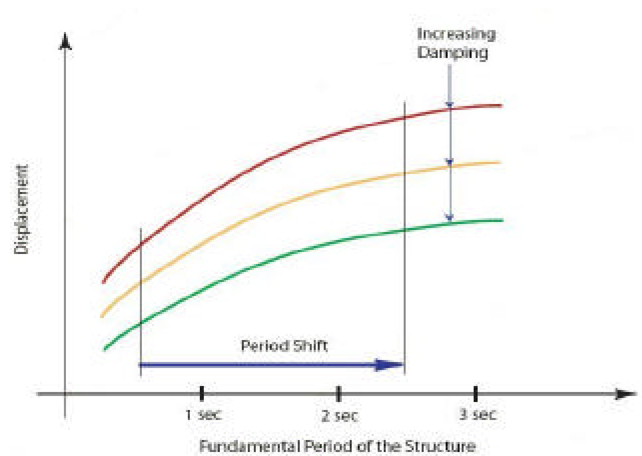


Figure 1.3: Increasing damping (Dynamic Isolation System [DIS], 2007).

3. Building remains operational after earthquake (Buildings can be used after earthquake).
4. The first dynamic mode of structure happens in isolation system as deformation. The higher modes that will produce deformation in the structure don't participate in the motion. On the other hand, if there is high energy in the ground motion at the higher frequencies, this energy cannot be transmitted to the structure (National Information Service for Earthquake Engineering, 1998).
5. The main concept of seismic resistance is to minimize drift and acceleration. Internal story drift is reduced by rigid structure. However, this method increases floor acceleration in the structure. On the other hand, reducing floor acceleration is achieved in the flexible structures that increase internal story drifts. Thus these two concepts are in contrast with each other. Consequently the only and best method to reduce simultaneously inters story drift and acceleration is using isolators. In the isolated structures, flexibility is provided in the isolator while above structure remind rigid (Naeim and J.M. Kelly, 1999).
6. In none isolated building, when it experiences earthquake, deformation happens in the building that are the primary cause of earthquake damage. However, in the isolated structures the shape of building remains rectangular (Figure 1.4). So, acceleration and inertia forces are reduced. Acceleration decreases due to the structural shifted period. Generally, structures with longer periods tend to reduce acceleration, while those with shorter periods tend to amplify acceleration (Multidisciplinary Center for Earthquake Engineering Research(MCEER), 2008).

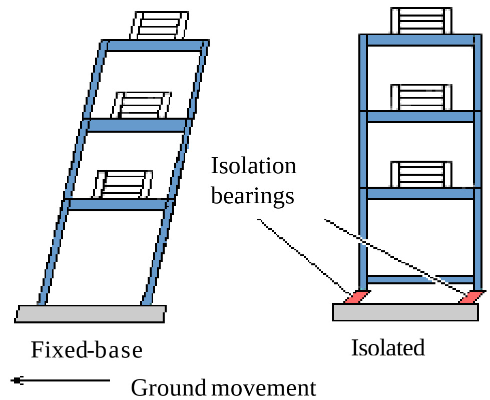


Figure 1.4: Performance of base isolated and fixed base building (Jacobs, 2008).

7. If the natural frequency of the building can be changed to frequency that does not coincide with that of earthquake, the building is less likely to fail. This is done by isolators in the structure. Meanwhile the fundamental frequency depends on height, stiffness of structure and etc. (National Information Service for Earthquake Engineering [NISEE], 1998).
8. Application of base isolation to the buildings provides better performing structures that remains essentially elastic during the large earthquakes. However, for conventional code design, fixed base ones provide minimum level of performance. Thus, the building does not collapse. They don't protect building against to structural and non-structural damages.

1.3 Background

Nowadays, the number of seismic isolated buildings increases around the world. Fortunately, public consciousness about this method increases more and more. Kobe Earthquake (happened in January 17, 1995) was the significant challenge in the number of base isolated buildings in Japan. According to the statistics, 3 years before Kobe Earthquake happened, the number of base isolated buildings was only 15. On the other hand, 3 years after the earthquake, this number increases to about 550 buildings dramatically as shown in Figure 1.5 (Clark et al., 1999).

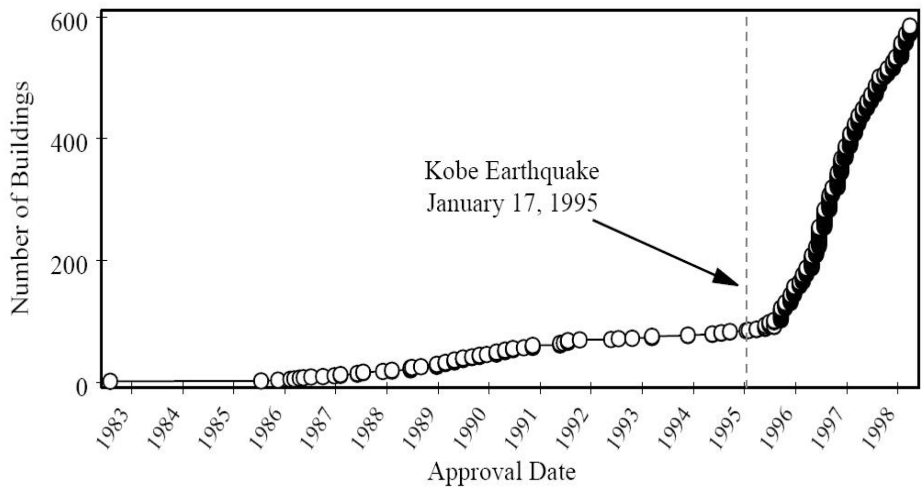


Figure 1.5: Number of isolation buildings before and after Kobe earthquake (Clark et al., 1999).

The first seismic isolators were rubber bearings (neoprene) that have been used since many years ago in the bridge construction between piers and girders. This allows girders to move freely under thermal movements. The concept of rubber seismic isolators was developed with discovering the Natural Rubber Bearings.

The main problem of Natural Rubber Bearings is related to support low-weight buildings. In these buildings, due to the low weight of the structure and high stiffness of the isolators, displacement is reduced. Therefore, higher acceleration is transmitted to the structure. If stiffness of the isolator is reduced with increasing thickness of the rubber layers, isolator will fail due to the P-? effects. Therefore, civil engineers decide to find some other methods for this type of buildings, which will be explained in Chapter 2.

1.4 Previous Work Done

This study was commenced by meticulously evaluation of previous scholarly discussions by many thinkers who deal with the studies related to some innovative isolator characteristics, isolated methods and etc. Information obtained from these evaluations, assisted immensely in the understanding of the subject matter.

The first part of this literature review is about the recent works dealing with the general consideration of isolated systems. Hong-Nan and Xiang-Xiang (2006) indicated the restriction of height-to-width ratio for isolated buildings with rubber bearings under different conditions. The factors that affected this ratio are: “site soil condition, seismic ground motion, period of the isolation system and layout of isolators”. The researchers confirmed that the soft soils produce small height-to-width ratios under different acceleration (Figure 1.6). In addition, the period of the isolated building affects this ratio significantly (Figure 1.7). This research is being completed with the latest conclusions that the stiffness of the structure influences this ratio negligibility.

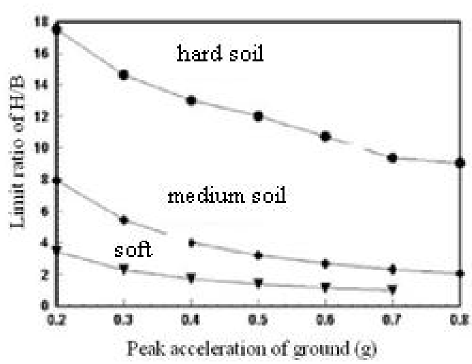


Figure 1.6: Effect of the soil condition isolation and acceleration (Hong Na and Xiang, 2006).

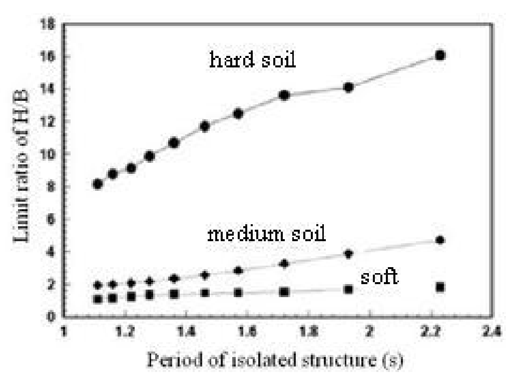


Figure 1.7: Effect of the period in isolation system (Hong Nan and Xiang, 2006).

Nagarajaiah and Buckle (2002) highlighted the stability of the elastomeric isolators under critical load test. Although flexibility of the isolator can be increased (larger periods are achieved) by rising thickness of the rubber, but this increment causes an existing unstable isolation system under large displacements. The authors proved this hypothesis by carrying out vertical load test on 6 bearings with different numbers of rubber layers (3, 4, and 8) and different dimensions (5 and 10 inch). In the end, it was concluded that the isolators with 8 layers showed lower vertical loads at maximum displacement than other bearings. Burtscher, Dorfmann and Bergmeister (1998) investigated mechanical characteristics of High Damping Rubber Bearings. The most remarkable conclusion of this research is related to the effects of the temperature on the isolator's characteristics, especially in rubber bearings whose motion of chains is strongly affected by the thermal energy in the system. According to the their test results, "modulus for -20°C is 2.2 times the modulus at +20°C and the damping at -20°C is 1.3 times the damping at +20°C. Further, moduli ratio between -20°C and +40°C is achieved by 2.5 This is very high, when we consider that this temperature range occurs during summer and winter". Higashino et al. reported on the durability of the low-friction slider bearing. The writers carried out different tests to clarify changes of a static and dynamic friction coefficient with respect to the passing time. According to the accelerated aging test, the static friction coefficient is raised by 0.1 power of the time while the dynamic friction coefficient does not change. At the constant vertical load test after 60 hours, there was no noticeable change in the dynamic friction coefficient. Finally, after 6 months, constant values were derived for the static and dynamic friction coefficient. Matsagar and Jangid (2004) investigated the response of the multi-story structure when an isolation system is modeled by neither bi-linear hysteric model nor the code specified equivalent linear elastic-viscous damping model. Furthermore, the effects of the isolator's hysteresis curve's shape and flexibility of the superstructure have been studied. The writers stated that code specified equivalent linear elastic-viscous damping model under-predicts acceleration of the superstructure and over-predicts bearing displacement in comparison to the bi-linear hysteric model. On the other hand, there is significant change in the frequency content of the superstructure acceleration. In addition, the authors concluded that the shapes and parameters of the bi-linear hysteric loop affect response of the structure significantly. At the end, the flexibility of the superstructure influences structural acceleration with no change in isolator displacement.

Until now, many methods for isolating structures have been proposed. Some of them are accepted as useful and practical among the various methods. In fact, most of the isolators represent some advantages and disadvantages. Therefore, engineers attempt to improve upon the disadvantages in the isolators by producing synthetic isolation systems. This literature continues with recent works on these combination systems. Constantinou et al. (1991) presented a synthetic isolation system, which consists of Teflon disc bearings and helical steel springs. In this method, sliding bearings carry vertical loads while helical springs prove effective in controlling bearing displacement when deformed in shear (springs carry no vertical loads). Three different shaking table tests were carried out by the authors on a six story model building: one of them without springs and the others with differing total spring stiffness. Their results of the tests demonstrated the effectiveness of this isolation system for reducing acceleration, shear force and etc. In the different groups, with and without springs, the same structural responses were derived for the same inputs. So springs served only to control bearings displacements and were not affected by structural responses. It should be noted, however, that they represent excessive permanent displacement at low shear forces. Furthermore, the same conclusions were made for springs with different stiffness. Despite this, the biggest disadvantage of this system is related to the large floor transmitted acceleration. Barga and Laterza (2004) proposed a new isolation system which includes Sliding Bearings for isolating and High Damping Rubber Bearings to provide restoring forces. This system is called a hybrid isolation system. In this system, sliders are mounted on the top of the rubber bearing. The authors carried out tests on two actual isolated buildings. The first one was isolated by 28 High Damping Rubber Bearings while in the other, 12 High Damping Rubber Bearings and 16 Sliders were used as isolation system. The results of the test illustrated that base and top floor acceleration of rubber bearing's structure were clearly higher than the hybrid structure. They go on to suggest, however, that the hybrid isolation system shows a longer period when compared to the rubber bearing. The authors continued their conclusions by expressing some advantages of the hybrid isolation system. They concluded that this system can be effective in overcoming typical design problems involved with rubber bearings, such as instability, low stiffness for low-rise buildings, vertical dimensions in the seismic gap and etc. Adding to what had already been concluded; this system can be designed according to the separate stiffness and damping, especially when Natural Rubber Bearings are used instead of High Damping Rubber Bearings. Following this isolation system, which is suitable for low-rise buildings, Adan and Sunaryati (2006) offered another suitable isolator for overcoming to the problem of these types of buildings. In their study, they suggested a circular elastomeric hollow rubber bearing in order to reduce horizontal stiffness of the isolation system. Researchers used Finite Element Method for conventional and elastomeric hollow rubber bearing to show the efficiency of this system. They concluded that the hollow isolators produce a lower horizontal stiffness than the conventional ones while the difference between their vertical stiffness is negligible.

Although there are various advantages to the isolators, they cannot provide high torsion capacity specially rubber bearings. According to the statistics, 42% of the collapses that happened during a Mexico City Earthquake in 1985 corresponded to the excessive torsion of asymmetric buildings (Colunga and Cruz, 2006). It is, therefore, better to minimize the effects of the torsion in the buildings. Hussain and Satari (2007) suggested using bracing systems for the bays located at the corner of the building, where lower gravity loads are presented and overturning forces cause increment uplift. This system reduced tension forces in surrounded isolators significantly. Calunga and Cruz (2006) studied torsion amplification in an asymmetric base isolated building. In their research, they concluded that asymmetric buildings with mass eccentricities produce higher torsion amplification than stiffness eccentricities. These amplifications are affected by a period range for different ground motions. Adding to what has already been done, they stated that when the ratio of the isolated building's period to the fixed base, one's period is greater than 8 (T_I / T_S = 8), torsion amplification for systems with mass eccentricities decreases significantly. For stiffness eccentricities systems, this amplification is negligible. In the end, the authors stated that asymmetric buildings with T_I / T_S < 2 are not suitable for isolating because they are imposed by very high torsion amplification. Ryan and Chopra (2006) considered the effects of the rocking and torsion on symmetric and asymmetric isolated buildings. They expressed the view that accidental torsion in symmetric buildings exists due to the axial load's effects in isolated bearings, but these accidental torsions are insignificant. Unlike lateral deformation, the variation of axial forces in isolators is directly related to the rocking effects. The maximum and minimum axial forces in symmetric systems are influenced by rocking to vertical frequency ratio but are not independent of other rocking-related parameters such as the building slender ratio, the plan aspect ratio or distribution of bearings over the plan.

In recent years, the performance of base isolated buildings under near-fault excitations is being significantly considered in order to take into account the effect of strong ground motions on isolation devices. Providakis (2007) highlighted the performance of a base-isolated steel-composite structure under near-fault excitation. In this research, the writer analyzed three different types of buildings. These consisted of a fixed base and two isolated buildings (with and without bracing) by using the static non-linear (pushover) analysis method. In the isolated buildings, different types of Lead Rubber Bearings were designed according to the same period range 1.5 to 2.5 seconds. The writer clarified that the isolated buildings reached the target displacement at the lower base shear of the fixed-base model but, unfortunately they increased first floor drift. Following this, the isolated building without a bracing system illustrated performance independent of the isolator properties while a bracing system affected the performance of the building seriously and reducing story drift by a factor of more than 1.5. Adding to what has already done, Providakis (2007), in another research presented the effects of lead rubber bearings and supplemental viscous dampers on seismic isolated buildings under near fault and far fault excitations. In near fault sites, isolators experience high displacement that bring these devices to the critical working condition. Under these conditions, increasing in damping of isolation system can be considered to be a useful solution. In this research the author examined symmetric and asymmetric buildings that, similar to the previous research, were isolated by three different types of Lead Rubber Bearings and viscous dampers to increase damping of the isolation system. It is expressed that the introduction of additional dampener in the isolation system controls isolator displacement and performs better in near fault excitation. Unfortunately, in far fault sites, although the isolator's displacement is reduced significantly, the response of the structure increases, especially inter story drifts. Finally, it should be noted that these additional damping in isolation systems show reliable performance for strong ground motions whereas they are ineffective under moderate and strong ground motions, in the far fault sites. This author completed his research in 2008 in another study, adding one more type of isolator (Friction Pendulum System). He found that in the near fault sites the additional damping to the isolation system needs to be controlled carefully (not more than 20 %) to minimize internal deformation for moderate ground shakings. Hussain and Satari, in 2007, also arrived at the same results.

Currently, the cost of isolated buildings compared to conventional code design, fixed base ones has become a worldwide topic of debate. Most of these arguments concern the optimization process of base isolated buildings. Optimum design of a base isolated building is related to the optimization of either the base isolation system or superstructure. But the most effective one corresponds to the optimization process that considers isolation system and superstructure simultaneously. Zou (2008) proposed a numerical optimization process for a base isolated concrete structure. In this optimization process, the author optimized the cost of the isolated building in the terms of inter story drift and isolator displacement for the superstructure and isolation system respectively. It should be noted, however, that the cost of the isolator depends on the provided effective stiffness. The lower effective stiffness produces a cheaper isolator. Moreover, the writer compares the cost of two concrete frames, which were isolated by linear and nonlinear isolators. A dynamic linear-elastic response spectrum analysis was used for analyzing these structures. It was found that the total cost of the linear base isolated structure was higher than the nonlinear one while linear isolators represented lower effective stiffness than nonlinear ones. On the other hand, low damping which is provided by linear isolators is the main reason of this increment in cost. Iemura et al. (2006) presented optimization design of a resilient isolation system for protecting equipments. This system is optimized according to the minimum displacement and keeping maximum acceleration under allowable level. Two different types of earthquake, moderate (T1) and strong (T2) were used at the shaking table test. It was confirmed that the optimum friction coefficient of the isolator showed the same linear behavior for two different types of earthquakes and rising with increasing at the level of allowable acceleration (Figure 1.8). In addition, the optimum period of the resilient isolation system increases for a higher value of ground acceleration. On the other hand, increasing the allowable acceleration, allows a shorter optimum period to be achieved (Figure 1.9).

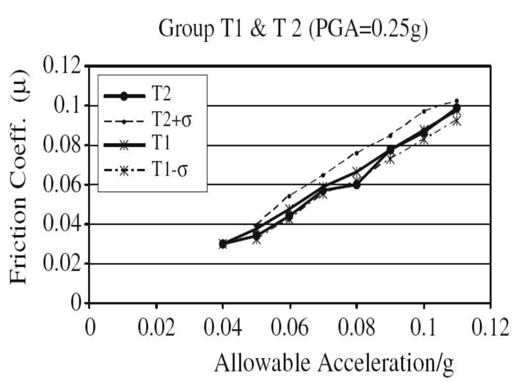


Figure 1.8: Optimum friction coefficient of resilient system (Iemura et al., 2006).

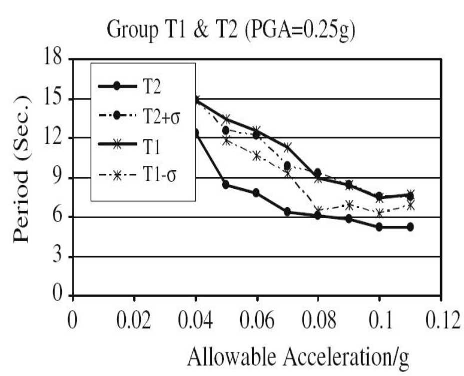


Figure 1.9: Optimum period of resilient system (Iemura et al., 2006).

Jangid (2006) presented the optimum Lead Rubber Bearing for near fault motions. In this research, the author investigated the optimum parameters of the LRB isolators to minimize seismic response of the proposed structure under near fault excitation. The results of the analysis on a five story building that was isolated by two LRB with different yield strengths clarified that under near fault excitations, increasing the yield strength reduces isolator displacement significantly, while its influence on transmitted acceleration is negligible. In addition, it is shown that there is optimum yield strength for LRBs that minimizes transmitted acceleration to the superstructure in the near fault sites. Consequently, the yield strength in the range of 10% - 15% total weight of the structure is the optimum value that minimizes structural acceleration and bearing displacement simultaneously. Adding to what has already been done, Jangid (1999) represented an optimum friction coefficient in sliding systems to minimize transmitted acceleration to the structure. In this study, the optimum friction coefficient is achieved under different factors such as period and damping of superstructure, damping ratio of isolation system, period of isolation system and etc. It is concluded that for every structure, there is an optimum friction coefficient that minimizes structural transmitted acceleration. In addition, the optimum friction coefficient of sliding system reduces with the increase in the period and damping of isolation system respectively. Further, the writer stated that the optimum friction coefficient decreases or increases under rising of structural damping. In another research this author (2004) illustrated the optimization of friction coefficient in Friction Pendulum Systems under near fault excitation. It is shown that isolators with a low friction coefficient are subjected to the high displacement. In fact, this displacement can be reduced by increasing the friction coefficient without influencing structural acceleration. It can be concluded that, friction coefficients between 0.05 - 0.15 minimize structural acceleration and bearing displacement simultaneously.

1.5 Objectives and Scopes

According to the previous works done, first of all the thesis focuses on the general considerations of seismic isolated buildings which are world-wide applications of seismic isolated buildings, different type of isolators, the installation process of them, their mechanical characteristics, cost of isolated buildings and location. Finally, the design methods for different types of isolators are discussed to present simple, concise, and practical information and principles required by practitioners in seismic isolated buildings. Furthermore, three reinforced concrete buildings with different height (three, six and nine story) are analyzed by three types of isolators (Lead Rubber Bearing, High Damping Rubber Bearing and Friction Pendulum Systems) to come up with the optimum case according to the seismic demand. Microsoft Excel spreadsheet is utilized in order to design Lead Rubber Bearings and High Damping Rubber Bearings. Finally, a three story fixed-base and optimum isolated structures are analyzed in order to clarify differences between the performances of these two types of structures. Moreover to what have already been done, these two buildings are designed and their materials are compared to reveal the differences.

The main objective of this research is to present the optimum isolator in order to save materials in the building by comparing transmitted acceleration to structure, horizontal displacement and seismic coefficient for three types of isolators in different buildings. Furthermore, to clarify the additional cost of base isolated building comparing to its conventional fixed base one.

1.6 Organization

The thesis contains the following chapters:

Chapter 1: The introduction part contains a short background that identifying the aim and scope of the thesis. It describes about the research method and outline of the thesis.

Chapter 2: This chapter provides an overview of the different types of isolators.

Chapter 3: In this chapter, world-wide applications of the seismic isolated buildings are discussed.

Chapter 4 presents the practical applications of the seismic isolators on the new and retrofit buildings. These buildings are reinforced concrete and steel frame structures.

In chapter 5, mechanical characteristic, location of the isolators and cost of the isolated buildings are considered.

Chapter 6 presents about the code requirements for designing a seismic isolated building. Furthermore, design procedures for three different types of isolators are discussed.

In chapter 7, three buildings with different height and different isolator types are analyzed. At the end, one of them is designed and its materials are compared with conventional fixed base one.

In chapter 8, on the base of the above studies, conclusion and remarks are defined Besides, suggestions are given for the future researches.

CHAPTER 2

ISOLATOR DEVICES AND SYSTEMS

2.1 Introduction

Base isolation is now a new technology and is used in many countries. In fact, there are a number of acceptable isolation systems which are proposed and patented each year. Unfortunately, some of these new isolation systems seem to be impractical, but the number continues to increase year by year.

Most of the popular isolation systems that are used today incorporate either Elastomeric Bearings (natural rubber or neoprene) or Slider Bearings (sliding surface being Teflon and stainless steel). Sometimes, different isolators are combined to provide some ideal isolation systems (Naeim and J.M. Kelly, 1999).

This chapter will explain as many of the available systems as possible. It should be noted, however, that the number of seismic isolation systems increases year by year, it is possible that some of them may be not discussed.

2.2 Foundation Isolation Systems

Smooth synthetic materials are situated under the foundation to protect buildings against earthquakes by absorbing energy in the course of sliding. This method can be used instead of rubber isolators for low and medium weight buildings.

Smooth synthetic materials are placed at two situations:

1. As a placement of the liner under the foundation of a structure. This approach is called Foundation Isolation (Figure 2.1).
2. Smooth synthetic materials are mixed in the soil as its properties. This method is called Soil Isolation.

Generally, the materials that are used for Foundation Isolation System should provide obligations including:

1. They should provide small friction coefficient during sliding to reduce transmitted acceleration.
2. For reducing sliding action due to the non seismic loads (wind load), the static friction coefficient should be slightly larger than the dynamic friction coefficient.
3. They should be resistant against environmental conditions and long term creep effects.
4. For protecting the structure and its content, these materials should induce minimal displacement during an earthquake (Yegian and Kadaka, 2004).

2.2.1 Some Methods for Isolation System

Yegian et al., (2004) suggested a Foundation Isolation method using a variety of synthetic materials to aid in the discovery of a smoother liner for applying in the seismic isolation of structures (Figure 2.1). They used cyclic and shaking table tests on models to determine the best materials for Foundation Isolation. These materials consist of geotextile high density polyethylenc (HDPE), polypropylene (PTFE), ultra molecular weight polyethylene and geotextile TIVAR 88-2.

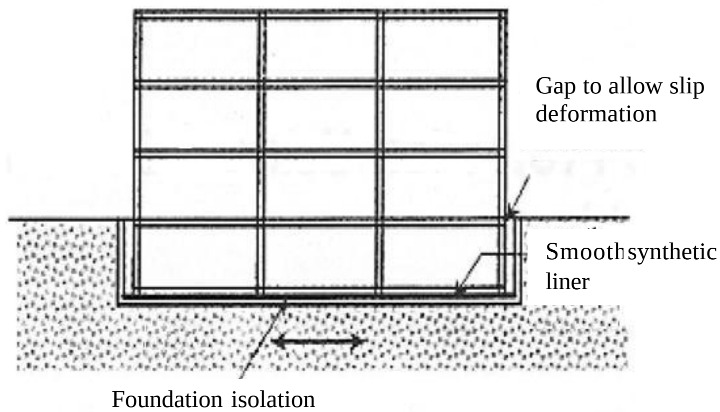


Figure 2.1: Foundation isolation by smooth synthetic materials (Yegian et al., 2004).

The results of the test showed that geotextile over an ultrahigh molecular weight Polyethylene Liner presented the ideal interface friction coefficient, making them suitable for Foundation Isolation. The static friction coefficient and the dynamic one are about 0.11 and 0.08 respectively.

The geometry of smooth synthetic liners is one of the most important factors that influence structural seismic behavior. This topic was investigated by Georgarakos et al., (2005). In this research, four different geometries were recommended and tested to find the optimal one. These four possible cases are shown at below figure.

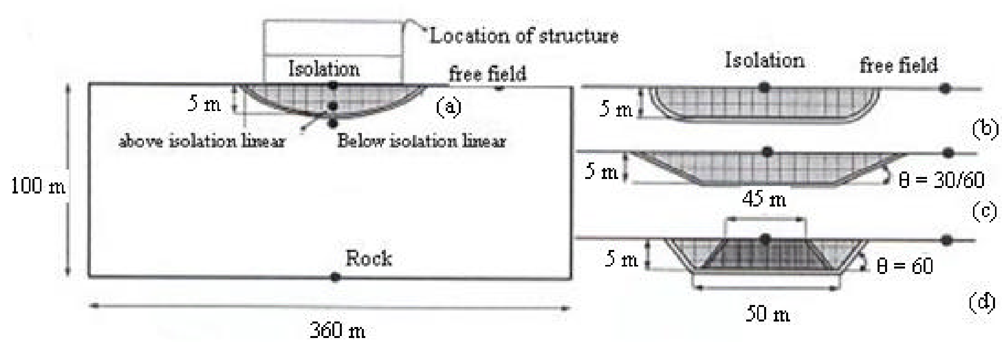
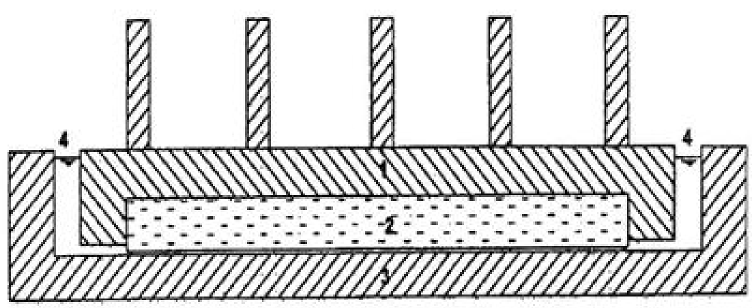


Figure 2.2: In-soil isolation systems: a) cylindrical liner geometry, b) tub liner geometry, c) trapezoidal liner geometry, d) compound trapezoidal liner geometry (Georgarakos et al., 2005).

Summarizing the results of all dynamic analysis, the most effective ones are cylindrical and the compound trapezoidal. They produce satisfactory results in reduction transmitted acceleration.

Another method for isolation system is proposed by Doudoumis et al., (2002). They used low shear resistance, artificial soil layers below the foundation, which allow the building to slip during severe ground motions (Figure 2.3). These soil layers consist of natural materials. Granular products of rocks (talc, chlorite, serpentine), high plasticity clay or a combination of them are cases in point. These materials produce low shear resistance and high strength under compression. The lower shear resistance can be provided by either combining these materials with wet bentonite which produce lubrication properties or subjecting these layers to the water as shown in the figure below.



1: Foundation 2: Soil Layer 3: Concrete Slab 4: Water Level

Figure 2.3: The basic concepts of interposing an artificial soil layer (Doudoumis et al., 2002).

The construction process of this method is expensive and problematic as well. To provide a satisfactory coefficient of friction, the water level surface should be checked frequently. These materials provide a coefficient of friction equal to 0.2 that, when compared to the other methods, is higher and causes more transmitted acceleration to the structure.

Xiao et al., (2004) looked for improving the Foundation Isolation System by introducing a simple, low cost isolation system that can be used at the time of construction or re-construction. The coefficient of friction in this sliding isolation system controls the transmitted base shear to the structure. The smoother sliding isolation systems produce the lower transmitted acceleration to the superstructure. In their project, they sought to discover the best material for a sliding system by testing five different materials. These materials consisted of sand, lighting ridge pebble, polypropylene, PVC sheet and polythene membrane. The shaking table experiment was used for testing these materials. Finally, the results of the test are summarized in the table below.

Table 2.1: Isolation level (Xiao et al., 2004).

Material	Isolation level (g)
Pebble(6-8 mm)	0.2
Polythene membrane	0.18
Polypropylene sheet (0.8 mm)	0.15
Polyvinyl chloride sheet (1.0 mm)	0.10

Based on the experimental results, Figure 2.4 illustrates the proposed construction plan by using the foundation isolation system.

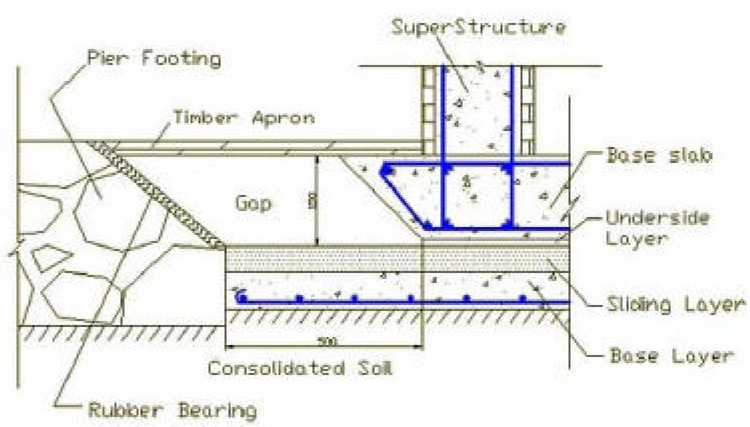


Figure 2.4: Proposed construction model (Xiao et al., 2004).

2.2.2 Advantages and Disadvantages of Foundation Isolation System

Since the sliding surface is installed below a concrete slab, the structure is stable when subjected to the wind loads. This is considered to be the best advantage of this system.

Against its advantage, the Foundation Isolation System produces some disadvantages in that:

1. They cannot provide restoring forces. In this case, according to the UBC 97 code, the seismic gap should be designed for three times design displacement as accepted term. Therefore this method is practical only for buildings that are surrounded by a sufficient area.
2. This method is more suitable for small areas and low-weight buildings. In other cases, due to the high-weight of the building, a thicker and larger area of base slab is needed, which makes this system uneconomical (Thurston, 2006).
3. Long term creep and the environmental conditions that usually occur in this isolation system are considered as the other disadvantages of this technique.
4. In most of the proposed Foundation Isolation Systems, water is used to fill the seismic gaps. If the building is constructed with brick veneer, masonry or timber-framed walls, the moisture ingress at the base of the wall is likely to be a problem.

2.3 Roball

Roballs are the newest isolation systems that are used for isolating light-weight buildings (such as wooden buildings) with 1 ton weight per column. Roballs are expected to use as the most economical isolation systems in the near future for both light and heavy buildings (Robinson Seismic Ltd).

The main parts of these isolators consist of balls and concave surfaces. These balls are filled with materials that produce friction forces to reduce and absorb transmitted forces. The friction coefficient that these isolators provide is 0.1 approximately. The latest versions of Roballs include restoring forces. Depending on the restoring force, these devices are produced as two models. The application for the first model is to locate balls in the concave surfaces, as shown in Figure 2.5. The other is to install a number of small balls in a close-packed area (bigger ball). The top and bottom surfaces of the surrounded big ball are flat while the side's surfaces are concave (Figure 2.6). The number of the balls that are put in a close-packed area depends on the design displacement. They can be designed with 7, 13, 19 and 25 solid balls in the desire sizes (Thurston, 2006).

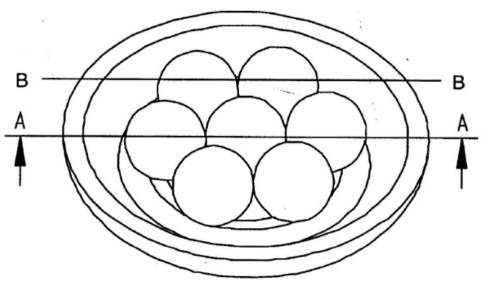


Figure 2.5: Roball with concave surface (Thurston, 2006).

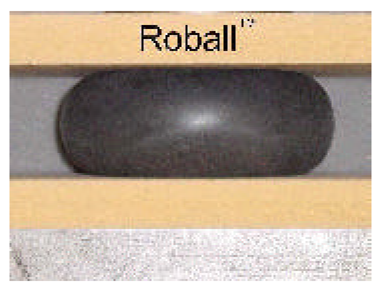


Figure 2.6: Robal in package (Robinson Seismic Ltd).

2.4 RoGliders

RoGliders are the newest generation of isolators invented in New Zealand. These types of isolators are capable of supporting light and medium vertical loads up to 1000 kN. The main concept of these isolators is a combination of rubber bearings and sliders. These isolators consist of two main components: cylindrical segment and two stainless steel plates. Two thin layers of Teflon (PTFE) are attached to the top and bottom of the cylindrical shape segments that slide on stainless steel plates.

Depending on the weight of the building, RoGliders are produced as two basic types:

The first type is a cylindrical shape segment fixed to the bottom plate, which can slide on top plates. Restoring force in these isolators is provided by covering the sides of the isolator with rubber skirts. Using rubber skirts for providing restoring forces imposes higher stiffness to the isolator and thus attracts high seismic forces. The results of the tests show that for the low seismic displacements, the isolator can be stable without rubber skirts. However, for high seismic zones, restoring force must be provided. Figures 2.7 and Figure 2.8 show RoGlider with and without rubber skirt (Thurston, 2006).

Figure 2.7: RoGlider with rubber skirt (Thurston, 2006).

Figure 2.8: RoGlider without rubber skirt (Thurston, 2006).

RoGliders without rubber skirts fail at high displacements due to the P-? effects as shown in Figure 2.9.

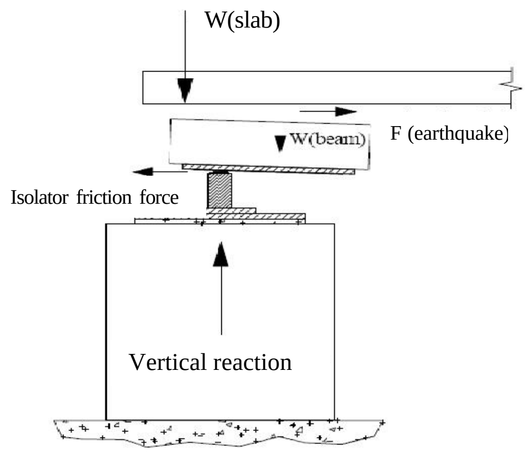


Figure 2.9: Failure of RoGlider isolation system (Thurston, 2006).

As can be seen from the illustration, the system fails when the summation of moments that is produced by weight of the beam and earthquake force become greater than the summation of moments related to the weight of the slab and isolator friction force. The best solution for this problem is to install an isolator invert. In this situation, the fixed part of the isolator should be connected to the beam while sliding part is attached to the foundation (Thurston, 2006). This type of RoGliders usually is used in light weight buildings.

The second types are the double acting RoGliders (Figure 2.10). These isolators produce restoring force. Generally, they consist of two stainless steel plates and a slider segment. Two layers of Teflon are attached to the top and bottom of slider that reduce the friction coefficient. The slider part can slide on both stainless steel plates. Two rubber members provide restoring forces that are connected to the slider segment and stainless steel plates. When the slider glides on both the top and bottom plates during an earthquake, one part of the rubber segment undergoes compression while the other part experiences tension. This process causes existing restoring force in the isolator. These models of RoGliders have an effective coefficient of friction about 11% approximately and are suitable for medium and high weight buildings (Robinson et al., 2004).

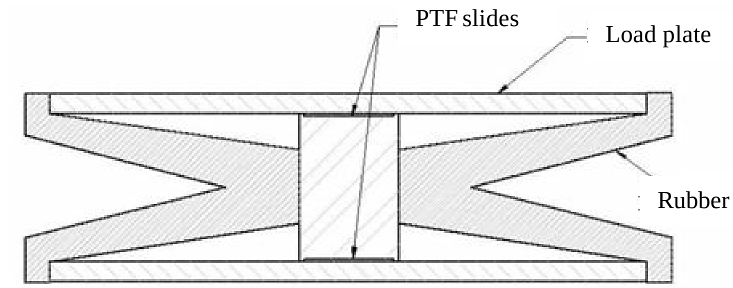


Figure 2.10: Double acting RoGlider section (Robinson et al., 2004).

2.5 Rocking Column

Tall and slender structures with top-heavy parts are subjected to the more over-turning moments than ordinary structures. These moments produce a high tension in the connection between the columns and the foundation. Providing this tension capacity in the foundation is usually expensive. Hence, the best method is to allow columns to roll on the foundation (Naeim and J.M. Kelly, 1999).

The potential application of using Rocking systems has been used since the 1960s in New Zealand. Studies show that Rocking systems have influence on the length of the structural period, while not affecting the period on higher modes (Ma and Khan, 2008).

In this isolation method, the top and bottom of the base columns are performed similar to spherical surfaces (provide restoring forces) that allow columns to revolve under existing ground motions. Because of the negligible damping that is produced by this isolation system, high acceleration is transmitted to the structure. Therefore, this isolation system is completed by adding energy dissipated devices at the base and locations that are prone to the uplift effects (Pollino and Bruneau, 2004). Despite this rocking isolation systems are used rarely in the world due to the complexity of these systems and need more investigations.

The south Rangitikei Bridge, completed in 1981, is one of the modern and rare structures with the Rocking isolation system in New Zealand. The dissipated energy devices are used in this structure to reduce transmitted energy and control uplift. The results of tests show that the natural period of this Rocking structure varies from 1.73 second to 4.33 second, depending on the lateral displacement. More detail is shown in Figure 2.11 and Figure 2.12 (Ma and Khan, 2008).

Figure 2.11: The pier base as built (Ma and Khan, 2008).

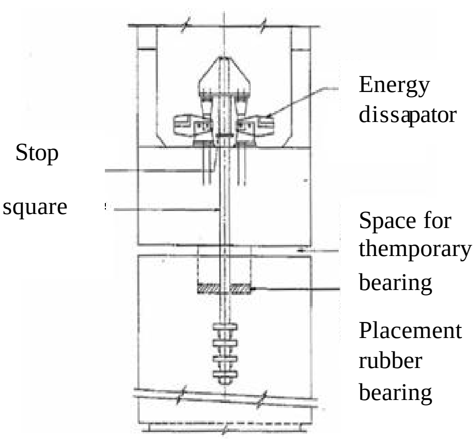


Figure 2.12: Schematic of the base detail (Ma and Khan, 2008).

2.6 Sleeved-Pile Isolation System

Sleeved-pile isolation systems are one of the oldest isolation systems in the world. In locations where, due to the soft soil layers, using pile foundations are necessary, it can be a good idea to use sleeved-piles for providing horizontal flexibility required for an isolation system. Therefore, in this case, using this isolation system is economical. For other types of soil, it’s better to use other isolation system methods.

The components of this simple isolation system consist of flexible piles which are enclosed in tubes with appropriate seismic gap for clearance. The value of damping that is provided by this isolation system is insignificant. Therefore, using sleeve-pile isolation systems should be followed by installing dissipated energy devices in the structure. These dampers are installed at the base of the structure (Figure 2.13) (Bozorgnia and Bertero, 2004).

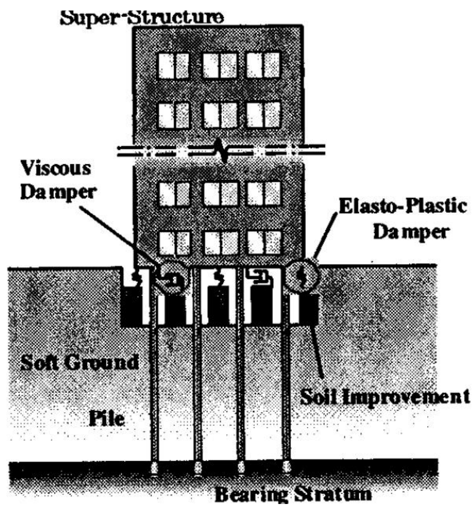


Figure 2.13: Section of sleeve isolation system (Bozorgnia and Bertero, 2004).

The system was applied in one of the earliest isolation system projects, the Union House in Auckland, New Zealand. This building is located in a soft soil area; required piles 10 meters in length. Elastic-plastic steel plate dampers were used to provide 12% damping and the structural period equal to 2 seconds. The Randolf Langenbach House in California is another application of this isolation system. This system increased the construction cost around 3% of the total cost (Naeim and J.M. Kelly, 1999).

2.7 Elastomeric Base Isolation Systems

The first application of elastomeric rubber isolations was in bridge construction. For many years, rubber bearings have been used in bridges at the connection of piers and girders to reduce transmitted stress due to thermal movements. These elastomeric bearings are made of neoprene (producing high vertical stiffness) or natural rubbers.

The first use of Natural Rubber Bearings for earthquake protection was in 1969 at the Pestalozzi School in Skopje, Macedonia. These bearings were made of large rubber blocks that were compressed by about 25% under vertical loads. The vertical stiffness of those isolators was only slightly more than the horizontal stiffness. Because of this, a number of steel plates are inserted in these types of isolators to reduce displacement and increase vertical stiffness while they don't affect horizontal stiffness (Bruce, 2007). The internal plates, called shim, provide a high value of vertical stiffness which is several hundred times the horizontal stiffness (Bozorgnia and Bertero, 2004). Depending on the provided damping, these isolators are produced at four different categories:

2.7.1 Low-Damping Natural Rubber Bearings (NRB)

Low-Damping Natural Rubber Bearings consist of two tick plates (load plate) and many thin steel shims. These steel shims are vulcanized to the layer of rubber (Figure 2.14). The vulcanization process involves a series of thermal and chemical processes under heat and pressure (Hasani, 2002).

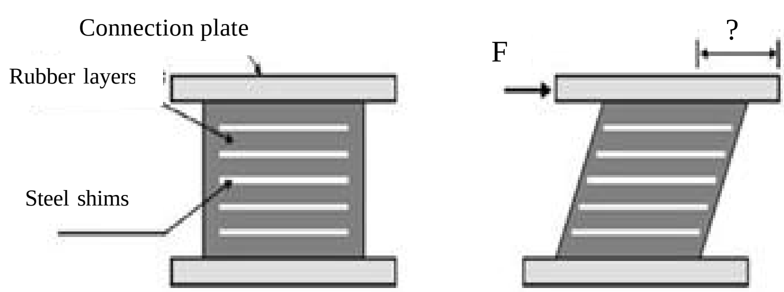


Figure 2.14: Low-damping Natural Rubber Bearing (Özden, 2006).

Under high horizontal displacement, the internal shims protect isolator from separating out by keeping top and bottom of the elastomer in the place (Jacobs, 2008). Instead of natural rubbers, these isolators can be produced in neoprene (Seismic Energy Products [SEP]). In this case, number and thickness of the shims are reduced due to the high stiffness of neoprene.

The load capacity of rubber isolators is increased by reducing the thickness of the rubber layers and increasing the thickness of the steel shims. Generally, these rubber bearings are used to provide recentering forces and horizontal flexibility in the structure. Isolation system damping can be increased by other separate components (T.E.Kelly et al.).

The application of low-damping natural rubber bearings has been widely used in Japan. In that country, because of the low damping (2-3% critical value) provided by these isolators, they combine with different kinds of energy-dissipated devices to produce an ideal isolation system. Dampers can be installed at different points of a structure. One of the methods illustrated in Figure 2.15 (Tachibana and Emeritus, 2007).

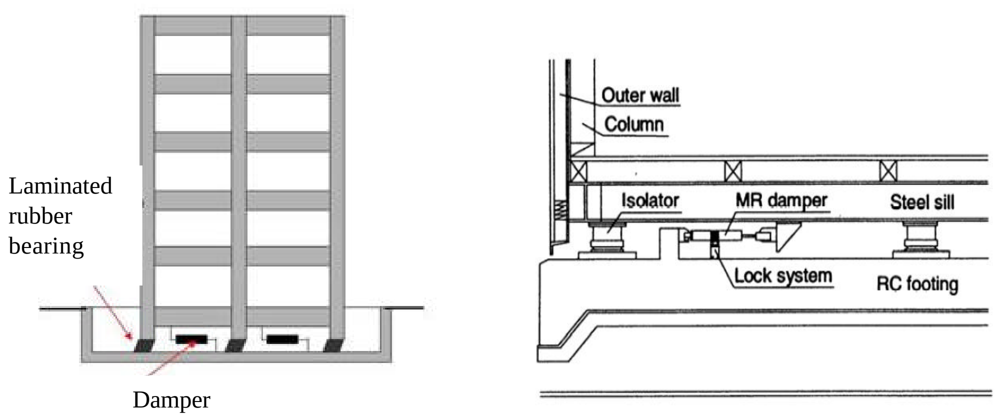


Figure 2.15: Combination of Natural Rubber Bearing and dampers (Tachibana and Emeritus, 2007).

The most advantage of Low Damping Rubber Bearings is simplicity in the manufacture process of these bearings. They are easy to model and the mechanical properties of these devices are unaffected by environmental conditions. The only disadvantage of these isolators is related to their combination with supplementary dampers. Application of dampers to the structure represents problems such as: an increased cost of isolation system; new and special connections become necessary; in some cases they need to be replaced after an earthquake and they affect higher modes of the structure (Naeim and J.M. Kelly, 1999).

2.7.2 Lead Rubber Bearings (LRB)

After Natural Rubber Bearings, Lead Rubber Bearings are most popular used in bridge construction. When applied in the structure, they should provide more flexibility and deflection control.

These isolators were invented and applied for the first time in New Zealand (Park, 2000) and now have been widely used in the United States and Japan. China joined this group recently (Fu-lin et al., 2006). These isolators consist of two thick steel plates, natural rubber layers and shims similar to Natural Rubber Bearings. The only difference between NRB and LRB lies in the method of providing damping in the isolation system. Instead of dissipated energy devices, damping is provided by adding one or more lead plugs to the isolator (Figure 2.16). The construction method of LRBs is same as NRBs, differing only in that holes are made in the rubbers and shims for inserting lead plugs. The steel plates (shims) in the rubber bearing compel the lead plugs to deform in the application of shear forces and provide damping by plastically deformation.

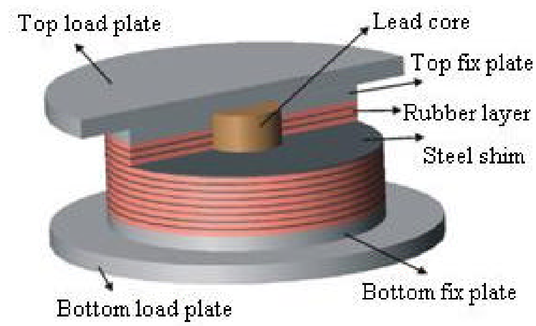


Figure 2.16: Lead Rubber Bearing.

Lead is chosen as material for providing damping in the isolator because (T.E.Kelly et al.):

1. Lead yields in shear at a comparatively low value of stress (~10 MPa).
2. When lead is plastically deformed at ambient temperature, its mechanical properties are restored by the simultaneous interrelated process of recovery and recrystallisation.
3. It is used in batteries that are widely available and are produced at a purity of 99.9 percent.

Lead Rubber Bearings are produced from 12 to 60 inch in diameter and have the capacity up to 4000 tons (Dynamic Isolation System [DIS], 2007). The main advantage of these isolators is that against other types of isolators which produce limit value of damping, they can be manufactured with a desired value of damping by increasing or decreasing the lead plugs' diameter. The only disadvantage of this isolation system is related to the inserted plugs. Higher modes in the structure are affected by these inserted plugs.

2.7.3 High Damping Rubber Bearings (HDRB)

The biggest disadvantage of NRBs and LRBs is related to the external objectives that are applied in the isolation system to provide damping. These objectives affect higher modes in structure. Therefore the best method for providing damping in the isolator is to insert damping in the rubber as its property. This alternative causes existing High Damping Rubber Bearings.

Natural rubber bearings with high damping were developed in 1982 by the Malaysian Rubber Producers Research Association (MRPRA) of United Kingdom. Extrafine carbon blocks, oils or resins and other proprietary fillers are mixed with the natural rubber as an extra additive to increase isolator damping. These materials at low shear strain show a high value of stiffness that causes stability of the structure when subjected to the wind load. At large strains, the modulus increases due to a strain crystallization process in the rubber that is accompanied by an increase in the energy dissipation (Naeim and J.M. Kelly, 1999). The manufacturing process of these isolators is the same as that for Natural Rubber Bearings.

Supper High Damping Rubber Bearings (HDRB-S) are another type of High Damping Rubber Bearings that produce 20% damping comparing to the HDRB. These natural rubbers are design to manifest both friction damping rubber molecules and viscous damping by viscous materials that exist between molecules (Kawaguchi Metal Industries Company [KMI], 2006).

All of the rubber bearings have the same stiffness in both directions. But often, in seismic protection, it is wise to have different stiffness in two in-plane directions. Burtscher at al. (1998) studied incline shims that were used in High Damping Rubber Bearings instead of flat shims (Figure 2.17). This caused higher stiffness in the direction of inclination angle while in the other direction, the isolator had the same stiffness when compared to the flat shims.

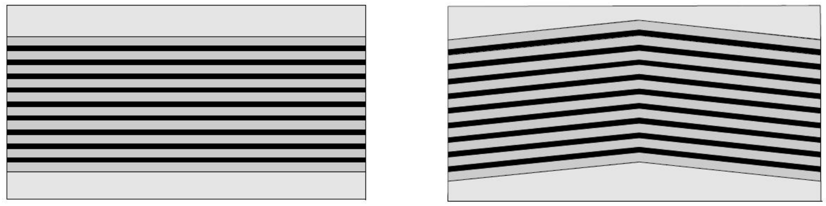


Figure 2.17: Section of the High Damping Rubber Bearing with flat and incline shims (Özden, 2006).

2.7.4 Fiber-Reinforced Elastomeric Isolators

Many applications of seismic isolated buildings have been carried out in the world. The isolators in these applications are large and heavy. They can reach a weight of 1 ton or more, which can cause an increase in the cost of construction. On the other hand, the thicker foundations are needed to perform under these isolators. This heaviness makes the production and construction process of an isolator difficult (Özden, 2006).

The steel shims that are implemented to provide vertical stiffness in the isolator, are considered the main reason for heaviness of these devices. The weight of two thick steel plates at the top and bottom of the isolators are the other factor. It is possible to reduce the weight of the isolator by replacing the steel shims with fiber materials. These materials are available with the same elastic stiffness of steel shims. However, money can be saved in the manufacturing process of these isolators by replacing the vulcanization process under pressure in the mold (done with steam heat) with microwave heating in an autoclave (J.M.Kelly and Takhirov, 2001).

2.8 Sliding Isolation Systems

The Sliding Isolation System is the simplest and one of the earliest isolation systems in the world. It was first proposed in 1909, by a medical doctor in England. He used talc to separate superstructure from foundation. This isolation was first accepted as seismic resistance strategy after Messimo-Reggio earthquake in 1908 by Italian government (Naeim and J.M. Kelly, 1999).

In this method, the building is supported by bearing pads with flat or curved surfaces. These isolators are generally composed of a slider part and two stainless steel plates at the top and bottom of slider. Layers of polytetrafluoroethlene (PTFE or Teflon) are attached between the slider and the stainless steel plates to reduce friction of the coefficient. The frictional characteristics of these isolators depend on velocity of the motion, temperature and clearness of the surface.

The biggest advantage of these isolation systems is related to the high vertical load capacity. The rubber isolators with shear modules smaller than 0.35 MPa are not able to support high vertical loads. The simple design process of the sliding isolation system is also considered an advantage. These systems do not need to be checked for maximum load capacity.

2.8.1 Flat Slider Bearings (FSB)

This simple isolation system made up of the following: a sliding pad, sliding plate, and rubber pad (Figure 2.18). The stainless steel is used for constructing sliding plates, which are coated with heat-stiffened resin. Sliding pads are made of PTFE or Teflon to reduce coefficient of friction. The rubber pads can be produced as a single layer pad or multi layer pad. The advantages of using rubber bearings are to reduce the initial stiffness of the isolator, the rotation of the slider and to reduce vertical stiffness (Higashino et al.).

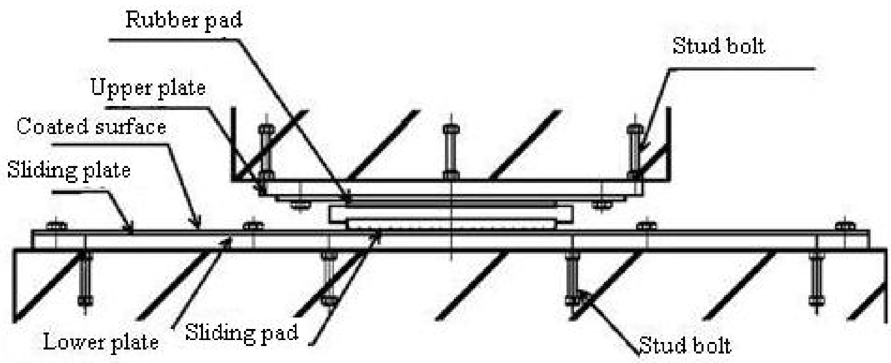


Figure 2.18: Flat Slide Bearing (Higashino et al.).

Flat Sliding Bearings cannot be used as the only isolation system in the structure because they cannot provide restoring forces and after an earthquake, permanent displacement exists. Hence, in this case, isolators should be designed for 3 times the design displacement that makes this isolation system uneconomical (UBC, 1997). Several sliding isolation systems with restoring devices have been proposed recently. The most notable of these systems are described briefly:

The simplest one is related to providing restoring forces by using oil jacks. In this system Flat Slide Bearings are used to support the weight of the building while in the seismic gap some oil jacks are installed to return the building to its original position (Figure 2.19) (Tachibana and Emeritus, 2007). Since the practicality of this method is debatable for medium and high-rise buildings, this method is not popular.

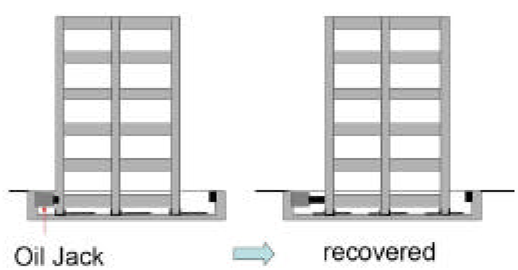


Figure 2.19: Using combination of oil jacks and Flat Slider Bearing (Tachibana and Emeritus, 2007).

The second method is to combine Flat Slider Bearings with Natural Rubber Bearings to provide restoring forces. FSBs and NRBs are compounded of two techniques. In the first, the isolators are installed parallel to each other. In this case, FSBs provide dependable resistance to the wind load by a high initial value of friction coefficient, energy dissipation and a high capacity of vertical loads. On the other hand NRBs supply restoring forces while supporting vertical loads. The second technique, called the Hybrid Isolation System is to install Flat Slider Bearings at the top of the Natural Rubber Bearings (Figure 2.20). But in the construction process of NRBs, it's better to use Neoprene, rather than Natural rubber. This offers a higher value of vertical stiffness and smaller value of displacement. In the high seismic zones, a high value of damping is required; it's a good idea to use High Damping Rubber Bearings instead of Natural Rubber Bearings or Neoprene (Braga and Laterza, 2004).

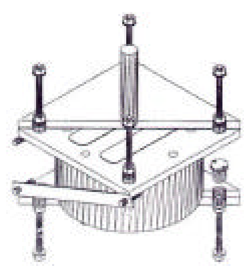


Figure 2.20: Hybrid isolation system
(Braga and Laterza, 2004).

The third method is involves the combination of Flat Sliding Bearings and helical steel springs. These devices are installed parallel to one another. In this isolation system, the weight of the building is carried by the sliders, while the steel springs provide restoring forces when deformed in shear (they carry no vertical loads). On the other hand, springs are effective in controlling the sliders displacement (Constantinou et al., 1990). This behavior is interesting for protection of no-ductile and non-structural components which don't have expected acceleration capacity. The stiffness of this isolation system is a combination of the sliders' stiffness and the springs' stiffness (Iemura et al., 2006).

The Resilient-Friction Base Isolation System is the forth method. This technique works to overcome to the two main problems of Flat Slider Bearings. The first one is restoring force and the second is the high friction coefficient of Teflon on stainless steel at a high velocity. The first problem is dealt with by inserting a rubber plug at the center of the isolator (they carry no vertical load). The second one can be solved by connecting many Flat Sliders interfaces in a single bearing. Thus high velocity between top and bottom of the isolator is shared by number of sliders (Figure 221).

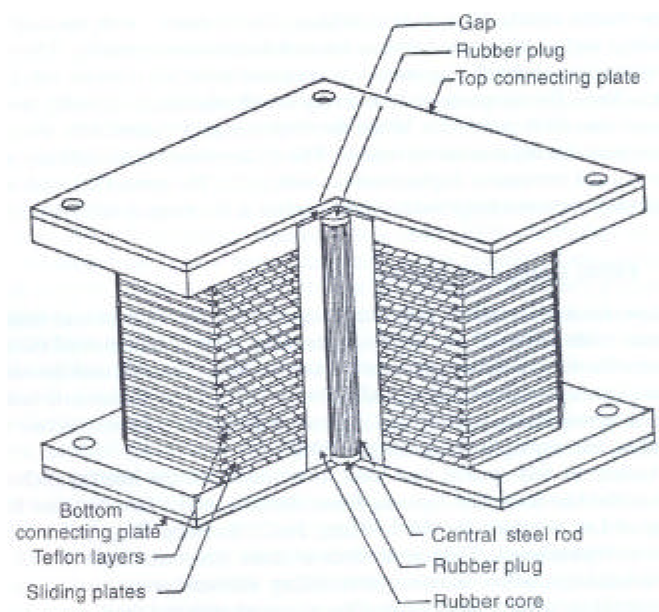


Figure 221: Resilient isolator (Naeim and J.M. Kelly, 1999).

2.8.2 Friction Pendulum Systems

Frictional Pendulum Systems (FPS) are frictional base isolation systems that produce sliding action and restoring forces. Conceptually, these systems are on the same order as Flat Slider Bearings with the difference that they can produce restoring forces. FPSs are known as the most cost-effective type of isolators. In this system, the characteristic of the pendulum is used to increase the natural period of the structure so as to transmit lower acceleration. A natural period of structure is independent of the structural mass and depends on the radius of slider, which has the advantage of controlling the structural response. Since earthquake induced displacements occur primarily in the bearings, lateral loads transmitted to the structure are greatly reduced.

Friction Pendulum Systems consist of two main types, Single pendulum Systems and Triple Pendulum Systems.

2.8.2.1 Single Pendulum Systems

Single Pendulum Bearings are the original Friction Pendulum Systems. The main parts in Single Pendulum Bearings are a slider and two concave stainless steel plates. The spherical slider is fixed to the top plate while it can slide on the bottom plate (Figure 2.22). The vertical load capacity of the bearing depends on slider's size (Earthquake Protection System [EPS]).

Figure 2.22: Single Pendulum Bearing at zero and maximum credible earthquake displacement (EPS).

A few numbers of Single Pendulum Bearings applications were used in retrofitting in Trans-European Motorway Bridge in Turkey (EPS, 2006) and the Benicia-Martinz Bridge in San Francisco. Using the Single Pendulum System instead of Rubber Bearings saved more than 30 million dollars in construction costs of the Motorway Bridge in Turkey. These bearings are 13 feet in diameter and have 40,000 pounds weight (Figure 2.23). They are the largest Friction Pendulum Bearings ever manufactured (EPS, 2003).

Figure 223: Friction Pendulum Bearing used in Benica-Martinez Bridge (EPS, 2003).

Single Pendulum Bearings have the same friction coefficient, lateral stiffness and period for all levels of earthquakes. The other problem is related to their large size. In fact, the period in these isolators depends on the isolator's radius. So, in the case of the long periods, larger isolators are required. On the other hand, thicker foundations are needed to support high-weight of these huge isolators. Consequently, this problem makes the application of the Single Pendulum Systems uneconomical. These disadvantages caused to bring about a new generation of Pendulum Systems called Triple Pendulum Bearings.

2.8.2.2 Triple Pendulum Systems

Triple Pendulum Systems consist of three independent pendulums in one bearing. Each pendulum is designed independently with different properties for different levels of earthquakes. Generally, they contain one core, two inner concave sliders and two main spherical surfaces as shown in Figure 2.24. The inner core slides on two inner concave sliders and these two parts glide on main concave surfaces.

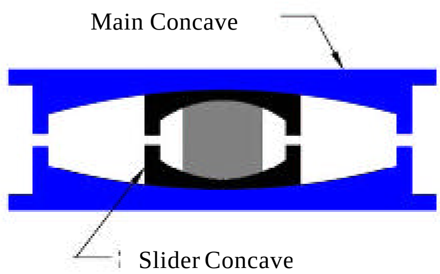


Figure 224: Cross section of Triple Pendulum Bearing component (EPS).

Figure 225: Concave and slider (EPS).

In this isolation system, each pendulum acts at different levels of earthquakes. The first pendulum is typically chosen for a service level earthquake and produces a low value of damping and a short period. The second pendulum is designed to minimize structural shear forces by increasing damping and structural period during Design Base Earthquake. This saves money in construction costs. The third pendulum is selected to minimize bearing displacement by increasing friction and lateral stiffness under Maximum Credible Earthquake. This aids to reduce bearings' dimension, save bearings cost and reduces the seismic gap(Figure 2.26) (EPS).

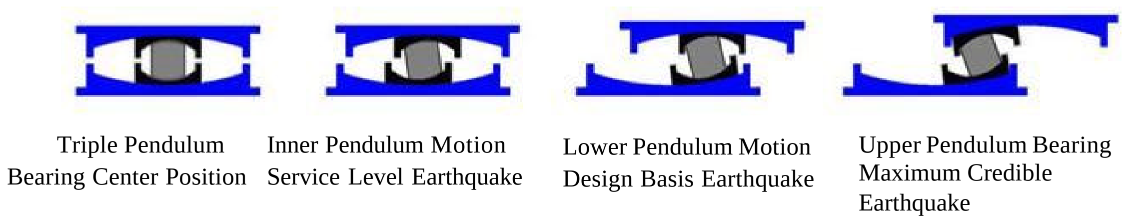


Figure 226: Triple Pendulum Bearing under different earthquakes (EPS).

Furthermore, for these bearings, three different radiuses and three friction coefficients are selected in order to minimize the response to the different earthquake levels. In fact, the period is chosen by selecting the radius of the bearing (independent of the structural mass), damping by selecting the appropriate friction coefficient and finally the vertical load capacity by choosing the core size.

Double Friction Pendulum Bearings is another type of pendulum system. They were first applied in Japan on a limited number of buildings. They contain two independent pendulums in one bearing which act in a similar fashion to the triple pendulums but missing the first pendulum. Constantinou (2004) considered the mechanical characteristics of Double Friction Pendulum Systems.

2.9 Advantages and Disadvantages of Different Isolation Systems

Advantages and disadvantages for different types of isolators are presented in the table below.

Table 2.2: Advantages and disadvantages of different isolation systems.

Isolator	Advantage	Disadvantage
Foundation Isolation System	Don't need a base slab	Provide no restoring forces Economical for small area and low-weight buildings Subjected to creep and environmental conditions Moisture ingress at the base of the walls due to the additional water
Roball Bearings	Economical for low-weight buildings High damping Provided high torsion capacity	Practical just for low-rise buildings Provided high stiffness and transmitted acceleration Produce at specific damping
RoGliders	Suitable and simple for low and medium weight buildings Resistance to torsion effects Provided high damping	Produce high stiffness and attract high seismic forces Represent specific damping Unstable at large displacement
Sleeve Piles	Economical in some cases	Low damping Illustrate large displacement Higher mode effectiveness, in the combination with dampers
Elastomeric Bearings	Manufacture at low cost Presented moderate-transmitted acceleration Easy to model Mechanical properties are not affected by environment	Provided large displacement Low damping Low torsion capacity Affected by P-Delta moments Change in properties under cyclic loads

Table 2.2: Advantages and disadvantages of different isolation systems (continued).

High Damping Rubber Bearings	Illustrate moderate transmitted acceleration No affection in higher modes	Restrict choice of damping and stiffness Low torsion capacity Their properties are influenced by scragging Affected by P-Delta moments
Lead Rubber Bearings	Represent moderate transmitted acceleration State wide option of stiffness and damping Easy to model Mechanical characteristics are not affected by environment	Low torsion capacity P-Delta moments influence stability of isolator Higher modes effectiveness Change in properties under cyclic loads
Friction Pendulum Systems	High torsion capacity Simple design Too resistance under service loads Installation advantages High damping Period is independent of building's weight Different properties for different levels of earthquake	High stiffness High transmitted acceleration Sticking Friction coefficient depends on pressure and velocity

CHAPTER 3

SEISMIC ISOLATED BUILDINGS IN THE WORLD

3.1 Introduction

In the recent decades, number of seismic isolated buildings increases significantly in the world. In the past, most of the people thought that application of seismic isolators were related to the important buildings due to the high cost of these devices. But recently, the numbers of isolator's manufacture companies increase dramatically. Therefore, using seismic isolator in the buildings is expanded to the residential ones.

In this chapter, seismic isolated buildings in some important countries will be discussed by considering some example buildings. At below table a few numbers of world-wide base isolated buildings till 1990 are shown:

Table 3.1: Applications of seismic isolation world-wide (May, 1990) (T.E.Kelly et al.).

Country	Constructed Facilities
Canada	Coal shiploader, Prince Rupert, BC
Chile	Ore shiploader, Guacolda
China	2 houses(1975); Weight station(1980); 4-story dormitory, Beijing(1981)
England	Nuclear fuel processing plant
France	4 houses (1977); 3-story school (1978); 2 nuclear power plant and waste storage facility(1982)
Greece	2 office buildings, Athens
Iceland	5 Bridges
Iran/Iraq	Nuclear power plant, Karun River; 12-story building(1968)

Japan	See table 3.3
Mexico New Zealand	4-story school (Mexico city) See table 3.2
Rumania	Apartment
USSR	3 buildings, Sevastopol 3-story building
South Africa USA	Nuclear power plant See table 3.4
Yugoslavia	3-story school, Skopje(1969)

3.2 Structures Isolated in New Zealand

Applications of seismic isola ted buildings in New Zealand consist of:

1. Rocking columns
2. Elastomeric bearings
3. Flexible sleeved-pile foundations

The combination of energy dissipation devices with different type of isolators converts to a new approach in this country. In fact, these devises are applied to control seismic displacements, transmitted accelerations, frequencies and etc. Table 3.2 illustrates a few applications of base isolated buildings in this country.

Table 3.2: Seismic isolated buildings in New Zealand (T.E.Kelly et al.).

Building	**Stories**	**Total Floor Area (m^2)**	**Isolation System**	**Date Completed**
Wiliam Clayton	4 stories	17000	Lead Rubber	1981
Union House	12 stories	7400	Flexible Piles and steel dampers	1983
Wellington Central Police Station	10 stories	11000	Flexible Piles and Dampers	1990
Press Hall, Press Houses, Petone	4 stories	950	Lead Rubber Bearing	1991
Parliament House	5 stories	26500	Retrofit of elastomeric bearing and LRB	Original Building 1921
Parliament Library	5 stories	6500	Retrofit of elastomeric bearing and LRB	Original 1883/1899 retrofit proposed

3.2.1 William Clayton Building

William Clayton Building was constructed in 1981 in New Zealand. This building was the first base-isolated structure in this country (Charleson, Wrightand, and Skinner, 1987) and the first building in the world that was isolated by Lead Rubber Bearings. This 4-story reinforced concrete frame structure was installed on 80 Lead Rubber Bearings.

Figure 3.1: Wellington Clayton Building during construction.

These Lead Rubber Bearings length structural period from 0.3 seconds for fixed base to 2.0 seconds after isolating. The maximum base shear of this building is reduced from 0.38 W to 0.2 W that is approximately half. This isolated building illustrates inter story drift about 10 mm (0.002 times high of the story) that is distributed identically for all stories. Water, gas pipes, electricity cables and stair ways were designed according to the 150 mm displacement (T.E.Kelly et al.).

3.2.2 Union House

The isolation system in this 12 story building is based on flexible sleeved-pile systems. In fact, this system consists of piles that are flexible laterally and are covered by steel jackets. These piles with 10-13 meter long, passing through out of soft soils and resting on the layers of sandstone. Beside, steel tubes provide 1200 mm seismic gaps for piles during earthquake. In this isolation system, damping is provided by steel cantilever dampers. Structural stiffness is increased by steel cross-bracings (R.Park, 2000). However, this isolation system is effective for buildings that are located on the soft soils. These piles are located under foundation to transfer vertical loads to the hard soil layers and provide flexibility at the foundation level.

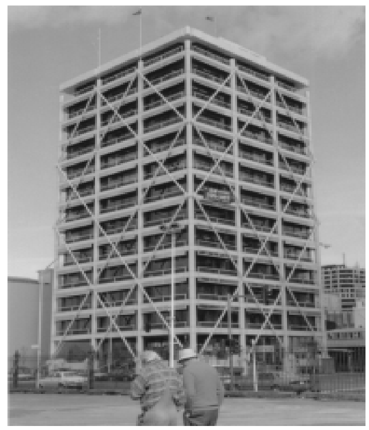


Figure 3.2: Union House, Auckland City (R.Park, 2000).

3.2.3 Wellington Central Police Station

Wellington Central Police Station was completed in 1991. In this building the same isolation system as well as Union House is used. This is 10-story building in high which supported on 16 meter long piles and covered with steel casts. In this system, damping is provided by using lead-extrusion dampers. These dampers were installed between top of the piles and basement (R.Park, 2000).

For constructing this building, different systems were proposed: cross-brace frame, moment resistance frame and seismic isolated cross-brace frame. Since this structure is located on soft soil region and piles foundations are needed, so the latest method is chosen. Finally, seismic isolated cross-brace frame method saved 10% cost at the compare to moment resistance frame (T.E.Kelly et al.).

Figure 3.3: Wellington Center Police Station. Figure 3.4: Lead extrusion damper.

3.3 Structures Isolated in Japan

Since Japan is located in high seismic zone, therefore this country achieved remarkable progresses in seismic resistance buildings comparing to other countries. The number of seismic isolated buildings in Japan increased after completing first modern isolated structure in 1986 and then decreased due to the high cost of these devices.

Combination of Natural Rubber Bearings and dampers, Lead Rubber Bearings and recently High Damping Rubber Bearings are the most popular isolation systems in this country (Naeim and J.M. Kelly, 1999). However, there are a lot of isolated buildings in Japan that a few of them are listed in the Table 3.3.

Table 3.3: Seismic isolated buildings in Japan (T.E.Kelly et al.).

Type	**Building Name**	**Story**	**Area (m^2)**	**Isolation system**	**Date**
Dwelling	Yachiyodai	2	114	EB+F	1982
Apartment	Sibuya Simizu	4	681	EB+S	1986
Office	Lab.J building	4	636	LRB	1987
Apartment	Asano Building	4	1186	HDR	1988
Apartment	Acoustic Lab	6	2065	EB+S	1989
Computer	Noukyo Center	3	5423	LRB	1990
Office	Kasiwa Kojya	4	2186	HDR	1990
Laboratory	Andou Tech	3	545	LRB	1991

EB=Elastomeric Bearing F=Friction damper S=Steel damper

3.3.1 The High-Tech R&D Center, Obayashi Corporation

This is a reinforced concrete frame structure with 5 stories in high that was completed in August 1986. This building is rested on 14 Laminate Rubber Bearings. Damping in this isolation system was provided by combination of 96 steel bar dampers with 32 mm in diameter and frication dampers which were used for providing additional damping in this structure. This building experienced Lbaraki Earthquake in 1989. The accelerograms that were installed on the top of the building showed high reduction in transmitted acceleration at the compare to other fixed base structures (T.E.Kelly et al.).

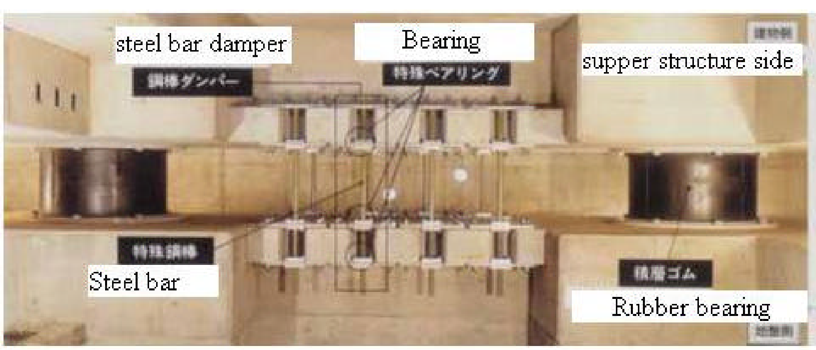


Figure 3.5: Isolation system used in the Obayashi High-Tech R&D Center (T.E.Kelly et al.).

3.3.2 West Japan Postal Computer Center

West Japan Postal Computer Center is one of the largest base isolated buildings in the world. This building is located in Sanda at 37 km from epicenter of Kobe Earthquake that produce strong ground motions in that zone (Figure 3.6). This six-story building is rested on 120 Elastomeric Isolators. The other components of isolation system are steel and lead dampers that were added to this system for increasing structural damping properties.

This isolation system shows its effectiveness with raising structural period to 3.9 second and reducing story accelerations to 127 cm/s^2 (0.13g). This building is one of the structures that experienced Kobe Earthquake and showed elastic behavior during sever ground motions, while the fixed base ones contiguous to the postal center experienced sever damages (Naeim and J.M. Kelly, 1999).

Figure 3.6: West Japan postal computer center (Naeim and J.M. Kelly, 1999).

3.3.3 The C-1 Building, Fuchu City, Tokyo

This large building was completed in 1992. This composite structure (steel and steel-reinforced concrete frame) consists of 7-story, a penthouse and one basement. 68 Lead Rubber Bearing (LRB) were used in isolation system to isolate 37844 m^2 area of 45000 m^2 total area. The dimensions of these LRBs are 1.1 and 1.5 meter in diameter with 180 to 200 millimeter lead plugs. A layer of rubber with 10 millimeter thickness was used for protecting isolators against environmental conditions. This isolation system shifts structural period to 3 second approximately (T.E.Kelly et al.).

3.4 Structures Isolated in the USA

The first isolated building in the united state is Foohill Communities Law and Justice Center that was mounted on elastomeric bearings. Since that time, a lot of buildings and bridges have been constructed and strengthened with seismic isolators. In the United States, buildings usually are designed for Design Basis Earthquake and Maximum Capable Earthquake. Design Base Earthquake (DBE) is used for designing superstructure and elements below isolation system whereas Maximum Capable Earthquake (MCE) is applied to check stability of the isolators under MCE displacement (Clark et al., 1993). Below table shows a few numbers of buildings which are isolated in the United State (Mayes 1990, 1991) (T.E.Kelly et al.).

Table 3.4: Seismic isolated buildings in United State (T.E.Kelly et al.).

Building	Story	Floor Area (m^2)	Isolation System	Date
Foothill Communities	4	17000	HDRB	1985/6
USC University Hospital	8	33000	NRB & LRB	1989
Rockwell Building (retrofit)	8	28000	LRB	1989
Kaiser Computer Center	2	10900	LRB	1991
Channing House (retrofit)	11	19600	LRB	1991
Long Beach Hospital (retrofit)	12	33000	LRB	1991

3.4.1 Foothill Communities Law and Justice Center

Foothill Communities Law and Justice Center is located in Rancho Cucamonga City, 97 km from Los Angeles and 21 km of San Andreas Fault. This building is the first isolated building in United State as well as the first isolated building in the world that is installed on High Damping Rubber Bearings.

This 4-story building is designed to endure 8.3 magnitude earthquake and mounted on 98 High Damping Rubber Bearings with 10 to 12 % damping. This building was completed in 1985 with total cost of $38 million. Before the plan was finalized, two alternatives were suggested. The first one corresponded to a fixed base building with 5 % structural damping. The natural period, base acceleration and roof acceleration were estimated 1.1 second, 0.8g and 1.6g respectively. An isolated building with 10 to 12 % damping was considered as another case for this building. On the other hand, natural period, base acceleration and roof acceleration change to 2 second, 0.35g and 0.4g respectively (Clark et al., 1993).

Clarck et al. (1997) investigated the behavior of bearings for this building, 12 years after installation. Two pairs of isolators were tested. One of them was removed from the building and another one was new isolators with the same characteristics. These two isolators were tested and the results compared. The results of the test illustrated that although they present negligible change from their original properties but still they provide satisfactory behavior under large magnitude of earthquakes.

Figure 3.7: Foothill communities law and justice center (T.E.Kelly et al.).

3.4.2 Pasadena City Hall

This building was constructed in Spanish Mission Style in California. So far the building has experienced many large earthquakes and some damages have been seen. According to the investigations, the building could not survive during large ground motions. Therefore this building was decided to strengthen with the best method.

Furthermore, the additional repairs had been done to restore its architectural and historical facade.

Generally, for retrofitting structures with isolation systems, the most significant part is to install isolators under building. For this reason, a concrete moat was built around this building, then a part of the ground under basement was excavated and new beams were fixed under old slabs. Finally the building was installed on totally 240 isolators to protect building against strong ground motions (Dywidag System International [DSI], 2007).

Figure 3.8: Pasadena City Hall (DIS, 2007).

Figure 3.9: Isolation System for Pasadena City Hall (DIS, 2007).

3.4.3 Oakland City Hall

Oakland city hall that is the first high rise governmental building in United State, was completed in 1914. The top section of this 18-story building is 98.7 meter above the street level. The podium that is the lowest and widest part of this building has 3-story and contains central rotunda-council chambers and administration offices of the Mayor and City Manager. The two other parts are 10-story office tower and 2-story clock tower (Figure 3.10).

The major part of this building consist of rivet steel frame with infill masonry walls, granite and terra coat. Terra coat over brick masonry walls are used for clock tower part. The building is rested on a continuous mat foundation.

Oakland city hall experienced heavily damages during Loma Prieta earthquake in October, 1989. According to the statistics, 20% in the north direction and 30% in the east-west direction of the building failed. After this earthquake, Federal Emergency Management Agency (FEMA) and State Historic Preservation Office (SHPO) carried out many investigations on the building. According to their studies, seismic isolation method was selected as the most effective and economical one for strengthening this historical structure.

The isolation system for this building consists of 110 Lead Rubber Bearings with the range of 737 mm to 940 mm in diameter. After installation process of isolators, the remind cracks were repaired by a mix of self-tapping anchors, metal lath, epoxy and cement plaster (Walters, 2003).

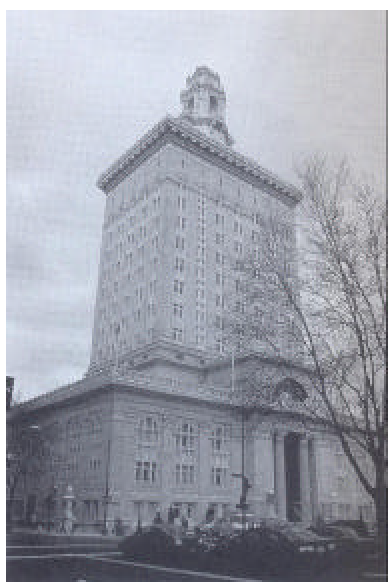


Figure 3.10: Oakland City Hall, California
(Naeim and J.M. Kelly, 1999).

3.5 Structures Isolated in Turkey

Earthquake and seismic engineering was an important engineering problem in Turkey. Recently, this country has completed all studies and investments about seismic isolation and refurbishment of existing buildings. However, Turkey can design and apply all kinds of seismic isolation systems in different structures (ERSE Company, 2009). Generally, applications of seismic isolated buildings in this country consist of:

1. Lead Rubber Bearings
2. Friction Pendulum Systems
3. Combination of rubber bearings and dampers

3.5.1 Antalya Airport

Antalya International Terminal Building is one of the most important airports in Turkey. This reinforced concrete structure was completed in 1998 (Figure 3.11). This 3-story building covers 55000 square meters, which is divided into five main sections separated by expansion joints. This structure was designed in 1996 based on the 1975 Turkish Earthquake Code. However, this code endures major revisions in 1998. Based on these revisions and new seismically maps, zone factor for Antalya was increased from Turkish zone 4 to Turkish zone 2. Because of the importance of this building, it was decided to strengthen at least for new Turkish Code.

Figure 3.11: Layout of Antalya International Airport Building (Yilmaz, Booth and Sketchley, 2006).

For strengthening this building, a number of retrofit options were investigated. The aim was to find economical one to provide high performance level after major earthquakes by protecting structural and architectural elements. Section enlargement was found to be incapable to provide sufficient lateral resistance. Adding shear walls to the structure was found to be troublesome for terminal building. After doing all investigations, seismic isolators (Lead Rubber Bearings) were proposed as the optimal solutions. Due to the lack of basement, these bearings were installed 1.2 m above ground level to minimize the flexural effects at the top of the column and foundation (Figure 3.12) (Yilmaz, Booth and Sketchley, 2006).

Figure 3.12: Installation of isolator in Antalya Airport (ERSE Company, 2009).

3.5.2 Istanbul's Ataturk International Airport

Istanbul's Ataturk international airport terminal building experiences Izmit Earthquake when the construction process was nearing completion (maximum horizontally ground acceleration was 0.1g). This building is a three story reinforced concrete structure with space frame roof. Dimensions of the building are 240 m by 168 m. A view of building is represented at Figure 3.13.

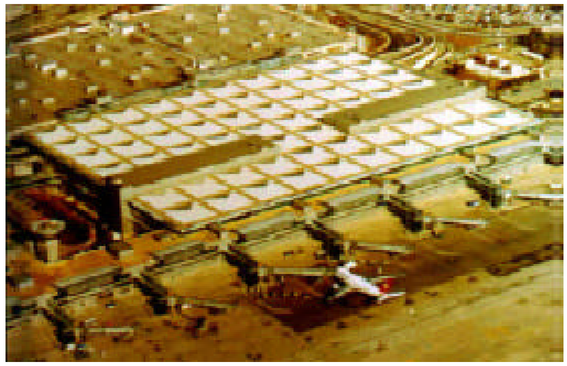


Figure 3.13: New Ataturk international airport terminal building (Constantinou et al., 2001).

For strengthening this airport, a number of retrofit options were investigated. The aim was to find economical one to provide high performance level after major earthquakes by protecting structural and non-structural elements. Finally, new technologies in the term of seismic isolators (Friction Pendulum Systems) and dampers were selected. Two alternative of seismic isolation were proposed: base isolation of entire building and isolation of the roof truss. The first one needs demolition and reconstruction of the base floor. Because mechanical and baggage handling systems are located in this floor, removal and reinstallation of these systems are not possible. Therefore, the second one was chosen. This method is included seismic isolation of the space-frame roof, jacketing and strengthening of existing reinforced concrete columns (Figure 3.14). Friction Pendulum Systems with isolated period of 3 seconds, friction coefficient of 0.09 and 260 mm displacement capacity are selected for isolation system (Constantinou et al., 2001).

Figure 3.14: Isolation and strengthening of terminal building (Constantinou et al., 2001).

3.5.3 Tarabya Hotel

Tarabya hotel with 14 story and 35000 square meters is located in Istanbul (Figure 3.15). This building is retrofitted by 139 Friction Pendulum Systems with the cost of 4 million dollars. Each isolator is designed for 32 mm design displacement and 1 tone vertical load. This isolation system shifts structural period from 1.5 seconds to 3 seconds. Diagonal steel bracings are used for strengthening building at the location of the isolators (Figure 3.16) (Murat, 2006).

Figure 3.15: Tarabya Hotel (Murat, 2006).


Figure 3.16: Diagonal bracings at the base level(Murat, 2006).

CHAPTER 4

PRACTICAL APPLICATION OF ISOLATION SYSTEMS

4.1 Introduction

For protecting buildings against to earthquake the only method is not strengthening, but to reduce earthquake forces exerted to structure. Among many advanced techniques, the more popular and economical methods are applying isolation systems and energy dissipation devices (MCEER, 2008).

However, seismic isolators are the newest method for retrofitting buildings such as historical buildings and the buildings with thin sections and etc. When the result of the seismic evaluation shows insufficient stiffness and strength of the expected building, seismic isolators are the best alternative for strengthening, without exerting important changes in the structure (Oskouei, 2006). In the other methods, such as section enlargement, additional shear walls and etc, building should be vacated during retrofitting process. However, it is not practical as always. For example, the historical or governmental type of buildings, it is not easy to move out building due to economical and political factors. Under these conditions, using isolator for retrofitting of these types of buildings is the most acceptable method. In this method, the building can be strengthened while it is operational.

In this chapter, installation process of the most popular isolation systems such as rubber bearings and sliders are described step by step for the new and strengthening steel and reinforced concrete frame structures.

4.2 Installation Process in Reinforced Concrete Frame Structures

4.2.1 Installation Process of Rubber Bearings in New Concrete Structures

The installation process of the rubber bearings in new reinforced concrete frame buildings is described in different stages as given below:

1. Reinforcement of foundation and installing bolts between reinforcements. Isolators should be installed on the platforms because of their heavy weights (Figure 4.1).

Figure 4.1: Installation stage (Dogan, 2007).

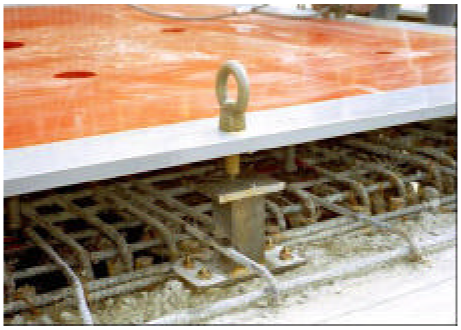
2. Pouring concrete of the foundation (Figure 4.2).

Figure 4.2: Concrete pouring of the foundation (Em-Ke Ltd).

3. Installing balance plates on the bolts and leveling them (Figure 4.3a,b,c,d). These bolts are used for connecting bottom plate of the isolators to the foundation.

(a)

(b)



(c)

(d)

Figure 4.3.a,b,c,d: Installing plates and leveling them (Dogan, 2007).

4. Filling the space under the plate with grout (Figure 4.4).

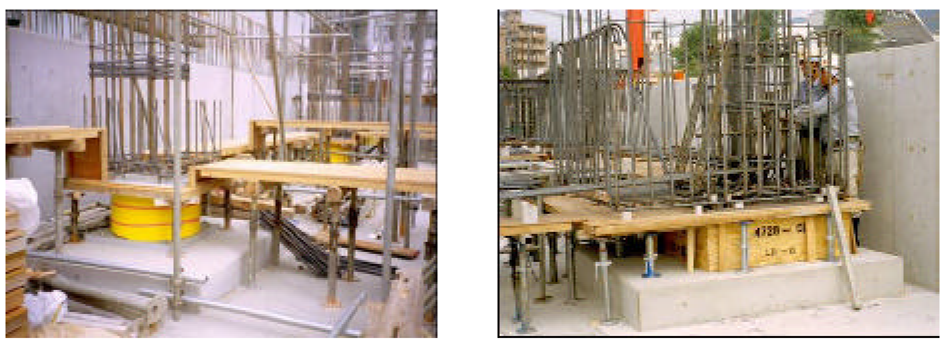
Figure 4.4: Grout pouring under the plate (Em-Ke Ltd).

5. Mounting the isolator on the platform (Figure 4.5 and 4.6).

Figure 4.5: Installation of the isolator (Dogan, 2007).

Figure 4.6: Mounted isolators (Em-Ke Ltd).

6. Installing column reinforcements on the top plate of the isolators and continuing other construction processes as well as fixed base buildings (Figure 4.7.a,b).



(a) (b)

Figure 4.7.a,b: Installing column reinforcements on the isolator (Em-Ke Ltd).

In the base isolated structures, an additional slab is performed at above of the isolator level to prevent columns of individual movements. In the case of using this additional slab, the top plate of the isolator is bolted to the concrete floor beams as shown in Figure 4.8.

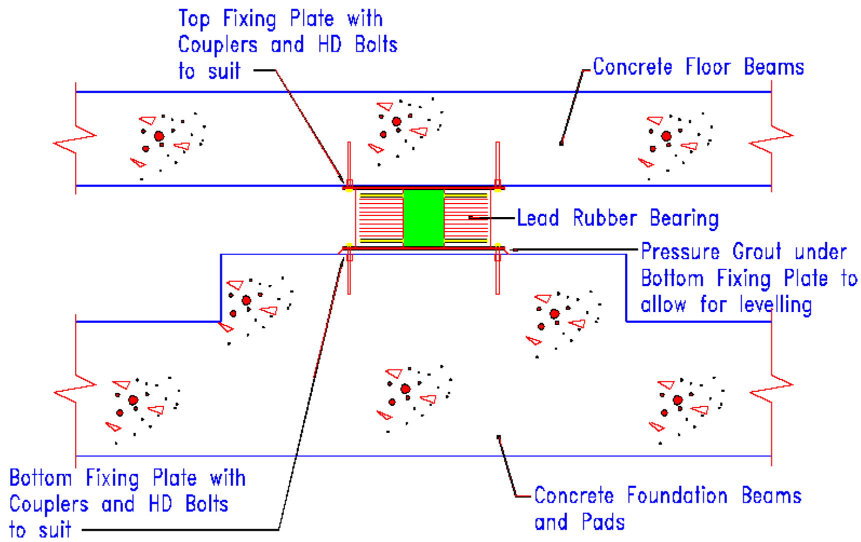


Figure 4.8: Installation of the isolator in concrete structure (T.E.Kelly et al.).

7. After finishing the construction process of the base isolated building, mechanical works are installed. At the isolated level of the building, these mechanical works should be produced with flexible materials and designed for maximum design displacement (Figure 4.9). Beside, staircases at the base level can be isolated of foundation by two different methods. In the first method, staircases are rested on the sliders, which are located on the foundation. In the second method, they are performed like cantilever near the foundation (Figure 4.10). This method is more economical than the first one.

Figure 4.9: Isolating mechanical works in base isolated building (Em-Ke Ltd).

Figure 4.10: Staircase, which performing as cantilever in the base isolated building (Em-Ke Ltd).

Sometimes, due to the high vertical load of a special column, it is more economical to install the expected column on a group of small isolators instead of one huge isolator (Figure 4.11). This method was applied to San Francisco City Hall and Tan Tzu Medical Center in Taiwan (DIS, 2007).

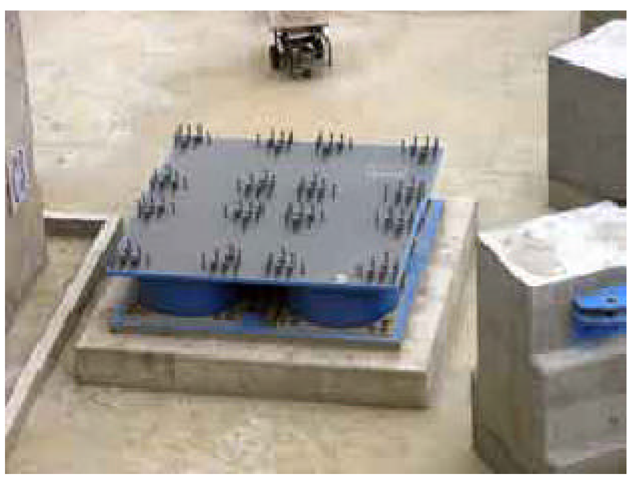


Figure 4.11: Group of isolators, which are located under a column with high vertical load (DIS, 2007).

In the case of providing additional damping in the isolation system, horizontally located energy dissipated devices (dampers) are used. In this case, one side of the damper is connected to pedestals (they are performed between isolators) and another side to the superstructure (Figure 4.12 and 4.13).

Figure 4.12: Isolators and pedestals ,which connect dampers.

Figure 4.13: Connection of damper to structure and pedestal (Lizundia, 2006).

4.2.2 Installation Process of Sliders in New Concrete Structures

Slider bearings also follow the same installation process as well as rubber bearings. The installation details of the sliders in the new reinforced concrete frame structures and bridges are shown in Figure 4.14 and 4.15 respectively.

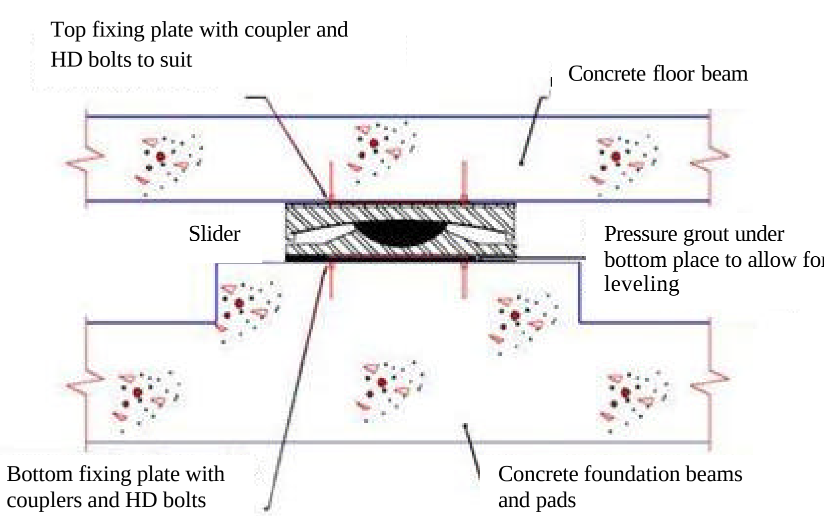


Figure 4.14: Connection details of sliders.

In this connection, top and bottom load plates of the sliders are bolted to the floor's beam and foundation respectively.

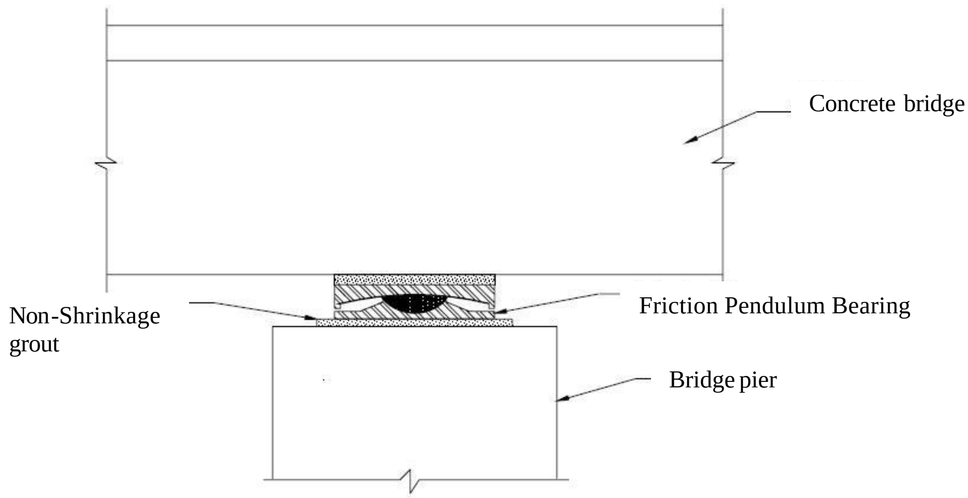


Figure 4.15: Connection details of the sliders in bridge (EPS, 2003).

Sliders provide some useful advantages compared to elastomeric bearings. Some of these benefits are given bellow (EPS, 2003):

1. In the steel structures, they don't need additional load plates for connecting. They are connected by welding columns to top plate of slider.
2. They present higher vertical stiffness than rubber bearings. On the other hand, in the application of the strengthening in buildings, they minimize vertical deflection in the columns.
3. These bearings can be installed with the concave surface, facing either up or down. P-Delta effects on the structural elements below the isolation system are minimized, when the concave surface is installed down. On the other hand, when the concave surface is faced up, the P-Delta moments minimize for the structural elements above the isolation system.

4. Due to the low height of these isolators, they can be located even at the small gaps such as staircases and elevators.

4.2.3 Install Process for Strengthening in the ReinforcedConcrete Structures

In this section, strengthening process of the reinforced concrete frame structures by isolators is described step by stepas shown bellow (Yilmaz, Booth and Sketchley, 2006):

1. Supporting the expected column by temporary steel columns (Figure 4.16.a,b). Hydraulic jacks are installed at the top of the columns between column and floor beams. They are preloaded to the gravity loads which supported by expected column.

(a) Antalya (Turkey) airport
(Yilmaz et al., 2006).

(b) Library, New Zealand
(Robinson Seismic Ltd, 2003).

Figure 4.16.a, b: Column supporting.

2. Defining bench marks on the column above and below the final position of the bearing. These bench marks are used to measure vertical deflection of the column at different steps.

3. Sawing measured part of the column by diamond chain saws (Figure 4.17.a,b) and removing the block of the concrete (Figure 4.18.a,b). According to the installed bench marks, the deflection of the column is measured. Usually, this movement is too small.

(a) Antalya (Turkey) airport (Yilmaz et al., 2006).

(b) Library, New Zealand (Robinson Seismic Ltd, 2003).

Figure 4.17.a, b: Sawing part of the column.

(a) Antalya (Turkey) airport (Yilmaz et al., 2006).

(b) Library, New Zealand (Robinson Seismic Ltd, 2003).

Figure 4.18.a, b: Block removing of concrete.

4. Pouring a bed of epoxy mortar on the bottom surface of the cut column and installing isolator into place (Figure 4.19). The gap above the isolator

is filled by epoxy or non shrinkage grout mortar. After curing epoxy mortar, the supporter steel columns can be removed.

Figure 4.19: Installing isolator into its place (Robinson Seismic Ltd, 2003).

5. The column above and below bearing is strengthened by steel jackets to reduce stress concentration and replace reinforcements (Figure 4.20). This stress concentration exists due to the cutting part of the column and reinforcements.

Figure 4.20: Strengthening column by steel jackets
(Yilmaz et al., 2006).

6. Fireproofing bearing by fire insulations. Then, brackets are defined to the isolator to support architectural finishes (Figure 4.21).

Figure 4.21: Wrapping isolator in fire insulation (Yilmaz et al., 2006).

4.3 Installation Process in Steel Structures

4.3.1 Installation Process of Rubber Bearings in New Steel Structures

The installation process of rubber bearings in new steel structures follows the same procedure as well as concrete buildings. The only difference between these procedures is corresponded to the connection of the superstructure to isolators. In steel structures, after finishing step 5 in section 4.2.1 (connection of the isolator to foundation) (Figure 4.22) columns are bolted to the load plate of the rubber bearings (Figure 4.23).

Figure 4.22: Connection of the isolator to foundation (Robinson Seismic Ltd).

Figure 4.23: Connection of the column to isolator (Robinson Seismic Ltd).

Usually, in the isolation process of medium and high rise buildings, an additional base slab is needed. This additional slab can be performed in steel structures in two ways:

1. Concrete slabs are performed similar to the floor slabs. In this way, top load plate of the isolator is bolted to the connected concrete beams. At next step, steel column is welded to the top load plate of the isolator and concrete beam respectively (Figure 4.24).

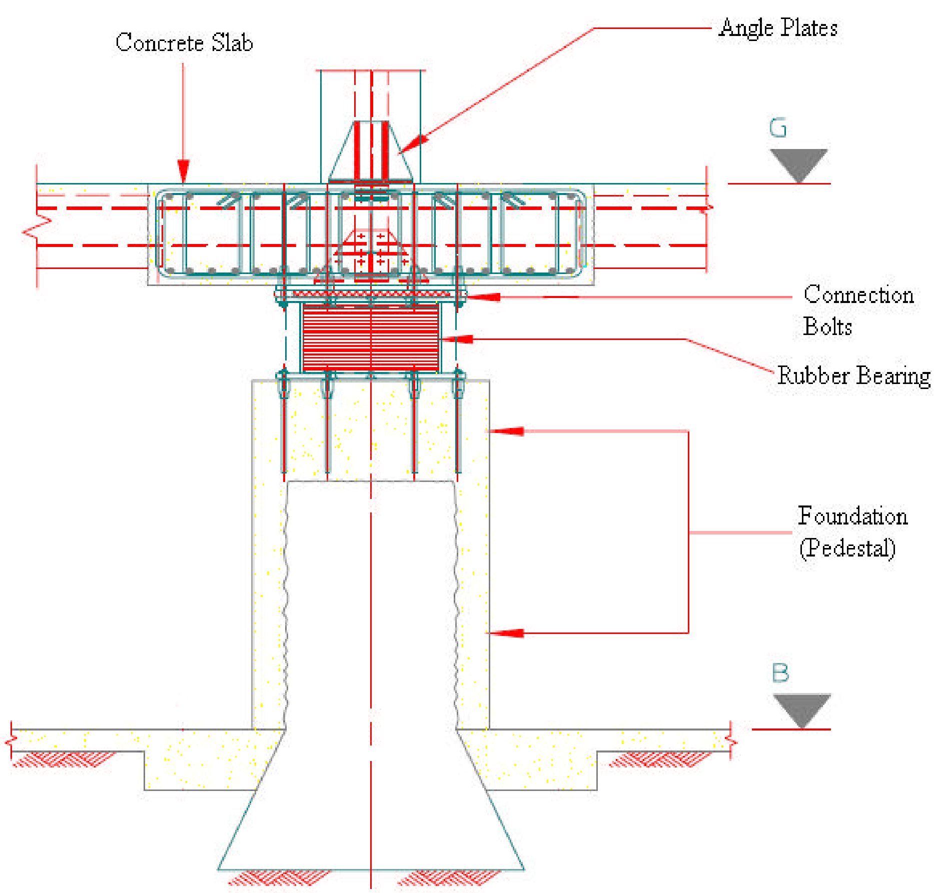


Figure 4.24: Connection details of rubber bearing to concrete slab and steel column (T.E.Kelly et al.).

2. Steel beams with horizontal bracings are another method. In this method, steel beams are connected to the top load plate of isolators by small columns (Figure 4.25).

Figure 4.25: Steel beams for connecting isolators at base level (Robinson Seismic Ltd).

4.3.2 Installation Process of Sliders in New Steel Structures

Slider bearings offer the same installation procedure with rubber bearings. Instead of welding steel column to the load plate of rubber bearing, it can be connected to the top concave plate of the slider. The connection details of slider to the girders in the bridges are shown in Figure 4.26.

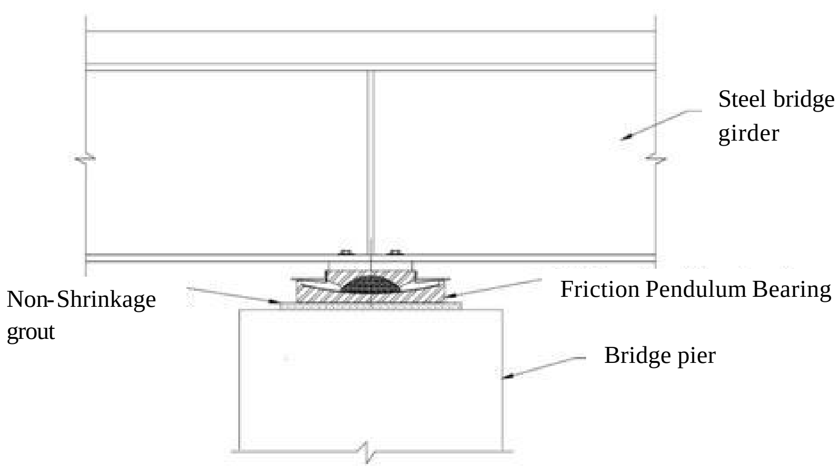


Figure 4.26: Connection details of sliders in bridge (EPS, 2003).

4.3 .3 Install Process for Strengthening in the Steel Structures

The strengthening process of rubber bearings in existing steel structures follows the same procedure as well as concrete buildings till step 3. Other steps are continued as follows (Figure 4.27):

4. Installing bolts and performing new foundation on the existing foundation.
5. Installing isolator on the bolts and filling the space under isolator with grout.
6. Welding existing column to the top load plate of isolator.
7. The column above the bearing is strengthened by concrete jackets to reduce stress concentration. This stress concentration exists due to the cutting part of the column.

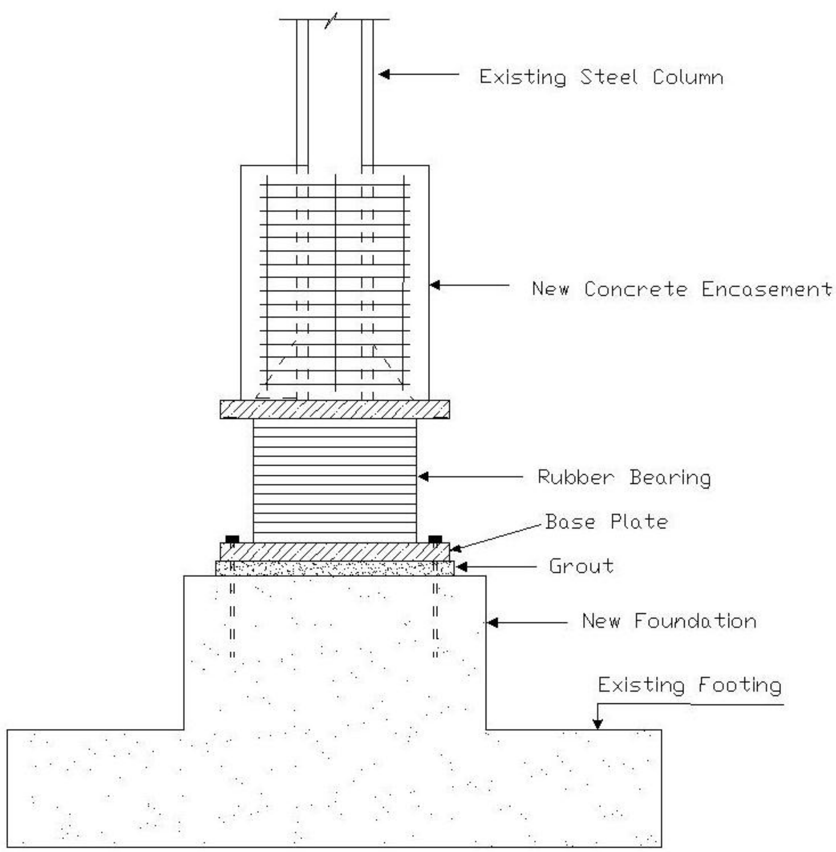


Figure 4.27: Connection details of isolator in existing steel structure.

CHAPTER 5

PROPERTIES OF ISOLATION SYSTEMS

5.1 Introduction

For designing seismic isolated structures, so many parameters should be kept in mind. The most important one is corresponded to the mechanical characteristics of the bearings. Generally, properties of the bearings are affected by increasing in the number of the cyclic loads. Therefore, different mechanical properties of the isolators are discussed in this chapter. Furthermore, different locations of the isolators and cost of the base isolated structures comparing to its conventional fixed base one will be considered in this chapter to reduce these complexities by presenting simple and concise practical information for practitioners in seismic isolated structures.

5.2 Mechanical Characteristics of Isolators

5.2.1 Mechanical Characteristics of Rubber Bearings

Lead Rubber Bearings and High Damping Rubber Bearings are made of natural rubber and a mixture of the rubber and additives respectively. The properties of the utilized rubber are affected by many factors, which are be described in this chapter.

5.2.1.1 Cyclic Change in Properties

In Lead Rubber Bearings, damping and stiffness are functions of the vertical loads and the number of the cyclic loads. According to the test results, these two properties of rubber are reduced with progress in the number of the cyclic loads

(Chou and Huang, 2007). Therefore, in designing a Lead Rubber Bearing, damping and stiffness should be derived from the first cycle of loading.

The properties of High Damping Rubber Bearings are affected by increasing the number of cyclic loads in to a process called scragging. These changes in structural molecules of High Damping Rubber Bearing affect the properties of rubber bearing.

5.2.1.2 Age Change in Properties

Rubber bearing properties are affected by time (Chou and Huang, 2007). A rubber bearing that was removed from a bridge in Kentucky State (USA) shows 10% increment in stiffness. Unlike High Damping Rubber Bearings, damping in Lead Rubber Bearings is not influenced by time because it is provided with lead cores (T.E.Kelly, 2001).

5.2.1.3 Vertical Deflection

Rubber bearings present high vertical stiffness (due to the inserted steel shims) either to resist the vertical loads or to control vertical displacement (Young, 2002). Generally, when these bearings are subjected to the vertical loads for the first time, they show very few vertical deflections (about 1 mm to 3 mm).

5.2.1.4 Long Term Vertical Deflection

Increasing in deformation under constant vertical load is called creep. Creeps in rubber include of physical creep and chemical creep. Physical creep occurs due to the stumble molecular chains, while chemical creep is related to rupture molecular chains. However, in structural bearings, physical creep is dominant. Therefore, rubber bearings are covered to protect from environmental conditions. In this case, the chemical effects in rubber bearings are negligible.

Creep in rubber bearings is quick in the first days but becomes slower with passing of time. Natural Rubber Bearings are more resistant to creep effects than other types. In rubber bearings, creep depends on the type and amount of filler as well as curing in the vulcanization process. According to the tests results, creep deflection is shown to be 20% of initial deformation in the first few weeks and 10% after a period of many years. Studies on an apartment, located in London, showed 0.1 cm creep deflection in a 25 cm Natural Rubber Bearing (Mullins, 1984).

The curing process of High Damping Rubber Bearings is different from Natural Rubber Bearings. These bearings show a 50% deflection of initial deformation at the first 22 hours. This number illustrates the high deflection compared to the 20% of Natural Rubber Bearings (T.E.Kelly et al.).

5.2.1.5 Wind Displacement

In Lead Rubber Bearings, resistance to the wind load is achieved by the elastic stiffness of the inserted lead core. From experiences that have been done, the wind displacement has a variety of 3.5 mm for a wind load of 0.01W to 11 mm for 0.03W load (T.E.Kelly, 2001).

Generally, in the High Damping Rubber Bearings, high initial shear modulus provides resistance to the wind load.

5.2 .2 Mechanical Characteristics of Slider Bearings

Steel and Teflon are used for making sliders bearings. Unlike Rubber Bearings which are affected by vertical loads and environmental conditions, they provide more resistance properties than other types.

5.2.2.1 Bearing Compression Strength and Stiffness

Generally, sliders offer more strength and stability than other types of bearings. It has been proven that in these bearings, the friction coefficient remains constant under high vertical loads (Higashino et al.). An isolator of the U.S. Court of Appeals project in San Francisco was subjected to the compressive load equalling nine times its design vertical load at zero displacement and maximum design displacement. This bearing was then tested under cyclic load compression and shear. The results of the test showed that the bearing retained its properties without any changes. Generally the compression stiffness which is produced by slider bearings is 7 to 10 times more than elastomeric bearings (EPS, 2003).

5.2.2.2 Unscragged and Scragged Properties

Slider bearings do not have the same properties in scragged or unscragged cyclic loads. Since the first cycle of loading for unscragged characteristics is stiffer than scragged properties, therefore higher shear forces are transmitted to the structure. On the contrary, scragged cyclic loads are less stiff and produce higher displacement in the bearing. So the first properties of cyclic loads should be considered in structural base shear design while the scragged properties are used for checking isolator stability at a maximum bearing displacement (EPS, 2003).

5.2.2.3 Temperature Effects

Varying temperature affects coefficient of friction in slider bearings. Low temperature increases the stiffness of the slider while high temperature decreases stiffness. Consequently, it is recommended that structural shear force and design displacement derive based on low and high temperature bearing properties respectively.

5.2.2.4 Aging Effects

The main characteristics of slider bearings are period, stiffness and damping. In fact, period and stiffness are functions of isolator's radius which is constant during the passing of time. In the slider bearings, damping is governed by the friction coefficient. On the other hand, the passing of time affects this coefficient. Beside, Higashino et al. prove that changing in dynamic coefficient of friction is negligible over time.

5.2.2.5 Fire Resistance

The main materials used in the slider bearing's construction process are stain less steel plate and Teflon. These materials offer the innate fire resistance. Nevertheless, the fire protection materials can be used for protecting isolators in two ways:

In the first method, the exterior part of the isolator is covered by heat insulation materials such as fireproof aggregates (Figure 5.1.a), fire blankets (Figure 5.1.b) and etc. At another method, the bearing is confined by pre-encased fire board (Figure 5.1.c). In this situation, the fire board is installed to allow the isolator's movement for Maximum Credible Earthquake displacement (EPS, 2003).


(a) Fire proof aggregate


(b) Fire blankets


(c) Pre-encased fire board

Figure 5.1.a,b,c : Different types of heat insulation materials (Dynamic Isolation System, 2007).

5.3 Location of the Isolators

According to seismic demands, isolators can be located at every point of the structures. In fact, for protecting more structural elements, it is better to locate isolators as near as possible to the foundation. However, the location of the isolators is affected by cost and practical considerations (Hartford Loss Control Department, 2002). Some of the most important locations for the isolators are discussed below:

1. Isolator is located at the connection of column to foundation (Figure 5.2). This is the most common location of the isolators.

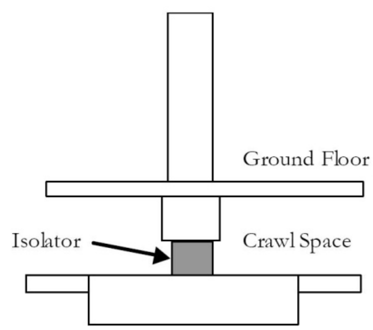


Figure 5.2: Location of the isolator at the base (T.E.Kelly et al.).

2. For buildings with basements, isolators can be located in three locations: Bottom (with or without base slab), top and mid-height of the columns the (Figure 5.3). If the isolator is located at the top of the column, the isolation system doesn't need an additional slab. Structural elements below the isolation system should be designed for P-Delta moments.

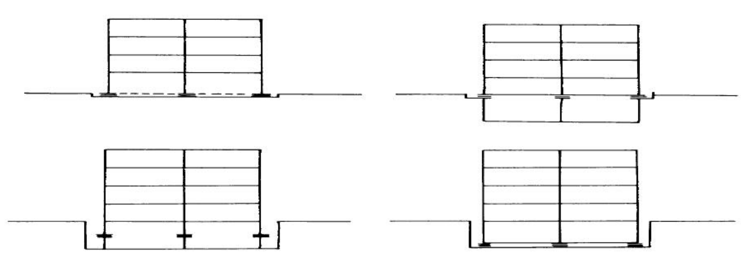


Figure 5.3: Installation at basement (Hartford loss control department, 2002).

3. Story isolation. In this method, isolators are installed at the desired story (Figure 5.4). The biggest disadvantage of this method is related to the design structural elements for P-Delta moments, which makes this method uneconomical. At times, though, due to the limitation in the seismic gap, this method should be carried out (Zhou et al., 2006).

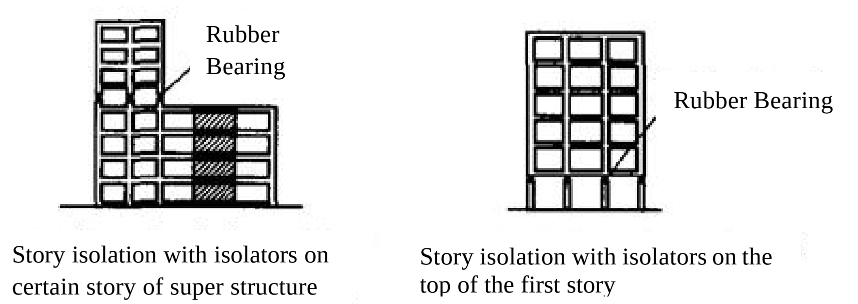


Figure 5.4: Story isolation (Zhou et al., 2006).

4. Top isolation. This method is useful for existing structures with planned additionalstories (Figure 5.5).

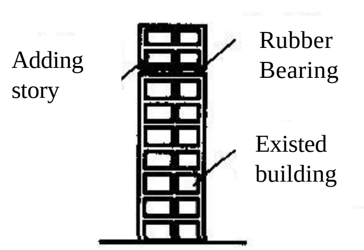


Figure 5.5: Top isolation (Zhou et al., 2006).

5.4 General Cost Considerations of Base Isolated Structures

The cost of the base isolated structures is the main question that concerns most engineers when talking about isolated buildings. This debatable topic is felt to be economical by some engineers and uneconomical by others. Yegian and Kadakal (2004) concluded that base isolated buildings are uneconomical compared to the equivalent fixed base structures (these devices should be applied for important buildings) while Carrillo and George (2004), Iemura et al. (2006), Zhou et al. (2006) consider them as economical. However, applications of base isolation to the buildings increase the cost of the structures in the up to 5% of the total cost (Naeim and J.M.Kelly, 1999).

5.4.1 Cost of Isolators

Cost of the isolators depends on displacement and vertical load capacity. The higher the vertical load capacity, the more expensive isolator is required.

Generally, an isolator's costs include direct and indirect costs:

1. Buying isolators (direct cost)
2. Installing them (indirect cost)
3. Designing isolation system: Generally, a few isolated buildings are prone to needing static analysis. At the least, a response spectrum analysis is needed. For engineers, doing response spectrum and time history analysis are time consuming methods that raise the cost of the isolated structure. In the manufacture of some isolators, the cost of designing the isolation system is expected to be covered by the vendor (indirect cost).

5.4.2 Cases Which Cause Increment or Saved Cost of the Isolated Structures

Some cases causing an increment in the cost of the isolated structures are:

1. In most cases, isolators need to perform under a concrete slab to reduce wind displacement and to control the individual column's displacement. In fact, compared to the fixed base buildings, base isolated buildings need one more slab.
2. Structural elements under an isolation system (except foundation) need to be designed for secondary moment effects (P-Delta moments) that cause an

increment in structural costs. Consequently, it is better to place isolators as near as possible to the foundation to save money.

3. In the isolated structures, staircases and elevators should be performed like cantilevers or supported on sliders.
4. Pipes, electric cables and any other joints are required to be constructed of flexible materials to endure total design displacement.

Factors that can save money are:

1. Structure above isolation system is designed for very low level of ductility (R_i =2). On the other hand, compared to the fixed base structures, isolated structures are designed for a high level of performance.
2. According to the UBC 97 code, the reduction factor varie s from 3.5 to 8.5 for special, intermediate and ordinary moment resistant frames. But for isolated moment resistant frame buildings, this coefficient is 2 regardless of the frame type. Consequently, instead of special moment-resistant frame, the structure can be designed in an intermediate frame, which means money saved.
3. In comparison to other methods, isolators in respect to seismic rehabilitation are known to be the most economical method (T.E.Kelly, 2001).
4. After an earthquake, damage costs (the most important factor in cost savings) are less in isolated structures than fixed base ones. Usually, damage costs tend to increase when a building is made stronger (T.E.Kelly, 2001).
5. Sometimes isolators are used to protect non structural elements and equipment. Generally, in a fixed base structural seismic design, codes provide minimum performance level which keeps a structure from collapse during an earthquake. But damages to the structural and non structural elements are unavoidable. Figure 5.6 shows damages to a fixed base structure affected by

earthquake. Illustrated in this figure, the frame is still standing whereas the surrounding walls were destroyed.

Figure 5.6: A reinforced concrete structure affected by 2003 Earthquake in Algeria (Carrillo and George, 2004).

CHAPTER 6

DESIGN SEISMIC ISOLATED BUILDINGS

6.1 Introduction

Generally, there are four basic types of isolation system's force-deflection relationships. These idealized curves are shown at Figure 6.1.

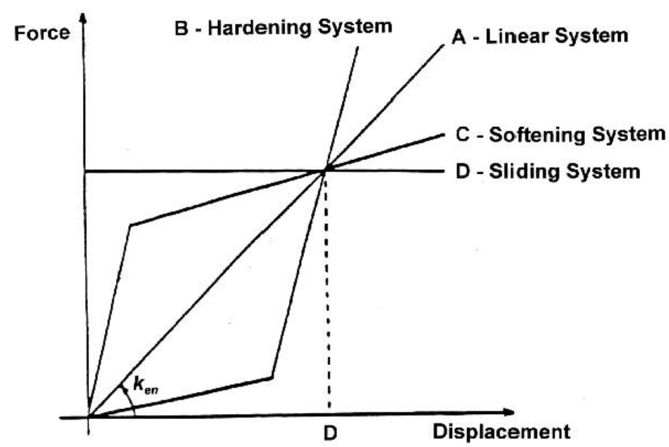


Figure 6.1: Idealize force-deflection relations for isolation systems (Commentary, 2003).

In this figure, linear isolation system is illustrated by curve A. This isolation system represents the same periods for different earthquake loads levels. Hardening isolation system that is presented by curve B, is soft initially and then stiff when the lateral load increases. Curve C shows softening systems. These systems are initially stiff and show softening behavior when the earthquake load is increased. A sliding isolation system is presented by curve D. In this system, effective period is lengthened when the lateral loads are increased and remained constant on superstructure (Commentary, 2003).

Generally, designing of base isolation system is based on trial and error procedures. These design procedures are started by primary design. Typically, this primary design is based on previous project's experience or engineer's experience. Sometimes, some manufactures provide catalogs which are useful sources for primary design. At this stage, design displacement, damping, natural period of isolation system are estimated and structure is designed according to these values. After passing this stage, an example of isolators is subjected to the prototype tests. According to the results of prototype tests, the primary design process may or may not need to be repeated. In order to minimize the number of trial and error process, it is necessary to have good and accurate data for primary design stage.

In fact, prototype design procedures are started by deriving seismic requirements of design codes. Among a few codes that provide information about seismic requirements for isolation systems, Uniform Building Code 1997 (UBC 97) and American Association of State Highway and Transportation Officials (AASHTO) are the most popular ones. After this stage, designing isolation system is started. Figure 6.2 illustrates a flowchart which describing the general procedures for designing isolated structure.

This chapter describes design procedures for seismic isolated buildings. These procedures are based on UBC 97, seismic requirements for structural elements under and above isolation system. At the other parts, designing procedures for Lead Rubber Bearings, High damping Rubber Bearings and Friction Pendulum Systems will be discussed.

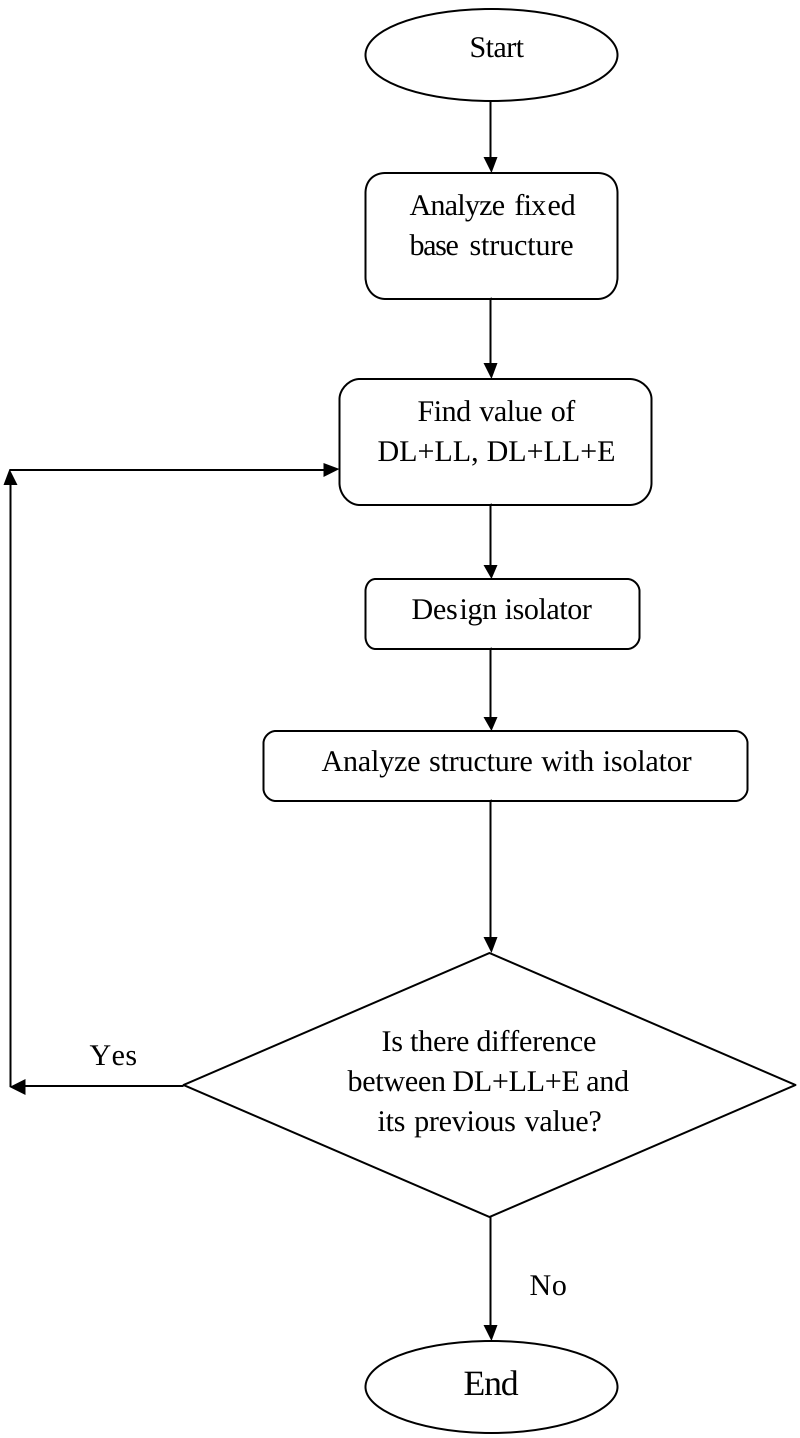


Figure 6.2: Design procedures for isolated building.

6.2 UBC-97 Requirements

6.2.1 Definitions

This section is started by some definitions of the code.

Design Displacement is the design base earthquake lateral displacement, expecting displacement due to torsion, which is necessary for designing isolation system

Design-Base Earthquake (*DBE*) is an earthquake with 10 percent probability of happening in 50 years with return period of 475-year.

Effective Damping is the value of viscous damping related to energy dissipated throughout the isolation system cyclic response.

Effective Stiffness that is equal to the exciting lateral forces in the isolation system divided by lateral displacement.

Isolation System is combination of the structural elements, including each type of installed isolators, structural elements that are used for transferring loads between each isolator and their connection to the structural elements.

Maximum Capable Earthquake (MCE) is an earthquake with 10 percent probability of happening in 100 years with return period of 1000-year. This level of the ground shaking may be happened at the expected site.

Maximum Displacement is the maximum displacement corresponded to the maximum capable earthquake, except displacement related to torsion, which is necessary for designing isolation system.

Total Design Displacement is the displacement corresponded to design base earthquake, including additional torsion displacement, required for designing isolation system.

Total Maximum Displacement is the displacement corresponded to that maximum capable earthquake, including additional torsion displacement, required for checking the stability of isolation system and vertical load stability of each isolator.

Wind-Resistance System is the combination of structural elements that supply stability of the isolated structure against lateral wind load. This stability may be provided by an integer part of isolator or separate devices can be used.

Stability of the Isolation System the stability of the isolation system against vertical loads should be checked for the lateral displacement equal to the maximum total displacement by analysis and test criteria.

6.2.2 Selection of Lateral Response Procedure

Any seismic isolated structure shall be analyzed according to the below criteria.

6.2.2.1 Static Analysis

This method of analysis shall be used for designing isolated buildings that provide below criteria:

1. The structure is located at least 10 kilometers (km) from all active faults.
2. The structure is located on Soil Profile Type S_A, S_B, S_C or S_D [Appendix A].
3. The structure above the isolation interface is equal to or less than four stories, or 65 feet (19.8 m), in height.
4. The effective period of the isolated structure at maximum displacement, T_M, (equation (6.4)) is equal to or less than 3.0 seconds.
5. The effective period of the isolated structure at design displacement, T_D, (equation (6.2)) is greater than three times the elastic, fixed base period of the structure above the isolation system.
6. The structure above the isolation system is of regular configuration.
7. The isolation system is defined by all of the following attributes:

7.1 The effective stiffness of the isolation system at design displacement is greater than one third of the effective stiffness at 20 percent of the design displacement.

7.2 The isolation system is capable of producing a restoring force.

7.3 The isolation system has force-deflection properties which are independent of the rate of loading.

7.4 The isolation system has force-deflection properties which are independent of vertical load and bilateral load.

7.5 The isolation system does not limit maximum capable earthquake displacement to less than C_{VM}/C_{VD} times the total design displacement.

6.2.2.2 Dynamic Analysis

Dynamic analysis may be used for designing isolated buildings that identified below:

1. **Response Spectrum Analysis**: This method of analysis may be used for designing isolated buildings that specified below criteria:

   1.1 The structure is located on Soil Profile Type S_A, S_B, S_C or S_D.

   1.2 The isolation system is defined by all of the attributes specified in static analysis criteria Item7.

2. **Time-History Analysis**: Time-story analysis shall be used for designing any seismic isolated building that doesn't provide response spectrum analysis criteria Item 1.1.

6.2.3 Static Lateral Response Procedure

Every seismic isolated structure should be designed and constructed to resist minimum earthquake displacements and forces that will be discussed in this section.

6.2.3.1 Minimum Lateral Displacement

6.2.3.1.1 Design Displacement

The isolation system that is designed should be stable to minimum lateral displacements which act in each horizontal axes of the structure. This value is obtained of below formula:

$$?_{?} ? \frac{?\frac{?}{???}??_{??}?_{?}}{?_{?}} \qquad ??G?$$

Where, g is ground acceleration, C_{VD} is seismic coefficient as set in Table 16.R [Appendix A], T_D is effective period ,in second, at design displacement, B_D is numerical coefficient related to the effective damping of isolation system at design displacement as set in Table A.16.C [Appendix A].

6.2.3.1.2 Effective Period at the Design D isplacement

At the design displacement, effective period of the seismic isolated structure shall be computed of:

$$?_{?} ? ?? ? \sqrt{\frac{?}{?_{?? ??} ?}} \qquad ??G?$$

Where, W is the total seismic load of the structure, K_{Dmin} is minimum effective stiffness of the isolation system at design displacement in horizontal direction.

6.2.3.1.3 Maximum Displacement

Maximum horizontal displacement at the most critical direction for the isolation system should be calculated of:

$$?_{?} ? \frac{?\frac{?}{???}??_{??}?_{?}}{?_{?}} \qquad ??G?$$

Where, C_{VM} is seismic coefficient as set in Table A.16.G [Appendix A] , T_M is effective period, in second, at maximum displacement, B_M is numerical coefficient related to the effective damping of isolation system at maximum displacement as set in Table A.16.C [Appendix A].

6.2.3.1.4 Effective Period at the Maximum Displacement

At the maximum displacement, effective period of the seismic isolated structure shall be computed of:

$$?_{?} \; ? \; ?? \; ? \sqrt{\frac{?}{?_{? \, ? \, ??} ?}} \qquad ??G?$$

Where, K_{Mmin} is minimum effective stiffness of the isolation system at maximum displacement in horizontal direction.

6.2.3.1.5 Total displacement

Total design displacement (D_{TD}) and total maximum displacement (D_{TM}) shall be include of displacement corresponded to the torsion effects. Total design displacement (D_{TD}) and total maximum displacement (D_{TM}) for an isolation system with uniform stiffness should not be less than the below values:

$$?_{??} \; ? \; ?_{?} ?? \; ? \; ? \frac{???}{?^{?} \, ? \, ?^{?}} ? \qquad ??G?$$

$$?_{??} \; ? \; ?_{?} ?? \; ? \; ? \frac{???}{?^{?} \, ? \, ?^{?}} ? \qquad ??G?$$

Where, D_{TD} is total design displacement, D_{TM} is total maximum displacement, y is the distance between the center of the rigidity of the isolation system and the element of interest, b is the shortest plan dimension and d is the longest plan dimension.

6.2.3.2 Minimum Lateral Forces

6.2.3.2.1 Structural element and isolation system at or below the isolation system

In base isolated structures, the structural elements at or below isolation system shall be designed for the lateral force V_b in accordance with the formula:

$$V_b=K_{Dmax}D_D \qquad (6.7)$$

Where, K_{Dmax} is maximum effective stiffness of the isolation system at the design displacement in the horizontal direction.

6.2.3.2.2 Structural elements above the isolation system

The structural elements above the isolation system shall be designed for the lateral force V_b with reduction factor R_I. This lateral force is called V_s that corresponded with the formula:

$$?_? \; ? \; \frac{?_{????} \; ?_?}{?_?} \qquad ??G?$$

Where, R_I is numerical coefficient related to the type of lateral-force-resistance system above the isolation system as set in Table A.16.E [Appendix A].

6.2.3.3 Vertical Distribution of Force

The lateral force V_s shall be distributed over the height of the structure corresponded with the formula:

$$?_? \; ? \; \frac{?_? \; ? _? \; ?_?}{s_{??}^{?} \; ?_? \; ?_?} \qquad ??G?$$

Where, W and h are weight and height of the corresponded story of the structure respectively.

6.2.4 Dynamic Lateral Response Procedure

As required by previous section, every seismic isolated structure should be designed and constructed to resist minimum earthquake displacements and forces as clarified in this section.

6.2.4.1 Isolation System and Structural Elements Below the Isolation System

The total design displacement of the isolation system shall not be taken as less than 90 percent of D_{TD}. The total maximum displacement of the isolation system shall not be taken as less than 80 percent of D_{TM}. The design lateral shear force on the isolation system and structural elements below the isolation system shall not be taken as less than 90 percent of V_b.

When dynamic analysis is used, design displacement and maximum displacement can be reduced by the formula:

$$? _{? ?} ? \frac{? _?}{\sqrt{? ? ? \; ?\frac{?}{?_?}?^{?}}} \qquad ?? G??$$

$$? _{? ?} ? \frac{? _?}{\sqrt{? ? ? \; ?\frac{?}{?_?}?^{?}}} \qquad ?? G??$$

6.2.5 Step by Step Design Procedure for UBC 97

1. Determine seismic zone factor Z. Establish seismic zone factor by finding the location of project in seismic map and the corresponding seismic zone factor Z from Table 16.I [Appendix A].
2. Determine site soil profile category. Establish the site soil profile type from Table 16.J [Appendix A].
3. Establish seismic source type. For each seismic hazard source, determine the corresponding seismic source type from Table 16.U [Appendix A].
4. Determine near source factor N_a and N_v. For each seismic source type, find corresponding near source factor N_a and N_v from Table 16.S and 16.T [Appendix A].
5. Compute Maximum Capable Earthquake response coefficient M_M. Calculate design base earthquake shaking intensity by multiplying Z and N_v. Use Table A.16.D [Appendix A] to find maximum capable earthquake response coefficient (M_M).

6. Establish seismic coefficients C_{VD} and C_{AD}. Using seismic zone factor (Z) and soil profile type, read seismic coefficients C_{VD} and C_{AD} from Table 16.R and 16.Q respectively [Appendix A].
7. Determine seismic coefficients C_{VM} and C_{AM}. Multiply M_M, Z, N_V and M_M,Z ,N_a that obtain above. With these coefficients and soil profile type, establish seismic coefficients C_{VM} and C_{AM} from Table A.16.G and A.16.F respectively [Appendix A].
8. Determine structural reduction factor R_I. Defined basic structural system and lateral force resisting system, read structural reduction factor R_I from Table A.16.E [Appendix A].
9. Select type of isolator and determine damping coefficients B_D and B_M from Table A.16.C [Appendix A].
10. Select a desired value for period of isolation system.
11. Calculate minimum design lateral displacement D_D from equation (6.1). If the value of this displacement is larger than the desire displacement, a stiffer isolation system is required.
12. Design the selected isolation system according to the above information.
13. Calculate minimum lateral forces V_b and V_s for structural elements below and above the isolation system from equations (6.7) and (6.8).
14. Analyze and design structure with selected isolation system and above information.
15. A model of the isolator is subjected to the prototype test to check stability of the selected isolator under different level of loading.

6.3 Design Procedures for Elastomeric Bearing

6.3.1 Introduction

There are various methods that are used for designing elastomeric bearings. Commonly, design procedures for elastomeric bearings are involved with trial and error procedures. Generally the design procedures for elastomeric bearings are:

1. Calculating service loads DL+LL and DL+LL+E for expected column.
2. Assuming plans dimensions, number of the rubber layers, lead–plug diameter (for LRBs) and finding isolator properties.
3. Computing maximum vertical load capacity of isolator at zero displacement and checking with DL+LL.
4. Determining maximum vertical load capacity of the isolator at DBE displacement and checking with DL+LL+E.
5. The previous procedure (step 4) should be repeated for MCE displacement.

This section starts with design procedure for Lead Rubber Bearings. At the end of this section, an Excel spreadsheet is proposed for designing LRB.

6.3.2 Design Procedures for Lead Rubber Bearing

Among various methods for designing Lead Rubber Bearings, the best and most practical one is proposed by T.E.Kelly et al. This method is used for designing Lead Rubber Bearings in this thesis which is described below.

6.3.2.1 Vertical Stiffness and Load Capacity

The most important factor for calculation vertical stiffness and vertical load capacity of isolator is shape factor which is formulated as follows:

$$?_? ? \frac{?_? ? ?_{??}}{??_? ?_?} \qquad ??G??$$

Where, A_b is bounded area of the rubber A_{pl} is area of the lead core, B_b is bounded plane dimension of the isolator and t_i is layer rubber thickness.

Vertical stiffness is computed as:

? ?? ? ?? ? ? ? / ?? ?? G??

Where, E_c is effective compressive modulus = ? ?? ? ??? ?? ? and A_r that is reduced area of rubber bearing (Figure 6.3), formulated as:

? ? ? ? G ?? ? ??• ? ? ? ? / ? ? ? ? ? ??? ?? G??

? ? ? ? ? ? ? ? ? ?? G??

Where, d is the isolator diameter and ? is applied displacement.

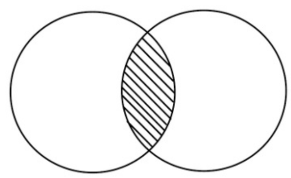


Figure 6.3: Area reduction of circular rubber bearing at ? displacement (T.E.Kelly et al.).

6.3.2.2 Compressive Rated Load Capacity

The vertical load capacity of the isolator is calculated by the summation of shear strains due to the all sources. This value should be checked with ultimate elongation at break of the elastomer (e_u), reduced by appropriate safety factor. The value of shear strain due to the vertical load is called e_{sc}, formulated as:

??? ? ? ? ? ? ? (6.16)

?? ? ? / ? ??? ? (6.17)

Where, e_c is compressive strain, P is the value of the service loads and t_i is rubber layer thickness

Due to the rotation, the value of shear strain is computed as follow:

??? ? ? ? ? ? / ? ? ? ? ? ?? G??

Where, ? is the applied rotation and T_r is total rubber thickness.

Also shear strain due to lateral load is calculated as follow:

$$?_{??} \; ? \; \frac{?}{?_?} \qquad ??G??$$

Seismic isolated building's codes such as UBC 97 don't specify any formulation for calculating vertical load capacity of the isolators. On the other hand, they provide general requirements for isolated structure. Only at AASHTO code, some formulas are proposed which are used in this thesis.

Generally, total shear strain corresponded to the service load DL+LL which is e_{sc} with safety factor 3 should be smaller than e_u. For ultimate load DL+LL+E, total shear strain which is the summation of e_{sc} , e_{sr} and e_{sh} should be smaller than e_u with safety factor 1. Finally critical vertical load capacity is calculated as fallow:

$$?_? \; ? \; \frac{? \, ??_? \, ???_? \, ? \, ?_{??} \, ?}{??_?} \qquad ??G??$$

6.3.2.3 Buckling Load Capacity

Buckling load capacity for the isolators with thick layers of rubber is more critical than others. The procedure for calculating buckling load capacity is described as follows:

$$?_? \; ? \; \frac{? \; ?_?^?}{??} \qquad ??G??$$

Where, I_m is moment of inertia.

$$? _? \; ? \; ??\,?_?\,? ? \;\; ?? \; ? \;\; ? \, ??_{??} \qquad ??G??$$

$$?_? \; ? \;\; ? \, ?? \; ? \;\; ?G?? \; ?_?^? ? \qquad ??G??$$

Where, $?_?$ is height of the bearing free to buckle, $?_{??}$ is thickness of internal shims and $?_?$ is buckling modulus.

Constant T, R, and Q are computed of below formulas:

? ? ? ? ? ? ? ? ? ? (6.??)

? ? ? ? ? ? (6.??)

? ? ? ? ? (6.??)

Finally buckling load is calculated as fallow:

? (6.??)

6.3.2.4 Lateral Stiffness and Hysteresis Parameters

Lead Rubber Bearings produce a force-deflection curve as shown in Figure 6.4

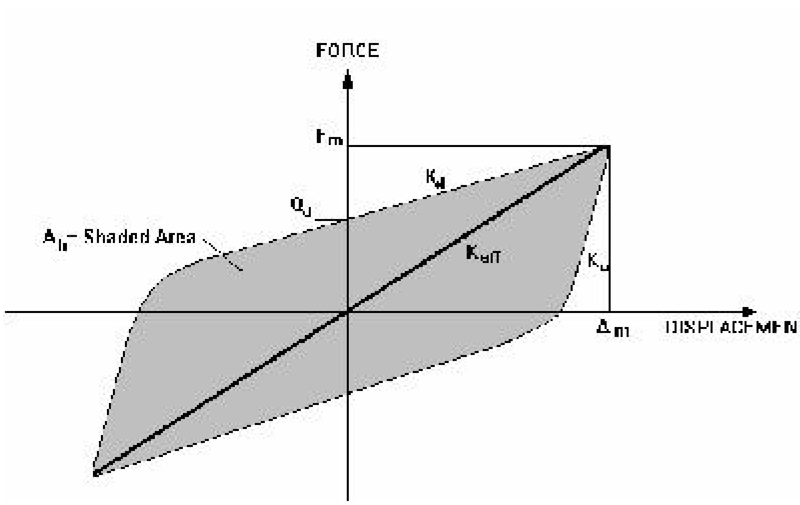


Figure 6.4: Force-deflection curve for rubber bearing (T.E.Kelly et al.).

For calculating lateral stiffness, the first stage is computing characteristic strength (Q_D) related to the lead core that formulated as follows:

? ? ? ? ? ? ? ? (6.??)

Where, s_y is lead yield stress and ? ?? is area of lead core.

Next stage is calculating lateral stiffness after yield stage (K_r) and elastic lateral stiffness (K_u):

? ? ? ? ? ? ? ? ? ? (6.??)

$$?_{?}\ ?\ \ ???_{?} \qquad ??(??)$$

Where, $?_{?}$ is shear modulus of rubber at shear strain ?.

At the ? displacement, shear strain force is formulated as follow:

$$?_{?}\ ?\ \ ?_{?}\ ?\ \ ?_{?}? \qquad ??(??)$$

Lastly, effective stiffness and effective period can be calculated of below formulas:

$$?_{???}\ ?\ \frac{?_{?}}{?} \qquad ??(??)$$

$$?_{?}\ ?\ \ ???\sqrt{\frac{?}{?\ s\ ?_{???}}} \qquad ??(??)$$

Effective damping is calculated easier at the compare to effective stiffness as illustrated below:

$$?_{?}\ ?\ \ ??_{?}??_{?}\ \ ?\ \ ?_{?}? \qquad ??(??)$$

$$?\ \ ?\ \ \frac{?}{??}?\frac{?_{?}}{?_{???}?^{?}}? \qquad ??(??)$$

$$?_{?}\ ?\ \frac{?_{?}\ ?_{?}^{?}}{??^{?}?} \qquad ??(??)$$

Where, $?_{?}$ is area of hysteric loop, $?_{?}$ and $?_{?}$ are maximum applied and yield displacement $?_{?}$ is spectral acceleration at effective period $?_{?}$ and B is numerical coefficient related to ? and calculated of Table A.16.C [Appendix A].

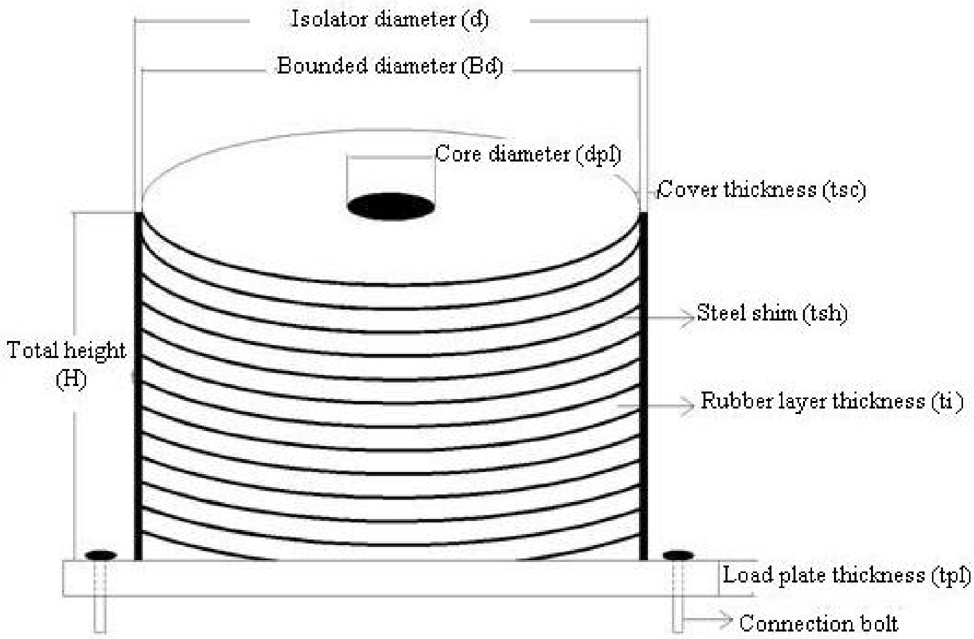


Figure 6.5: Lead Rubber Bearing's details.

6.3.2.5 Excel Spreadsheet for Designing Lead Rubber Bearing

In this section an Excel spreadsheet is proposed for designing Lead Rubber Bearings. The accuracy of this program is checked by an example of a heavy-weight isolated building which is solved by T.E.Kelly et al. in Seismic Isolation for Designers and Structural Engineer's book, page 129 (T.E.Kelly et al.).

Table 6.1: Characteristics of the building and selected isolator properties.

Building Characteristics		Isolator Properties	
Z	0.4	B	870
C_A	0.4	ti	10
C_V	0.48	N	21
N_a	1	d_{pl}	175
N_v M_M	1.2 1.21	t_{sc} t_{sh}	10 3
R_I	2	t_{pl}	40

Table 6.1: Characteristics of the building and selected isolator properties (continued).

I	1	T_r	210
C_{AM}	0.48	H	350
C_{VM}	0.58		
b	32.00		
d	70.00		
Max DL+LL	4948		
Max DL+LL+E	8358		
Seismic weight	102114		

Table 6.2: Seismic performance of isolator under gravity load, DBE and MCE.

Parameter	**Gravity Load**	**DBE**	**MCE**
D	----------	185.3	255
?	----------	30.55 %	26.76 %
P_{cr}	23188	15263.95	12370.34
Total e	1.09	3.89	4.96

This spreadsheet consists of 6 different tables (each table includes of input and output main columns). At the first table, basic characteristics of the isolator are calculated. At the second table, the stability of the isolator is checked under service gravity loads at zero displacement automatically by computing vertical load capacity. At the third and fourth tables, seismic performance for Design Base Earthquake (DBE) and Maximum Credible Earthquake (MCE) is calculated respectively. Finally, the last two tables check the stability of the isolator against service loads (DL+LL+E) at total design displacement and total maximum displacement automatically by computing vertical load capacity.

Table 6.3: Bearing properties.

Input			Output	
B	870		594166.5000	Ag
tsc	10		850.0000	Bb
dpl	175		567162.5000	Ab
N	21		24040.6250	Apl
t1	10		543121.8750	Abn
sy G	0.008 0.0004		210.0000 2669.0000	Tr p
?NS	0		20.3493	Si
tsh	3		192.3250	Qd
E	0.00135		0.0004	?50
			1.0860	Kr
			214.1688	Fy
			20.1149 28107788990.6250	?y I
			270.0000	Hr
			0.4161	Eb
			15038947150.7970	constant T
			293.2076	constant R
			0.0116	constant Q
			850.0000	?
			567450.1731	Ar
			10.6473	Ku
Bearing Properties				

Table 6.4: Gravity load capacity of isolator.

Input			**Output**	
?u	6.5		0.9741	Ec
f	0.33		55272.6802	Kvi
Pdl+ll	4948		0.0090	?c
?NS	0		1.0930	?sc
E?	1.5		0.0000	?sh
k	0.87		0.0000	?sr
			1.1019	?
			2.1450	Allowable ?
			23183.2669	Pcr
			TRUE	? check
			TRUE	P check
			0.0003	Adjusted G
			0.8542	Adjusted K*r
			Vertical Stiffness Calculation	
			2632.032393	Kv
			1595.781441	Vertical Stiffness Kv
Gravity load Capacity				

Table 6.5: Seismic performance for DBE.

Input			**Output**	
# Isolators	27		185.0000	Assume a displacement
W	102114		350.3481	F
B	1.82		1.8938	Ke
Cv	0.48		51.1319	Total Ke
g	9810		10.4092	M
			2.8335	Te
			126846.1381	Ah
			31.1635	Damping(%)
			0.0931	SA
			185.8838	SD
Seismic Performance for DBE				

Table 6.6: Seismic performance for MCE.

Input		Output	
# Isolators	27	255.4000	Assume a displacement
W	102114	410.4823	F
B	1.74	1.6072	Ke
CVM	0.581	43.3948	Total Ke
g	9810	10.4092	M
		3.0757	Te
		181004.8581	Ah
		27.4926	Damping(%)
		0.1086	SA
		255.4617	SD
Seismic Performance for MCE			

Table 6.7: Load capacity at DBE.

Input		Output	
f DBE	0.75	230.8800	D TD
P dl+ll+E	8358	0.9741	Ec
(D TD)/(DD)	1.248	818.0430	?
		373642.7842	Ar
		36394.8045	Kvi
		0.0230	?c
		2.8039 1.0994	?sc ?sh
		0.0000	?sr
		3.9263	?
		4.8750	Allowable ?
		15265.2353	Pcr
		TRUE	? check
		TRUE	P check
Load Capacity at DBE			

Table 6.8: Load capacity at MCE.

Input			Output	
f MCE	1		318.7392	DTM
P dl+ll+E	8358		0.9741	Ec
(D TM)/(DM)	1.248		787.9755	?
			303012.4560	Ar
			29515.0330	Kvi
			0.0283	?c
			3.4575	?sc
			1.5178	?sh
			0.0000	?sr
			5.0036	?
			6.5000	Allowable ?
			12379.6220	Pcr
			TRUE	? check
			TRUE	P check
Load Capacity at MCE				

6.3.3 Design Procedures for High Damping Rubber Bearing

Generally, designing High Damping Rubber Bearings follows the same procedure as well as Lead Rubber Bearing with this difference that in High Damping Rubber Bearings, damping is provided by extra additive to rubber instead of inserted lead cores. The rubber properties of High Damping Rubber Bearings are changed from manufacturer to manufacturer. Therefore, the properties which are used for designing High Damping Rubber Bearings in this thesis are derived of Bridgestone Corporation.

Among various methods that are applied for designing High Damping Rubber Bearings, the most practical one is proposed by Chen and Scawthorn (2002) which is used for designing High Damping Rubber Bearing in this thesis. These design procedures are described step by step briefly.

Step 1: specifying soil condition, selecting appropriate value of shear strain (? ? ? ?) and defining desire values of damping and period for the isolated structure.

Step 2: calculating horizontal effective stiffness and design displacement of below formulas:

? ??? ? ? / ? ? ?? / ?? ? ?   ?? G? ??

? ? ? ? ? / ??? ? ?? ?? / ??   ?? G? ??

Step 3: determining total rubber high, formulated as follows:

?? ? ?? / ????   ?? G? ??

Step 4: choosing rubber properties of Table 6.9 that is proposed by Bridgestone Company.

Table 6.9: Relation of Rubber Hardness and Materials (Chen and Scawthorn, 2002).

Rubber Hardness IRHD±2	**Young Modulus E (N/cm^2)**	**Shear Modulus G (N/cm^2)**	**Modified Factor K**
30	92	30	0.93
35	118	37	0.89
40	150	45	0.85
45	180	54	0.8
50	220	64	0.73
55	325	81	0.64
60	445	106	0.57
65	585	137	0.54
70	735	173	0.53
75	940	222	0.52

Step 5: calculating total area and thickness of layers:

5.1: determining shape factor (S_i) and compressive modulus E_c:

?? / ?? ? ? G? ? ?? ??? ? / ? ? 400   for S_i >10   (6.40)

?? ? ? G? ? ? ?? ??? ?   (6.41)

Chen and Scawthorn (2002) consider that "The stiffness ratio K_v/K_h is require to be greater than 400 for $S_i > 10$, since the P-Delta effect has been ignored in computing horizontal stiffness K_h".

5.2: Calculating cross-section area A_0 based on axial stress s_c under axial load P_{DL+LL}:

$$s_c ? \frac{? ? ?? ??}{? ?} < 7.8\ (MN/m^2) \quad (6.42)$$

5.3: Obtaining cross-section area A_1 based on shear strain at axial load P_{DL+LL}:

?? ? ?? ???? ?? / ????? ? ?? / ? ?? G??

Where, e_b is elongation of rubber at break and safety factor 3 is derived of AASHTO code.

5.4: Computing minimum area A_{sf} based on shear failure:

? ? ? ???G?? / ? ?? ?? G??

5.5: determining design cross-section area (A):

? ? • ?? ?? ? ?? ? ?? ?? ? ?? G??

5.6: assuming bearing diameter ???, obtaining cross-area section and checking with reduce area ?? as formulated below:

?? ? ?? / ? ?? ? ??• ?? ?? G??

? ? ? ???? ? ? ?? / ? ? ?? G??

5.7: determining individual rubber layer thickness ti:

?? ? ? / ??? ?? G??

5.8: obtaining total height h of the individual bearing:

? ? ?? ? ?? ? ? ???? ? ??? ??G??

Where, n is number of rubber layers, ??? is thickness of internal steel shim and ??? is thickness of the end load plates.

Step 6: checking stability of the isolator against vertical load P_{DL+LL} at zero displacement:

???????? ? ??????? / ??G ? ?? / ? ?? G??

?? ? ????? / ? ? ?? ? ?G?G / ?G?? ?? G??

Step 7: verifying stability of the isolator under vertical load $P_{DL+LL+E}$ at ? displacement:

??? ? ????????? / ???G?? ?? G??

??? ? ? / ?? ?? G??

? ? ????G / ?? ? ?? ??G??

??? ? ??G / ?G?G? ??G??

Where, ??? is shear strain under compression, ??? is shear strain under earthquake, ??? is shear strain under rotation and ? is rotation angle of the bearing induced by earthquake.

??? ? ???? ??? ? ?? / ?G? ??G??

Where, safety factor 1.33 is derived of AASHTO code.

6.4 Design Procedures for Friction Pendulum Systems

Friction Pendulum Systems are the newest isolation system in the world. Generally, they consist of three different types: Single Pendulum Bearings, Double Pendulum Bearings and Triple Pendulum Bearings. All of these bearings implement characteristics of pendulum systems to shift isolated structural period and to provide damping.

Design procedures for Friction Pendulum Systems are easier than elastomeric bearings because they don't need to be checked for service load's capacities. On the other hand, it is assumed that these isolation systems provide high resistance under vertical loads effects.

The design procedures for friction pendulum bearings are described step by step briefly:

Step 1: determining desire value of structural isolated period and obtaining disk radius of below formula:

$$? \; ? \; ??? \overline{\frac{?}{?}} \qquad (??G??)$$

Where, T is structural isolated period and R is radius of disk.

As it is shown of above formula and Figure 6.6, period of isolated structure is independent of structural mass.

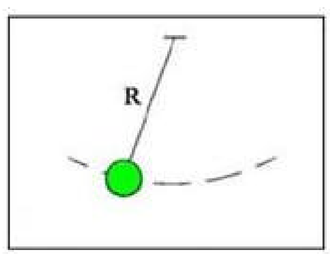


Figure 6.6: Period of pendulum system (EPS).

Step 2: calculating design displacement of 6.38.

Step 3: obtaining damping of isolation system, formulated as follow:

$$? \; ? \; \frac{??}{? \; ?\frac{?}{?} ? \; ??} \qquad ??G??$$

Where, μ is friction coefficient

Step 4: computing horizontal and effective stiffness:

$$?_{?} \; ? \; \frac{?}{?} \qquad ??G??$$

$$?_{???} \; ? \; \frac{?}{?} ? \; \frac{?G?}{?_{?}} \qquad ??G??$$

Where, $?_{?}$ is horizontal stiffness, $?_{???}$ is effective stiffness and W is weight of the building.

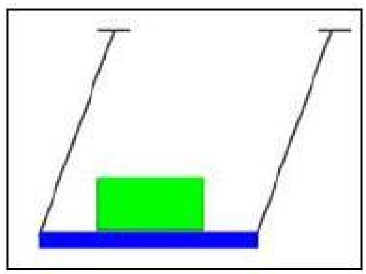


Figure 6.7: Stiffness of pendulum system (EPS).

Step 5: estimating vertical displacement and disc diameter:

$$?_{?} \; ? \; \frac{?_{?}^{?}}{??} \qquad ??G??$$

$$\frac{?\frac{?}{?}?^{?}}{??} ? \; ?_{?} \qquad ??G??$$

Where, $?_{?}$ is vertical displacement and d shows disc diameter

Step 6: checking recentering force under earthquake load

$$\frac{?}{?} ? \; ? \qquad ??G??$$

CHAPTER 7

ANALYSIS

7.1 Introduction

In this chapter, three reinforced concrete buildings with different heights (three, six and nine stories) are analyzed using three types of isolators (Lead Rubber Bearing, High damping Rubber Bearing and Friction Pendulum Systems) to consider the optimum one according to the seismic demands. Excel spreadsheets are proposed in this thesis for design Lead Rubber Bearings and High Damping Rubber Bearings in order to save time. In addition, a 3-story fixed base and an optimum isolated structure are analyzed to clarify differences between the performances of these two types of structures. Furthermore, these two buildings are designed and their materials are compared to consider differences between them. ETABS (version 9.1.4) computer program and UBC 97 code are used in this research for modeling and designing structures. These information and design details used in this modeling were acquired during participation in this project.

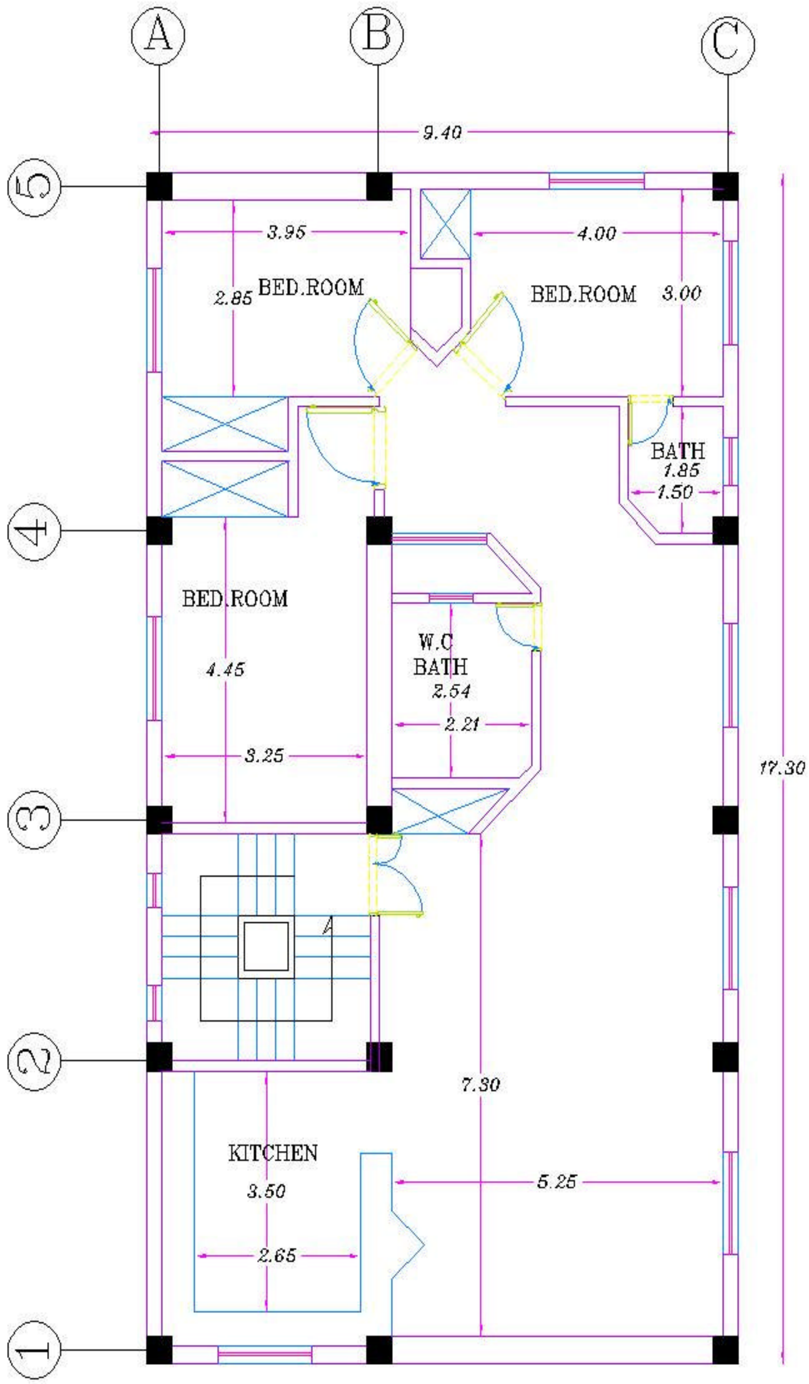
7.2 Selection of the Isolation System

Generally, when comparing isolators, they should be fixed by same conditions. The structural shifted period is considered to be one of the most important factors in the isolation systems (T.E.Kelly, 2001). For the low and medium rise buildings, the application of isolation systems is more appropriate when the structural shifted period falls in the range of the 1.5-3 seconds ($1.5= T_i = 3$) (Colunga and Cruz, 2006). In another study, Providakis (2008) carried out the comparison of the Friction Pendulum Systems by defining period range 1-2 seconds, where the most of the isolation's periods (FPS) of base-isolated structures lies. In addition T.E.Kelly et al. complete their research by defining a fix period for different isolation systems. A conclusion of this short literature is that a period around 2 seconds is selected for designing different isolation systems and all structures as a primary factor.

According to the UBC 97 code, structural elements above the isolation system should be designed for elastic force, which is reduced by a ductility factor 2. In the base isolated structures, this elastic force depends on total effective stiffness of the isolation system and design displacement. Consequently, to design a base isolated structure economically, these two values should be minimized conservatively. In this research, all of the isolators were designed according to the provided minimum elastic force as second factor.

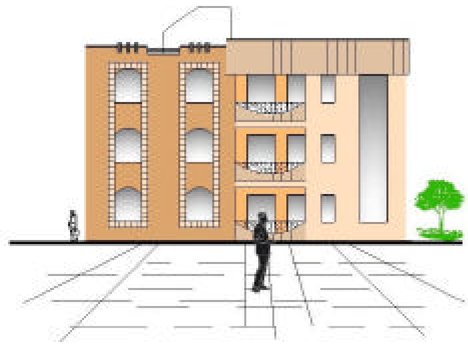
7.3 Description of the Example Buildings

The example buildings are regular in plane with moment resistance concrete frames in three different heights (3-story, 6-story and 9-story) which are designed by the author of this thesis. These residential buildings are located on high seismic zone and rock soils. The dimensions of the buildings are 17.30 m × 9.4 m in plan and 2.8 m in height for the first floor and 3.2 m for other floors. The plan layout and facades of the example buildings are shown at Figure 7.1.a,b,c,d.



(a) Plan of the example buildings.

Figure 7.1.a: Details of the example buildings.



(b) Facade of 3-story building.

(c) Facade of 6-story building.

(d) Facade of 9-story building.

Figure 7.1,b,c,d: Details of the example buildings (continued).

7.4 Materials Definitions

The example buildings are moment resistances concrete frame that providing concrete characteristics as illustrated in Table 7.1 and 7.2.

Table 7.1: Analysis parameters.

Parameter	Value
Mass per unit volume (N.s^2/m)	2400
Weight per unit volume (N/m)	24000
Modulus of elasticity (N/m^2)	2.18×10^{10}
Poisson ratio	0.2

Table 7.2: Design parameters.

Parameter	Value
Specified concrete compressive strength, $?_?^?$ (N/mm^2)	20
Bending reinforce yeild stress, f_y (N/mm^2)	400
Shear reinforce yeild stress f_{ys}, (N/mm^2)	300

7.5 Applied Loads

All of the structures are subjected to the vertical and lateral loads. On the other hand, dead load and live load are applied as vertical loads while earthquake loads are considered as lateral loads.

7.5.1 Estimatio n of the Dead Load

ETABS computer program calculates weight of the concrete structural elements automatically. Therefore, the weights of the non-structural elements are additional dead loads that should be applied on the constructed model. These additional loads including tile loads and partition wall loads which should be applied on slabs. In the

Figure 7.2 and 7.3, sections of the floors are shown for calculating additional dead loads on slabs.

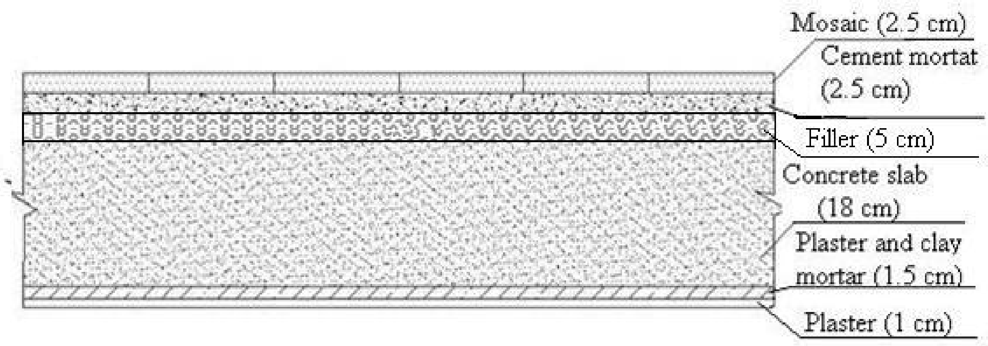


Figure 7.2: Story floor details.

Table 7.3: Dead load calculation for story floors.

Parameter	Value (k N/m^2)
Mosaic	22.5 × 0.025 = 0.56
Cement mortar	21 × 0.025 = 0.52
Filler	6 × 0.05 = 0.3
Concrete slab	24 × 0.18 = 4.32
Plaster and clay mortar	16 × 0.015 = 0.24
Plaster	13 × 0.01 = 0.13
Summation	6.07
Partition	1

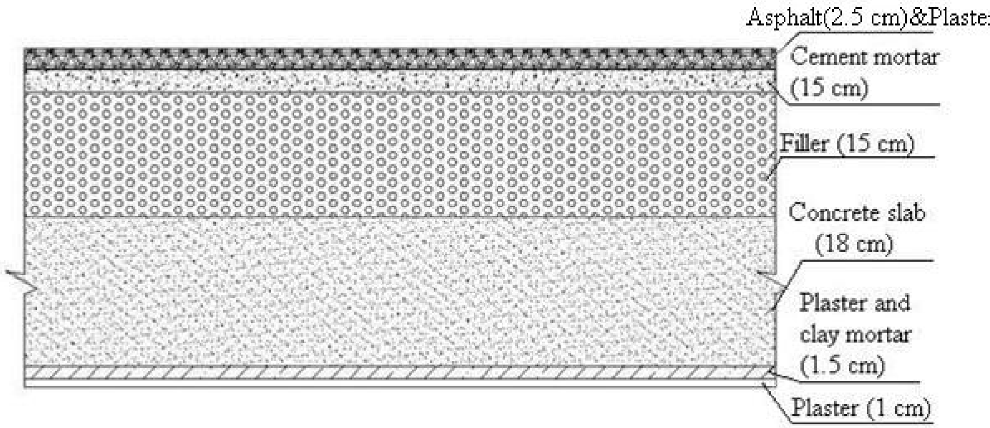


Figure 7.3: Roof floor details.

Table 7.4: Dead load calculation for roof floor.

Parameter	Value (kN/m^2)
Asphalt	22 × 0.025 = 0.55
Plaster	0.15
Cement mortar	21 × 0.015 = 0.315
Filler	6 × 0.15 = 0.90
Concrete slab Plaster and clay mortar	24 × 0.18 = 4.32 16 × 0.015 = 0.24
Plaster	13 × 0.01 = 0.13
Summation	6.6
Partition	------

7.5.2 Estimation of the Live Load

According to the Table 16-A UBC 97 code [Appendix A], for residential occupancies the value of live load is assumed 1.9 (kN/m^2) for all stories.

7.5.3 Estimation of the Earthquake Load

All the example buildings are located on high seismic zone in a distance of 10 kilometer to the known seismic source on rock soils. According to these assumptions, seismic coefficients are calculated based on chapter 16 and its corresponded Appendix, division IV, UBC 97 code as shown in the Table 7.4.

Table 7.5: Seismic definition.

Seismic zone factor (zone 4)	Table 16-I [Appendix A]	0.4
Soil profile type	Table 16-J [Appendix A]	S_B (Rock)
Seismic source type	Table 16-U [Appendix A]	A
Near source factor (N_a)	Table 16-S [Appendix A]	1
Near source factor (N_v)	Table 16-T [Appendix A]	1.2
Seismic coefficient (C_A)	Table 16-Q [Appendix A]	0.4
Seismic coefficient (C_V)	Table 16-R [Appendix A]	0.48
Maximum capable earthquake response coefficient (M_M)	Table A-16-D [Appendix A]	1.21
M_M Z N_a	Calculate	0.484
M_M Z N_v	Calculate	0.58
Seismic coefficient (C_{AM})	Table A-16-F [Appendix A]	0.484
Seismic coefficient (C_{VM})	Table A-16-G [Appendix A]	0.58
Importance factor (I)	Fix	1
Lateral force coefficient (R_I)	Table A-16-E [Appendix A]	2
Fixed base lateral force coefficient (R)	Table 16-N [Appendix A]	5.5

Dynamic response spectrum analysis, as a linear elastic analysis method, is used for the estimation of the earthquake load on the example buildings. This method of analysis can provide approximate results for linear and non-linear isolators that are precise sufficiently (Chopra, 2001). Still this method of analysis is recommended in design codes such as Uniform Building Code (UBC 97) and International Conference of Building Officials 1997 for designing seismic isolated structures. On the other hand, in this method, non-linearity behavior of the isolators is modeled by bilinear behavior, which are linearized by effective stiffness and effective damping (Figure 7.4). As illustrated in Figure 7.5, each isolator provides vertical stiffness in vertical direction, horizontal effective stiffness and effective damping in horizontal direction. In fact, the horizontal effective stiffness that also called secant stiffness, is defined as the slope of a line from point O to point B (Figure 7.4) (Zou, 2008).

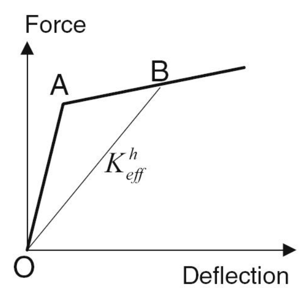


Figure 7.4: Bilinear modeling of non-linear isolator (Zou, 2008).

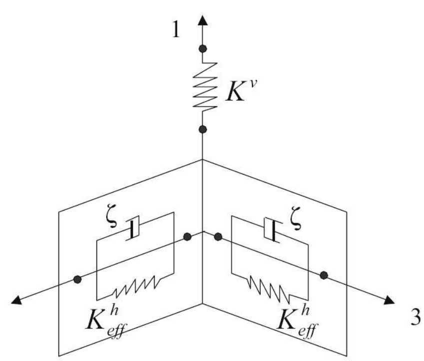


Figure 7.5: Definition of linear and nonlinear isolator (Zou, 2008).

UBC 97 code defines specific spectrums for different seismic zones and soil types. These standardized spectrums are derived from many records for specific zones. This is one of the most important advantage for this method of analysis while in the other methods such as time history analysis, no one can guaranty that future earthquakes present the same records as defined.

In Figure 7.6, design elastic response spectrum that is used for estimating earthquake load in the example buildings, is defined according to the UBC 97 code (Chapter16) for 5 % structural damping, as follow:

From Table 7.4, seismic coefficient (C_A) is derived as 0.4. According to the UBC 97 code (Chapter 16), maximum elastic spectrum acceleration, $?_?$ and $?_?$ are calculated as follow:

maximum spectrum acceleration = 2.5 C_A= 2.5× 0.4 = 1

$?_? ? \frac{?_?}{?G ?_?} ? \frac{?G ?}{?G ? ?G} ? ?G?$ $\quad\quad$ $?_? ? ?G?_? ? ?G ? ?G? ? ?G??$

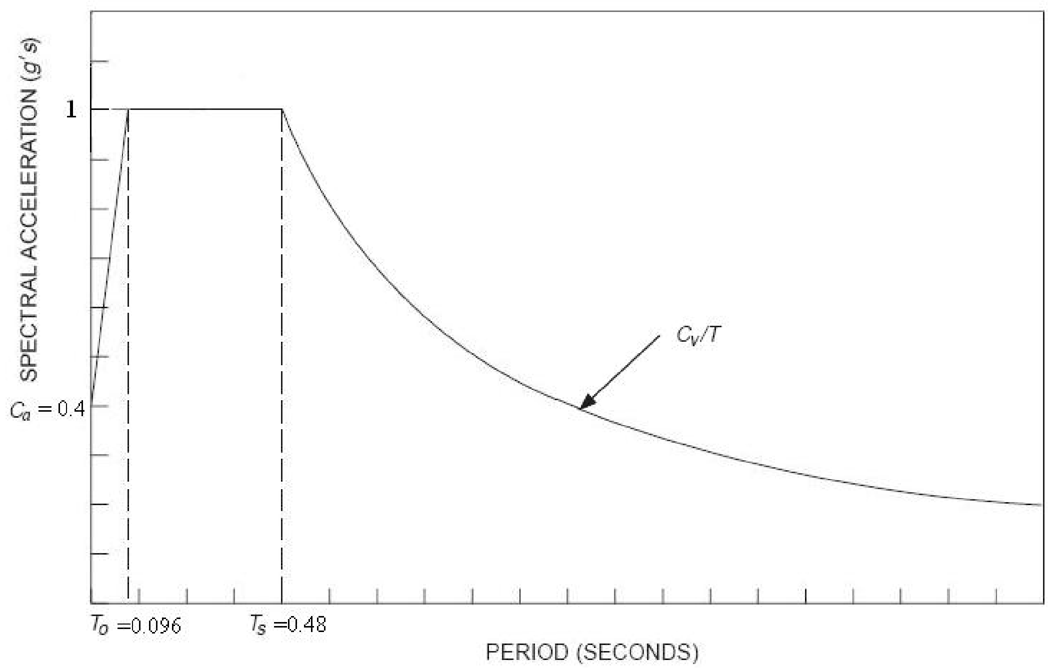


Figure 7.6: Design response spectra.

7.6 Various Assumptions

Various assumptions are made for the structures under consideration:

1. Structures stay in elastic limit during earthquake excitation. Since propose of applying isolation system in structure is reduced earthquake forces, this will be considered as a reasonable assumption.
2. Floors in each story are assumed as rigid diaphragms.
3. Force-deflection behaviors of the superstructures are assumed as linear with viscous damping.
4. The structures are subjected to the horizontal components of ground shakings. On the other hand, the vertical effects of the earthquake are negligible.
5. The non-linearity behavior of the isolation systems is modeled by bilinear behavior and then linearized by effective stiffness and effective damping.

7.7 Analysis Results

In this section, three reinforced concrete buildings with different heights (three, six and nine stories) are analyzed by three various isolators (Lead Rubber Bearing, High damping Rubber Bearing and Friction Pendulum Systems) to come up with the optimum cases according to the seismic demands. Microsoft Excel spreadsheet is utilized in order to design Lead Rubber Bearings and High Damping Rubber Bearings. All of the isolation systems are designed according to the fix period 2 second and minimum provided elastic base shear. The final results for each isolator derived in Excel are presented. Furthermore, running the ETABS program for each building, the value of the story accelerations, isolator displacements and their seismic coefficients are shown afterwards in the graphs for each type of isolator in order to reach the optimum one.

On the other hand, a three story fixed base and optimum isolated structure are analyzed in order to clarify differences between performances of these two types of structures. In addition to what has already been done, these two buildings are designed and their materials are compared to reveal the differences.

7.7.1 Three Story Building

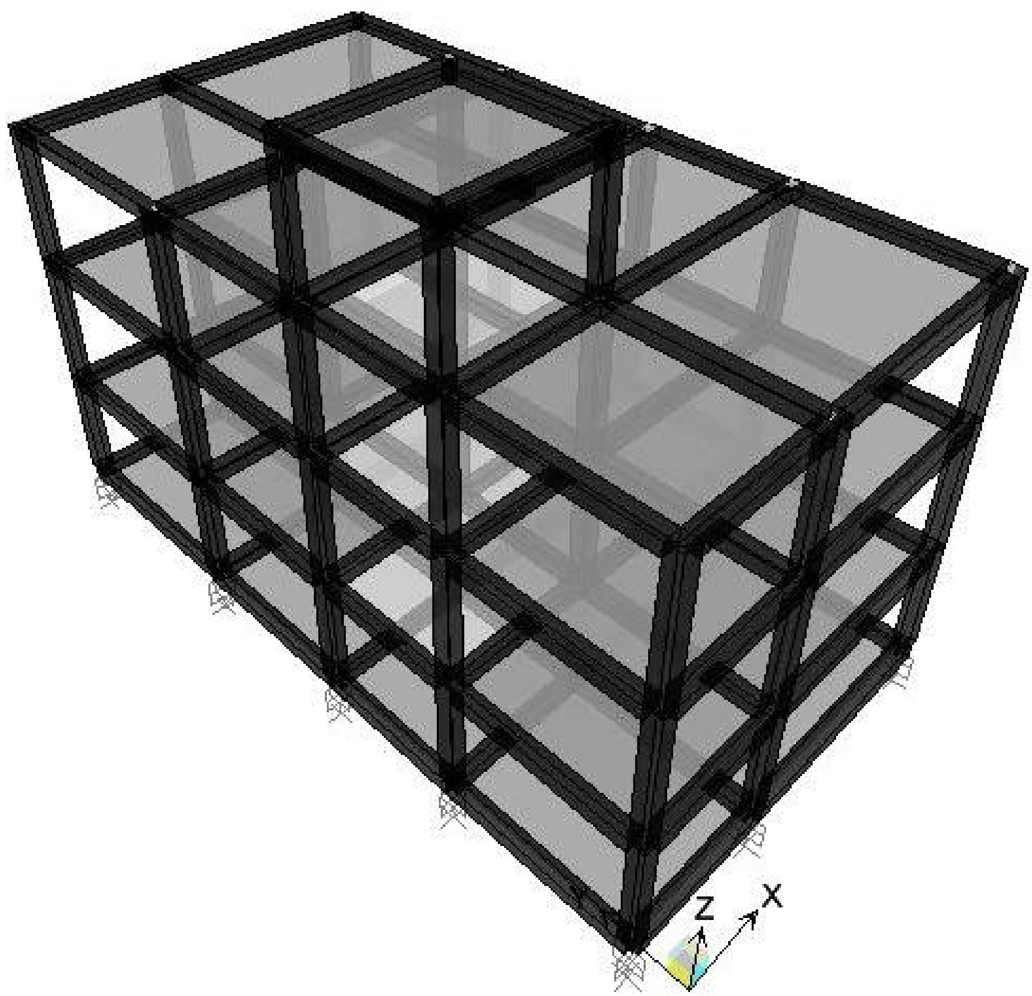


Figure 7.7: 3-D model of 3-story isolated building.

7.7.1.1 Design Result for Lead Rubber Bearing

In this section, design results for Lead Rubber Bearing in the 3-story base isolated building are shown in the different tables.

Table 7.6: Lead Rubber Bearing properties.

Input			Output	
B	500		196250.0000	Ag
tsc	10		480.0000	Bb
dpl	60		180864.0000	Ab
N	21		2826.0000	Apl
t1	11		178038.0000	Abn
sy	0.008		231.0000	Tr
G	0.0004		1507.2000	p
?NS	0		10.7386	Si
tsh	3		22.6080	Qd
E	0.00135		0.0004	?50
			0.3349	Kr
			25.9732	Fy
			10.0474	?y
			3066406250.0000	I_m
			291.0000	Hr
			0.1169	Eb
			451432431.4660	constant T
			97.4656	constant R
			0.0108	constant Q
			480.0000	?
			180955.7368	Ar
			2.5851	Ku
Bearing Properties				

Table 7.7: Gravity load capacity for Lead Rubber Bearing.

Input		Output	
?u	6.5	0.2722	Ec
f	0.33	4478.3701	Kvi
Pdl+ll	606	0.0123	?c
?NS	0	0.7926	?sc
E?	1.5	0.0000	?sh
k	0.87	0.0000	?sr
		0.8049	?
		2.1450	Allowable ?
		2042.5438	Pcr
		TRUE	? check
		TRUE	P check
		0.0003	Adjusted G
		0.2356	Adjusted K*r
		Vertical Stiffness Calculation	
		213.2557189	Kv
		180.4975025	Vertical Stiffness Kv
Gravity load Capacity			

Table 7.8: Seismic performance for Design Base Earthquake.

Input		Output	
# Isolators	15	157.0000	Assume a displacement
W	5908	59.5913	F
B	1.56	0.3796	Ke
Cv	0.48	5.6934	Total Ke
g	9810	0.6022	M
		2.0425	Te
		13289.2143	Ah
		22.6182	Damping(%)
		0.1506	SA
		156.3236	SD
Seismic Performance for DBE			

Table 7.9: Seismic performance for Maximum Capable Earthquake.

Input			Output	
# Isolators	15		206.0000	Assume a displacement
W	5908		71.1338	F
B	1.5		0.3453	Ke
CVM	0.58		5.1796	Total Ke
g	9810		0.6022	M
			2.1414	Te
			17720.3823	Ah
			19.25	Damping(%)
			0.1806	SA
			205.9594	SD
Seismic Performance for MCE				

Table 7.10: Load capacity at Design Base Earthquake.

Input			Output	
f DBE	0.75		188.4000	D TD
P dl+ll+E	638		0.2722	Ec
(D TD)/(DD)	1.2		441.4810	?
			92902.5239	Ar
			2299.1915	Kvi
			0.0252	?c
			1.6254	?sc
			0.8156	?sh
			0.0000	?sr
			2.4662	?
			4.8750	Allowable ?
			1048.6403	Pcr
			TRUE	? check
			TRUE	P check
Load Capacity at DBE				

Table 7.11: Load capacity at Maximum Credible Earthquake.

Input			Output	
f MCE	1		247.20	DTM
P dl+ll+E	645		0.2722	Ec
(D TM)/(DM)	1.2		411.4513	?
			67776.2768	Ar
			1677.3563	Kvi
			0.0350	?c
			2.2524	?sc
			1.0701	?sh
			0.0000	?sr
			3.3575	?
			6.5000	Allowable ?
			765.0269	Pcr
			TRUE	? check
			TRUE	P check
Load Capacity at MCE				

Table 7. 12: ETABS output for Design Base Earthquake.

Output	
Spring effective stiffness (KE2)	0.3795
Spring effective stiffness (KE3)	0.3795
Spring effective damping ratio(DE2)	0.1761
Spring effective damping ratio(DE3)	0.1761
Spring stiffness along axial 1 (K1)	180.4975
initial spring stiffness (K2)	2.5850
initial spring stiffness (K2) Yield force	2.5850 25.9732
Post-Yield stiffness ratio	0.0911
Etabs Output for DBE	

Table 7.13: ETABS output for Maximum Credible Earthquake.

Output	
Spring effective stiffness (KE2)	0.3453
Spring effective stiffness (KE3)	0.3453
Spring effective damping ratio(DE2)	0.1427
Spring effective damping ratio(DE3)	0.1427
Spring stiffness along axial 1 (K1)	180.4975
initial spring stiffness (K2)	2.5850
initial spring stiffness (K2)	2.5850
Yield force	25.9732
Post-Yield stiffness ratio	0.0911
Etabs Output for MCE	

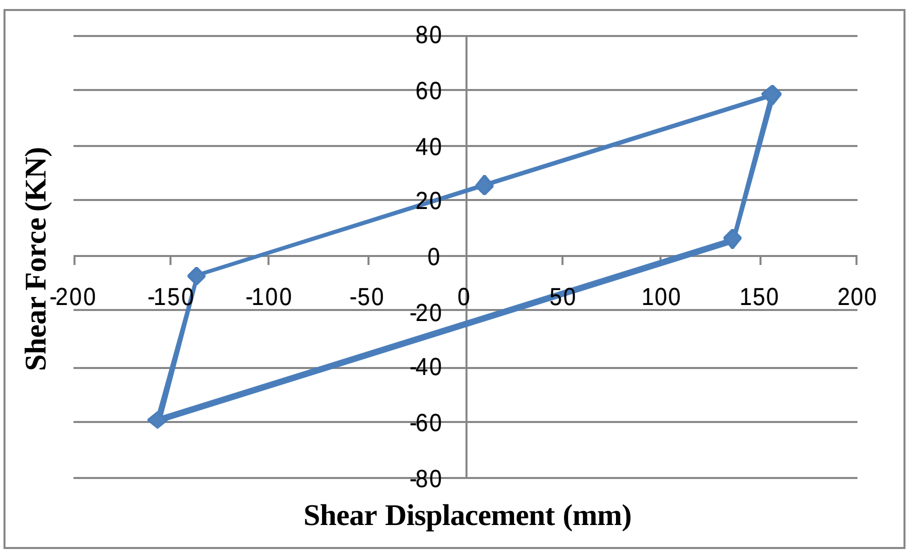


Figure 7.8: Hysteresis curve for LRB in 3-story isolated building.

7.7.1.2 Design Result for High Damping Rubber Bearing

In this section, design results for High Damping Rubber Bearing in the 3-story base isolated building are shown in the Table 7.14.

Table 7.14: Design results for High Damping Rubber Bearing.

Input		Output	
P DL+LL	606.0000	0.6091	Kem
g	9.8100	176.8834	Dd
T	2.0000	5.9379	Total Ke
B	1.3500	**Design Procedure**	
Cv	0.4800	212.2601	D(TD)
W	5908.0000	117.9223	tr
?max	1.5000	125.0000	Use tr
E	4.4500	**Calculate the Area (A) and Thickness (t)**	
G	1.0600		
k	0.5700	9.0941	min S
€b	500.0000	18.0000	Use S
t steel	2.0000	1648.1020	Ec
Cover	25.0000	0.0773	Min A0 (m2)
P DL+LL+E	642.0000	0.0238	Min A1 (m2)
b (m)	17.0000	0.0718	Min Asf (m2)
d (m)	9.0000		
D(TD)/D(D)	1.2000	0.0773	Min A (m2)
TM	2.2000	510.0000	Use d
BM	1.3200	0.2042	A (m2)
CVM	0.5800	TRUE	Check A
P DL+LL+E (MCE)	650.0000	130.8770	ß
		0.0977	A r
		7.0833	Rubber layer thickness t
		8.0000	Use ti
		15.6250	N
		204.2500	h
		Shear strain and Vertical Load Capacity	
		0.1945	?dl+ll
		1.6667	€
		TRUE	Check ?

Table 7.14: Design results for High Damping Rubber Bearing (continued).

2967.9912	σc
42739.2000	σ max
TRUE	Check σ
Earthquake Load Capacity at DBE	
0.4306	γsc
1.6981	γeq
0.0059	θ
0.7610	γ sr
2.8897	γ total
3.7500	γ max
TRUE	Check γ
2012.7446	Kvi
128.8157	Kv
Earthquake Load Capacity at MCE	
240.4509	DM
288.5411	DTM
111.1452	β
0.0654	A re
0.6513	γsc
2.3083	γeq
0.0080	θ
1.0345	γ sr
3.9941	γ total
5.0000	γ max
TRUE	Check γ

7.7.1.3 Design Result for Friction Pendulum System

According to the fixed period 2 second and equation (6.56), radius of disk calculated as:

? ? ??? ?/? ? ? ? ??? ?/?G? ? ? ? ? ?

Total horizontal stiffness is computed by equation (6.58)

? ?? ? ?/? ? ????/???? ? ?G?? ?? ?? ?

For estimating damping and design displacement, a value of damping is assumed, then design displacement is calculated by equation (6.38). At the end, the value of damping that is clarified by equation (6.57), should agree with the assumed damping ± 1%.

Assume that provided damping is 28%

?? ? ?G?/??? ? ?G?? ? ?G/?G? ? ?G?? ? ? ? ? ? ?G?/(? ??G??/? ? ?G??) ? ?G?

Total effective stiffness is calculated by equation (6.59)

? ?? ? ????/???? ? ?G? ? ????/??? ? ?G? ?? ?? ? ?? ? ?G?/?? ? ?G? ?? ?? ?

Vertical displacement is achieved by equation (6.60)

?? ? ?G???/? ? ? ? ?G??

Use a disc with 35 cm in diameter and 1.5 cm in depth. The accuracy of this diameter is checked by equation (6.61)

??G?/??? /? ? ? ? ?G?? ? ?G??

Restoring force is checked by equation (6.62). ??G?? ? ?G? ?

7.7.1.4 Analysis Results

Table 7.15: Transmitted accelerations for different 3-story buildings.

Story	Acceleration (cm/?$^{?}$)		
	LRB	HDRB	FPS
0	71	85	98
1	69	82	96
2	70	83	97
3	73	91	102

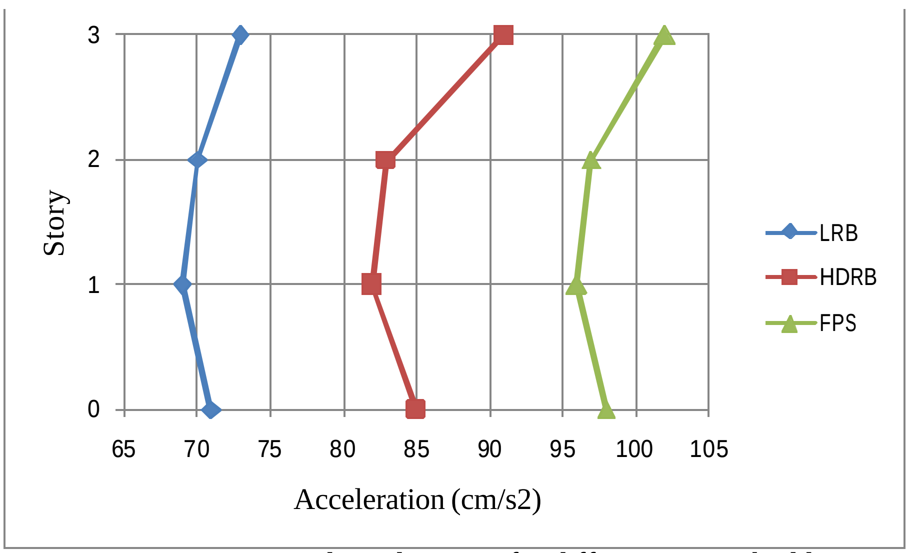


Figure 7.9: Transmitted accelerations for different 3-story buildings.

Table 7.16: Maximum displacements for different 3-story buildings.

Isolator Type	LRB	HDRB	FPS
Maximum Displacement (mm)	157	176	143

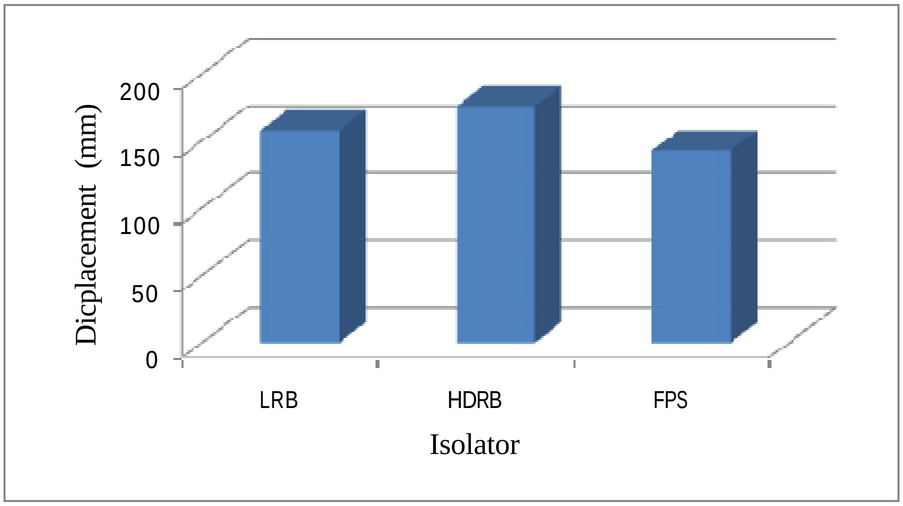


Figure 7.10: Maximum displacements for different 3-story buildings.

Table 7.17: Seismic coefficients for different 3-story buildings.

Isolator Type	LRB	HDRB	FPS
Seismic Coefficient	0.074	0.087	0.101

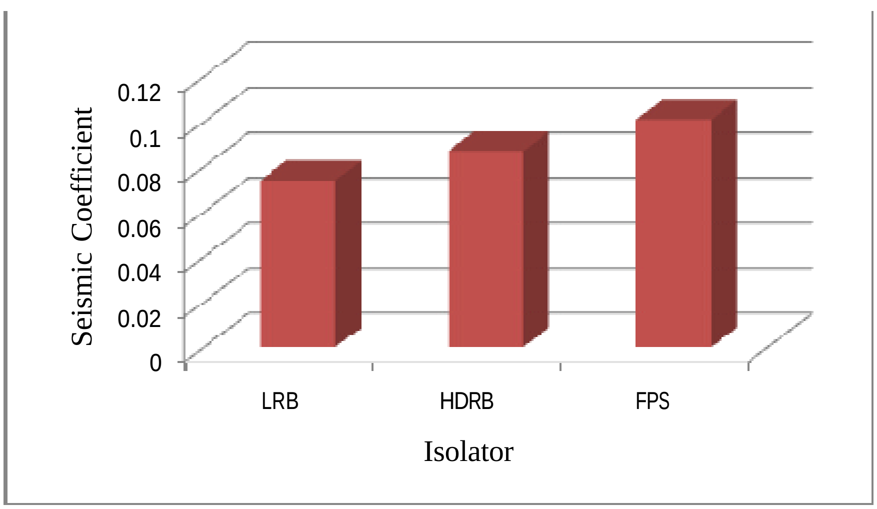


Figure 7.11: Seismic coefficients for different 3-story buildings.

7.7.2 Six Story Building

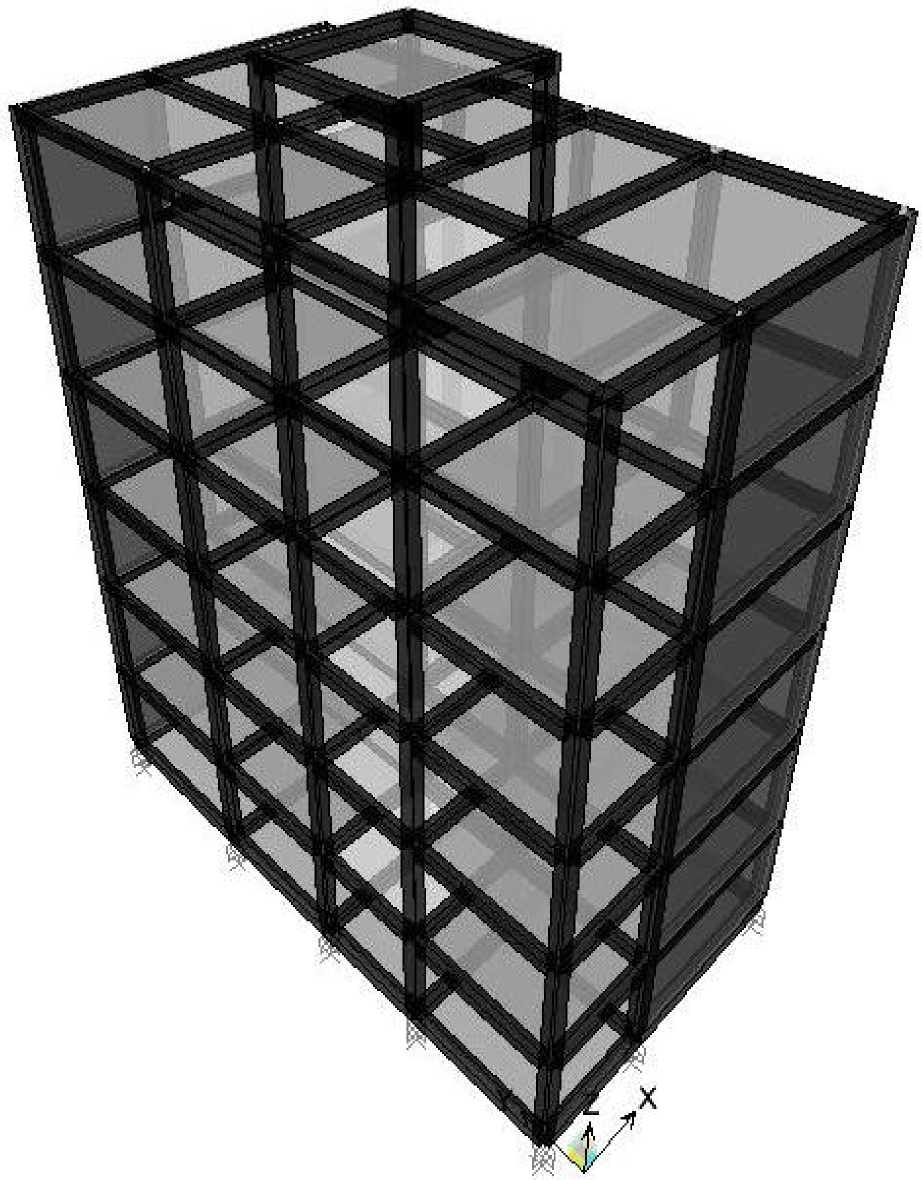


Figure 7.12: 3-D model of 6-story isolated building.

7.7.2.1 Design Result for Lead Rubber Bearing

In this section, design results for Lead Rubber Bearing in the 6-story base isolated building are shown in the different tables.

Table 7.18: Lead Rubber Bearing properties.

Input		**Output**	
B	620	301754.0000	Ag
tsc	10	600.0000	Bb
dpl	90	282600.0000	Ab
N	18	6358.5000	Apl
t1	11	276241.5000	Abn
sy	0.008	198.0000	Tr
G	0.0004	1884.0000	p
?NS	0	13.3295	Si
tsh	3	50.8680	Qd
E	0.00135	0.0004	?50
		0.5968	Kr
		57.8803	Fy
		11.7506	?y
		7249639850.0000	I_m
		249.0000	Hr
		0.1793	Eb
		1634935824.4144	constant T
		148.5929	constant R
		0.0126	constant Q
		600.0000	?
		282743.3388	Ar
		4.9257	Ku
Bearing Properties			

Table 7.19: Gravity load capacity for Lead Rubber Bearing.

Input			Output	
?u	6.5		0.4187	Ec
f	0.33		10762.5676	Kvi
Pdl+ll	1259		0.0106	?c
?NS	0		0.8505	?sc
E?	1.5		0.0000	?sh
k	0.87		0.0000	?sr
			0.8612	?
			2.1450	Allowable ?
			5754.7688	Pcr
			TRUE	? check
			TRUE	P check
			0.0003	Adjusted G
			0.4662	Adjusted K*r
			Vertical Stiffness Calculation	
			597.9204245	Kv
			467.4387198	Vertical Stiffness Kv
Gravity load Capacity				

Table 7.20: Seismic performance for Design Base Earthquake.

Input			Output	
# Isolators	15		153.0000	Assume a displacement
W	12652		122.1970	F
B	1.6		0.7987	Ke
Cv	0.48		11.9801	Total Ke
g	9810		1.2897	M
			2.0605	Te
			28740.2881	Ah
			24.4782	Damping(%)
			0.1456	SA
			153.7608	SD
Seismic Performance for DBE				

Table 7.21: Seismic performance for Maximum Capable Earthquake.

Input			Output	
# Isolators	15		210.0000	Assume a displacement
W	12652		148.7706	F
B	1.5		0.7084	Ke
CVM	0.58		10.6265	Total Ke
g	9810		1.2897	M
			2.1878	Te
			40338.1921	Ah
			20.5599	Damping(%)
			0.1767	SA
			210.4247	SD
Seismic Performance for MCE				

Table 7.22: Load capacity at Design Base Earthquake.

Input			Output	
f DBE	0.75		183.6000	D TD
P dl+ll+E	1971		0.4187	Ec
(D TD)/(DD)	1.2		571.2189	?
			174327.4884	Ar
			6635.7404	Kvi
			0.0270	?c
			2.1596	?sc
			0.9273	?sh
			0.0000	?sr
			3.1139	?
			4.8750	Allowable ?
			3548.1451	Pcr
			TRUE	? check
			TRUE	P check
Load Capacity at DBE				

Table 7.23: Load capacity at Maximum Credible Earthquake.

Input			Output	
f MCE	1		252.0000	DTM
P dl+ll+E	2093		0.4187	Ec
(D TM)/(DM)	1.2		544.5145	?
			136114.3589	Ar
			5181.1654	Kvi
			0.0364	?c
			2.9371	?sc
			1.2727	?sh
			0.0000	?sr
			4.2465	?
			6.5000	Allowable ?
			2770.3606	Pcr
			TRUE	? check
			TRUE	P check
Load Capacity at MCE				

Table 7.24: ETABS output for Design Base Earthquake.

Output	
Spring effective stiffness (KE2)	0.7986
Spring effective stiffness (KE3)	0.7986
Spring effective damping ratio(DE2)	0.1947
Spring effective damping ratio(DE3)	0.1947
Spring stiffness along axial 1 (K1)	467.43871
initial spring stiffness (K2)	4.9257
initial spring stiffness (K2)	4.9257
Yield force	57.8803
Post-Yield stiffness ratio	0.0946
Etabs Output for DBE	

Table 7.25: ETABS output for Maximum Credible Earthquake.

Output	
Spring effective stiffness (KE2)	0.7084
Spring effective stiffness (KE3)	0.7084
Spring effective damping ratio(DE2)	0.1555
Spring effective damping ratio(DE3)	0.1555
Spring stiffness along axial 1 (K1)	467.4387
initial spring stiffness (K2)	4.9257
initial spring stiffness (K2)	4.9257
Yield force	57.8803
Post-Yield stiffness ratio	0.0946
Etabs Output for MCE	

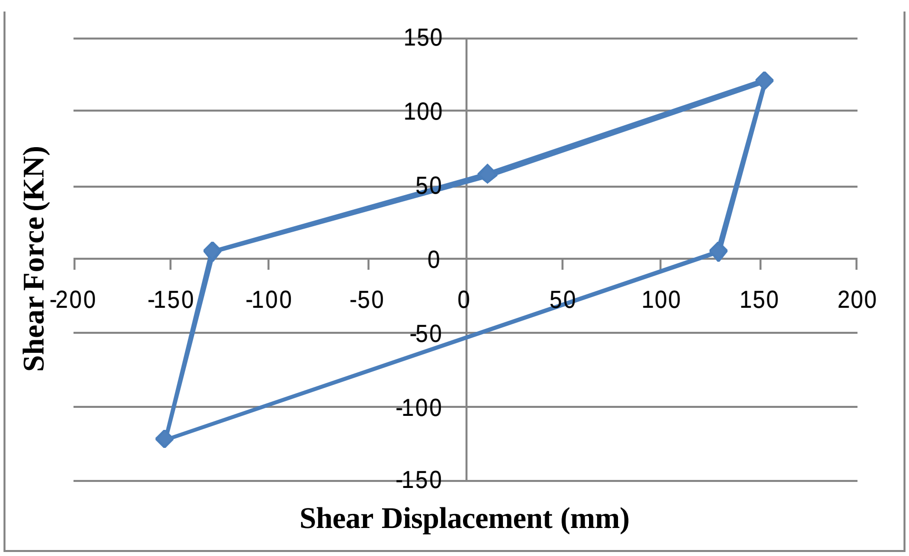


Figure 7.13: Hysteresis curve for LRB in 6-story isolated building.

7.7.2.2 Design Result for High Damping Rubber Bearing

In this section, design results for High Damping Rubber Bearing in the 6-story base isolated building are shown in the Table 7.26.

Table 7.26: Design results for High Damping Rubber Bearing.

Input		Output	
P DL+LL	1259.0000	1.2654	Kem
g	9.8100	176.8834	Dd
T	2.0000	12.7160	Total Ke
B	1.3500	**Design Procedure**	
Cv	0.4800	212.2601	D(TD)
W	12652.0000	117.9223	tr
?max	1.5000	135.0000	Use tr
E	4.4500	**Calculate the Area(A) and Thickness (t)**	
G	1.0600		
k	0.5700	9.0941	min S
€b	500.0000	22.0000	Use S
t steel	2.0000	2459.7820	Ec
Cover	25.0000	0.1606	Min A0 (m2)
P DL+LL+E	2030.0000	0.0405	Min A1 (m2)
b (m)	17.0000	0.1612	Min Asf (m2)
d (m)	9.0000		
D(TD)/D(D)	1.2000	0.1612	Min A (m2)
TM	2.2000	620.0000	Use d
BM	1.3200	0.3018	A (m2)
CVM	0.5800	TRUE	Check A
P DL+LL+E (MCE)	2100.0000	140.0301	ß
		0.1730	A re
		7.0455	Rubber layer thickness t
		8.0000	Use t
		16.8750	N
		216.7500	h
		Shear strain and Vertical Load Capacity	
		0.2239	?dl+ll
		1.6667	€
		TRUE	Check ?

Table 7.26: Design results for High Damping Rubber Bearing (continued).

4172.2728	σc
48367.4074	σ max
TRUE	Check σ
Earthquake Load Capacity at DBE	
0.6297	γsc
1.5723	γeq
0.0059	θ
1.0413	γ sr
3.2433	γ total
3.7500	γ max
TRUE	Check γ
5319.2786	Kvi
315.2165	Kv
Earthquake Load Capacity at MCE	
240.4509	DM
288.5411	DTM
124.5924	β
0.1283	A re
0.8784	γsc
2.4469	γeq
0.0080	θ
1.1922	γ sr
4.5174	γ total
5.0000	γ max
TRUE	Check γ

7.7.2.3 Design Result for Friction Pendulum System

According to the fixed period 2 second and equation (6.56), radius of disk calculated as:

? ? ??? ?/? ? ? ? ??? ?/?.? ? ? ? ? ?

Total horizontal stiffness is computed by equation (6.58)

? ?? ? ?/? ? ?????/???? ? ??.?? ?? ?? ?

For estimating damping and design displacement, a value of damping is assumed, then design displacement is calculated by equation (6.38). At the end, the value of damping that is clarified by equation (6.57) should agree with the assumed damping ± 1%.

Assume that provided damping is 28%

?? ? ?.?/??? ? ?.?? ? ?.?/?.? ? ?.??? ? ? ? ? ? ?.??/(? ??.??/? ? ?.???) ? ?.??

Total effective stiffness is calculated by equation (6.59)

? ?? ? ?????/???? ? ?.?? ? ?????/??? ? ??.?? ?? ?? ? ?? ? ??.?/?? ? ?.? ?? ?? ?

Vertical displacement is achieved by equation (6.60)

?? ? ?.???[?]/? ? ? ? ?.???

Use a disc with 40 cm in diameter and 1.5 cm in depth. The accuracy of this diameter is checked by equation (6.61)

?(?.??/?)?[?]/? ? ? ? ?.?? ? ?.???

Restoring force is checked by equation 6.62. ??.??? ? ?.?? ?

7.7.2.4 Analysis Results

Table 7.27: Transmitted accelerations for different 6-story buildings.

Story	**Acceleration (cm/? ?)**		
	LRB	**HDRB**	**FPS**
0	88	107	108
1	82	101	106
2	75	92	102
3	73	90	102
4	75	94	104
5	84	108	113
6	98	128	129

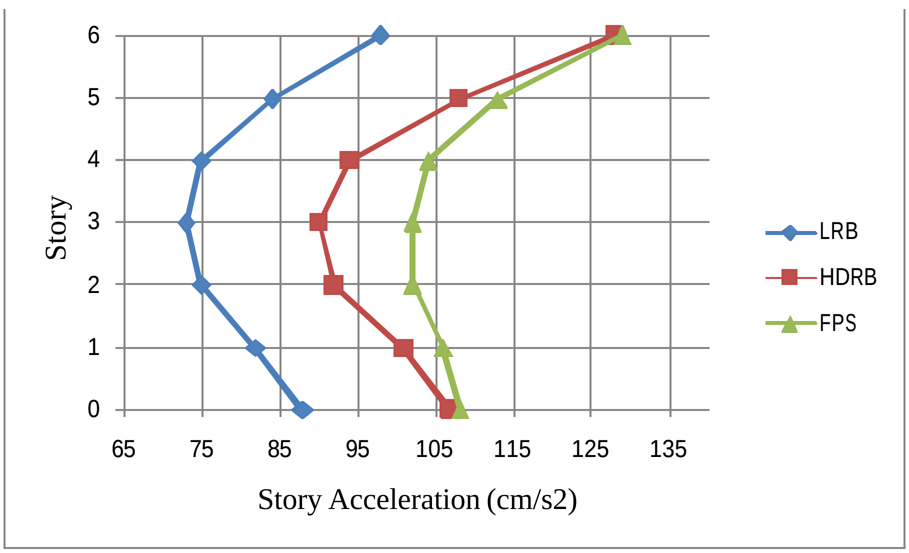


Figure 7.14: Transmitted accelerations for different 6-story buildings.

Table 7.28: Maximum displacements for different 6-story buildings.

Isolator Type	LRB	HDRB	FPS
Maximum Displacement (mm)	153	176	143

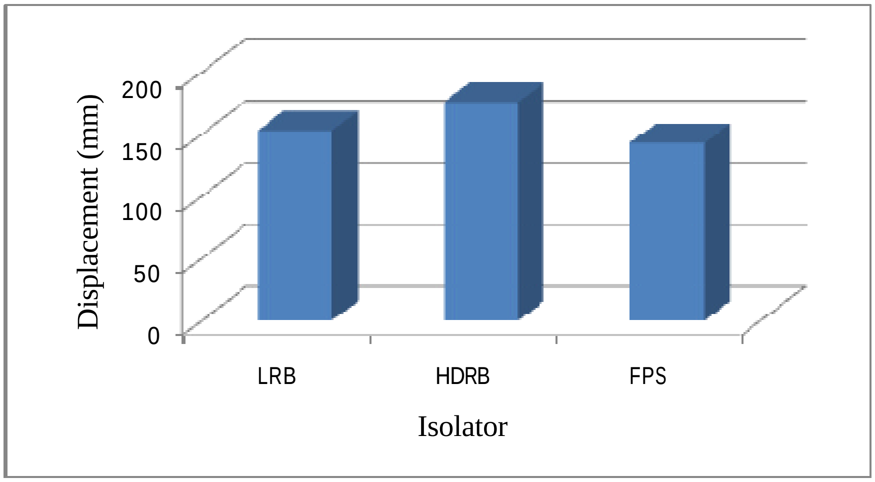


Figure 7.15: Maximum displacements for different 6-story buildings.

Table 7.29: Seismic coefficients for different 6-story buildings.

Isolator Type	LRB	HDRB	FPS
Seismic Coefficient	0.077	0.088	0.11

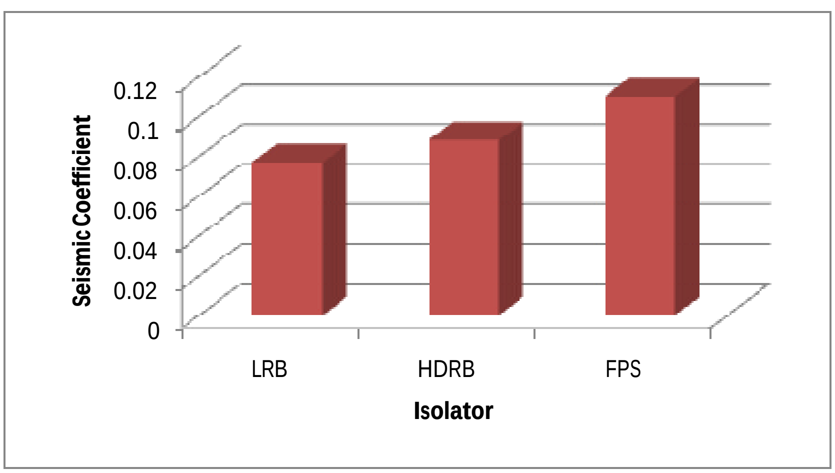


Figure 7.16: Seismic coefficients for different 6-story buildings.

7.7.3 Nine Story Building

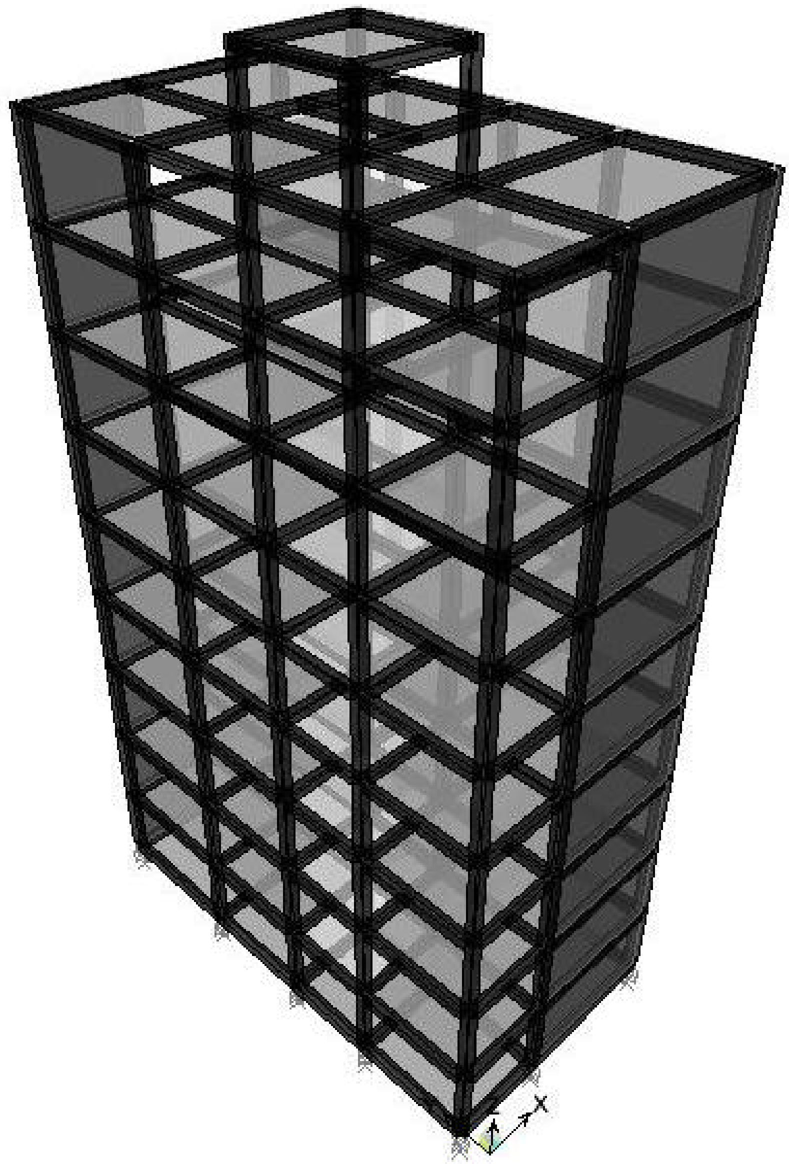


Figure 7.17: 3-D model of 9-story isolated building.

7.7.3.1 Design Result for Lead Rubber Bearing

In this section, design results for Lead Rubber Bearing in the 9-story base isolated building are shown in the different tables.

Table 7.30: Lead Rubber Bearing properties.

Input			Output	
B	660		341946.0000	Ag
tsc	10		640.0000	Bb
dpl	130		321536.0000	Ab
N	18		13266.5000	Apl
t1	11		308269.5000	Abn
sy	0.008		198.0000	Tr
G	0.0004		2009.6000	p
?NS	0		13.9453	Si
tsh	3		106.1320	Qd
E	0.00135		0.0004	?50
			0.6640	Kr
			118.3078	Fy
			18.3371	?y
			9309479850.0000	I_m
			249.0000	Hr
			0.1962	Eb
			2296429263.3843	constant T
			165.3357	constant R
			0.0126	constant Q
			640.0000	?
			321699.0877	Ar
			6.4518	Ku
Bearing Properties				

Table 7.31: Gravity load capacity for Lead Rubber Bearing.

Input		Output	
?u	6.5	0.4582	Ec
f	0.33	13399.1799	Kvi
Pdl+ll	2136	0.0145	?c
?NS	0	1.2126	?sc
E?	1.5	0.0000	?sh
k	0.87	0.0000	?sr
		1.2271	?
		2.1450	Allowable ?
		7232.8956	Pcr
		TRUE	? check
		TRUE	P check
		0.0003	Adjusted G
		0.4679	Adjusted K*r
		Vertical Stiffness Calculation	
		744.3988843 570.2271407	Kv Vertical Stiffness Kv
Gravity load Capacity			

Table 7.32: Seismic performance for Design Base Earthquake.

Input		Output	
# Isolators	15	138.0000	Assume a displacement
W	19436	170.7034	F
B	1.78	1.2370	Ke
Cv	0.48	18.5547	Total Ke
g	9810	1.9812	M
		2.0521	Te
		50800.2578	Ah
		34.3388	Damping(%)
		0.1314	SA
		137.6488	SD
Seismic Performance for DBE			

Table 7.33: Seismic performance for Maximum Capable Earthquake.

Input		Output	
# Isolators	15	187.0000	Assume a displacement
W	19436	193.6309	F
B	1.73	1.0355	Ke
CVM	0.58	15.5319	Total Ke
g	9810	1.9812	M
		2.2429	Te
		71602.1298	Ah
		31.4884	Damping(%)
		0.1495	SA
		187.0458	SD
Seismic Performance for MCE			

Table 7.34: Load capacity at Design Base Earthquake.

Input		Output	
f DBE	0.75	165.600	D TD
P dl+ll+E	3312	0.4582	Ec
(D TD)/(DD)	1.2	618.2044	?
		216909.8906	Ar
		9034.5754	Kvi
		0.0333	?c
		2.7885	?sc
		0.8364	?sh
		0.0000	?sr
		3.6582	?
		4.8750	Allowable ?
		4876.8761	Pcr
		TRUE	? check
		TRUE	P check
Load Capacity at DBE			

Table 7.35: Load capacity at Maximum Credible Earthquake.

Input			Output	
f MCE	1		224.4000	DTM
P dl+ll+E	3557		0.4582	Ec
(D TM)/(DM)	1.2		599.3702	?
			181082.5292	Ar
			7542.3198	Kvi
			0.0492	?c
			3.5873	?sc
			1.1333	?sh
			0.0000	?sr
			4.7635	?
			6.5000	Allowable ?
			4071.3545	Pcr
			TRUE	? check
			TRUE	P check
Load Capacity at MCE				

Table 7.36: ETABS output for Design Base Earthquake.

Output	
Spring effective stiffness (KE2)	1.236980991
Spring effective stiffness (KE3)	1.236980991
Spring effective damping ratio(DE2)	0.293387968
Spring effective damping ratio(DE3)	0.293387968
Spring stiffness along axial 1 (K1)	570.2271407
initial spring stiffness (K2)	6.451832636
initial spring stiffness (K2)	6.451832636
Yield force	118.3078062
Post-Yield stiffness ratio	0.072523352
Etabs Output for DBE	

Table 7.37: ETABS output for Maximum Credible Earthquake.

Output	
Spring effective stiffness (KE2)	1.035459329
Spring effective stiffness (KE3)	1.035459329
Spring effective damping ratio(DE2)	0.264883597
Spring effective damping ratio(DE3)	0.264883597
Spring stiffness along axial 1 (K1)	570.2271407
initial spring stiffness (K2)	6.451832636
initial spring stiffness (K2)	6.451832636
Yield force	118.3078062
Post-Yield stiffness ratio	0.072523352
Etabs Output for MCE	

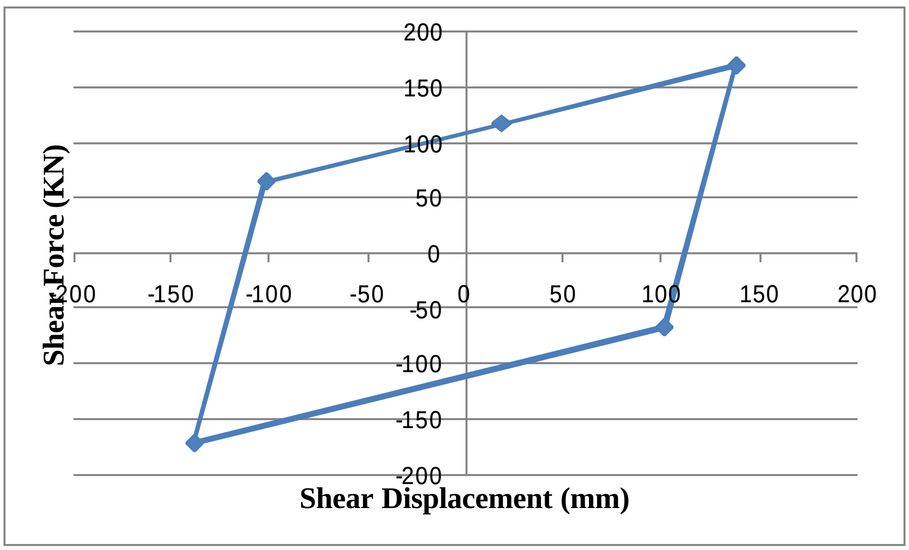


Figure 7.18: Hysteresis Curve for LRB in 9-story isolated building.

7.7.3.2 Design Result for High Damping Rubber Bearing

In this section, design results for High Damping Rubber Bearing in the 9-story base isolated building are shown in the Table 7.38.

Table 7.38: Design results for High Damping Rubber Bearing.

Input		Output	
P DL+LL	2136.0000	2.1468	Kem
g	9.8100	176.8834	Dd
TD	2.0000	19.5343	Total Ke
BD	1.3500	**Design Procedure**	
CVD	0.4800	212.2601	D(TD)
W	19436.0000	117.9223	tr
?max	1.5000	145.0000	Use tr
E	4.4500	**Calculate the Area (A) and Thickness (t)**	
G	1.0600		
k	0.5700	9.0941	min S
€b	500.0000	25.0000	Use S
t steel	2.0000	3175.0750	Ec
Cover	25.0000	0.2724	Min A0 (m2)
P DL+LL+E	3479.0000	0.0605	Min A1 (m2)
b (m)	17.0000	0.2937	Min Asf (m2)
d (m)	9.0000		
D(TD)/D(D)	1.2000	0.2937	Min A (m2)
TM	2.2000	670.0000	Use d
BM	1.3200	0.3524	A (m2)
CVM	0.5800	TRUE	Check A
P DL+LL+E (MCE)	3731.0000	143.1328	ß
		0.2125	A re
		6.7000	Rubber layer thickness t
		8.0000	Use t
		18.1250	N
		229.2500	h
		Shear strain and Vertical Load Capacity	
		0.2864	?dl+ll
		1.6667	€
		TRUE	Check ?
		6061.5262	sc

Table 7.38: Design results for High Damping Rubber Bearing (continued).

51172.4138	σ max
TRUE	Check σ
Earthquake Load Capacity at DBE	
0.7735	γsc
1.4639	γeq
0.0059	θ
1.1322	γ sr
3.3695	γ total
3.7500	γ max
TRUE	Check γ
8433.7930	Kvi
465.3127	Kv
Earthquake Load Capacity at MCE	
240.4509	DM
288.5411	DTM
129.0467	β
0.1654	A re
1.0657	γsc
1.9899	γeq
0.0080	θ
1.5391	γ sr
4.5947	γ total
5.0000	γ max
TRUE	Check γ

7.7.3.3 Design Result for Friction Pendulum System

According to the fixed period 2 second and equation (6.56), radius of disk calculated as:

? ? ??? ?/? ? ? ? ??? ?/?G? ? ? ? ? ?

Total horizontal stiffness is computed by equation (6.58)

? ?? ? ?/? ? ?????/???? ? ??G?? ?? ?? ?

For estimating damping and design displacement, a value for damping is assumed, then design displacement is calculated by equation (6.38). At the end, the value of damping that is clarified by equation (6.57) should agree with the assumed damping ± 1%.

Assume that provided damping is 28%

?? ? ?G?/??? ? ?G?? ? ?G/?G? ? ?G?? ? ? ? ? ? ?G?/(? ??G??/? ? ?G??) ? ?G??

Total effective stiffness is calculated by equation (6.59)

? ?? ? ?????/???? ? ?G?? ? ?????/??? ? ??G ?? ?? ? ?? ? ??G/?? ? ?G? ?? ?? ?

Vertical displacement is achieved by equation (6.60)

?? ? ?G???/? ? ? ? ?G??

Use a disc with 45 cm in diameter and 1.5 cm in depth. The accuracy of this diameter is checked by equation (6.61)

(??G?/??)?/? ? ? ? ?G?? ? ?G??

Restoring force is checked by equation 6.62. ??G?? ? ?G? ? ?

7.7.2.4 Analysis Results

Table 7.39: Transmitted accelerations for different 9-story buildings.

Story	Acceleration (cm/?$^{?}$)		
	LRB	**HDRB**	**FPS**
0	104	126	129
1	99	121	125
2	93	113	121
3	88	105	115
4	84	99	110
5	83	99	108
6	87	106	114
7	95	121	128
8	109	142	150
9	126	167	177

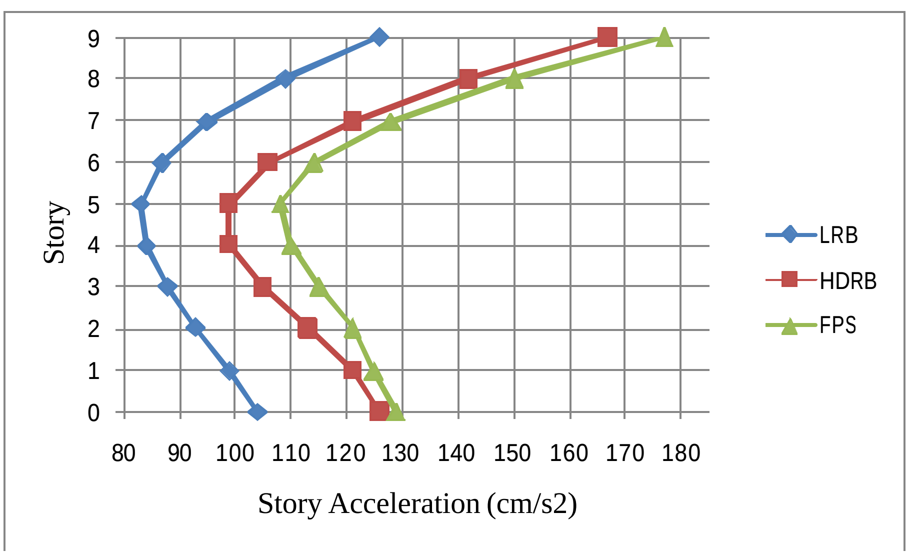


Figure 7.19: Transmitted accelerations for different 9-story buildings.

Table 7.40: Maximum displacements for different 9-story buildings.

Isolator Type	LRB	HDRB	FPS
Maximum Displacement (mm)	200	246	166.4

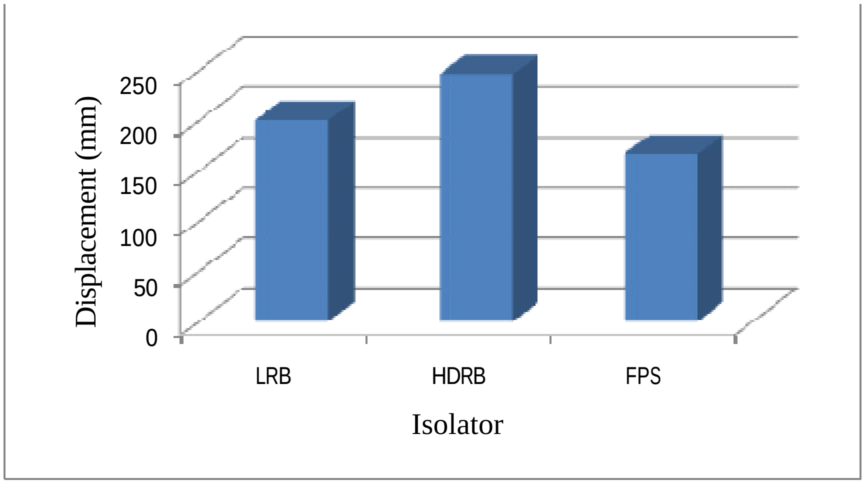


Figure 7.20: Maximum displacements for different 9-story buildings.

Table 7.41: Seismic coefficients for different 9-story buildings.

Isolator Type	LRB	HDRB	FPS
Seismic Coefficient	0.09	0.11	0.12

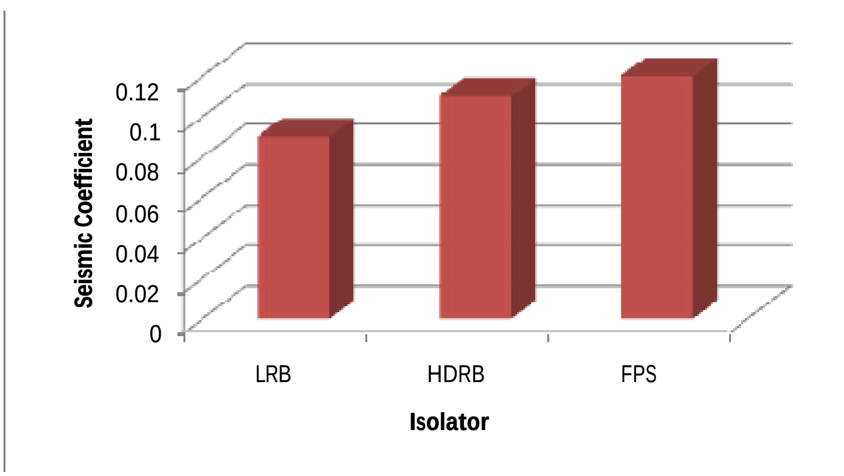


Figure 7.21: Seismic coefficients for different 9-story buildings.

7.7.4 Results and Discussions

At the end of each analysis section, three characteristics of the building, consisting of transmitted acceleration, maximum building displacement and seismic coefficient, are presented.

Figures 7.9, 7.14 and 7.19 show transmitted acceleration for 3, 6 and 9 story buildings in different levels of the stories. LRB isolators produce the minimum level of the transmitted acceleration between the other types. This is followed by HDRBs and FPSs respectively. The most important factor in this reduction corresponds to the high damping and low effective stiffness that is provided by LRB isolators. Comparing the transmitted acceleration graphs for LRBs and HDRBs, the effects of the damping on transmitted acceleration in rubber bearings are illustrated clearly. Generally, rubber bearings represent lower effective stiffness than slider bearings. A rubber bearing provides the specific damping at lower horizontal stiffness than Friction Pendulum Systems. This is proved in the 9story building clearly. In this example, LRB isolation system produce 34.3 % damping in 1.23 kN/mm effective stiffness while Friction Pendulum System provide 29 % damping in 1.84 kN/mm effective stiffness. This is considered to be the most important reason that Friction Pendulum Systems exert higher acceleration to the superstructure among other types. Therefore, an isolation system minimizes the transmitted acceleration to the superstructure which produces minimum effective stiffness and high damping, which these two factors are found in LRB isolators.

At times, due to the limitation in the seismic gap or in the important buildings like museums, an isolation system with minimum displacement is required. Figures 7.10, 7.15 and 7.20 present the maximum structural displacement for the use of different isolation systems. As can be seen from these figures, FPSs minimize displacement in comparison to the other types. It is followed by LRBs and HDRBs respectively. The most important factor in minimizing displacement for FPSs is related to the high friction coefficient, which causes increment damping in this type of isolator. Furthermore, comparing the displacement graphs for LRBs and HDRBs, the effects of the damping on displacement in rubber bearings is shown clearly. It should be noted that with increasing damping in the isolation system, displacement will be decreased. As it is shown in the 9-story buildings, isolators are subjected to more displacement than other buildings. One of the most important factors in increasing displacement corresponds to the torsion effects. According to the Calunga and Cruz (2006) investigation, for minimizing torsion effects that cause an increase in total displacement, the ratio of the isolated building's period to the fixed base one's period should be bigger than 2 which in this building, is approximately 2. Generally, one of the advantages in FPSs is the minimizing displacement that they offer. It should be noted, however, that most of the isolation systems, which present minimum displacement, offer high acceleration and base shear coefficient.

According to the UBC 97 code, transmitted elastic base shear to the super structure is related to the effective stiffness and displacement. Therefore, for reducing transmitted shear force, an isolation system with minimum effective stiffness and displacement is required. Figures 7.11, 7.16 and 7.21 show seismic coefficients for different buildings. It is noted that LRBs represent minimum base shear coefficient. This is pursued by HDRBs and FPSs. An LRB isolator supplies high damping with lower effective stiffness than other types. Comparing to the seismic coefficient graphs for LRBs and HDRBs, the effects of the damping on seismic coefficient in rubber bearings is noticeable. It is shown that, with an increase damping in the isolation system, transmitted base shear coefficient will be decreased.

7.7.5 Comparison of Fixed Base and Seismic Isolated Buildings

In this section, a LRB 3-srory isolated building is designed according to the UBC 97 code and checked with its conventional fixed base to consider their differences in performances and materials. According to the code , the ductility factor 5.5 is used for moment resistance fixed base building while 2 is considered for base isolated one. In order to make the design procedure easier, the same dimensions and thickness of the slabs are used for both buildings (slab sections are assumed which only carry gravity loads). Therefore the weight of the concrete and steel for the slabs in the both buildings are assume to be same and it is not considered in comparison procedure. The dimensions of other sections for two buildings are shown in Table 7.42.

Table 7.42: Dimensions of the structural elements.

Type of Building	Structural Element	Dimension
Fixed Base Building	Beams	35×30
	Columns	40×40 and 35×35
Base Isolated Building	Beams	35×30
	Columns	35×35

The base isolated building includes one more slab than the fixed base one that is considered in comparison procedure. This slab is located at the base of the building and is used to prevent columns of individual movements and better performance in the isolated building. In the UBC 97 code, there is no obligation to use base slab but since this slab was considered in the section 7.7.1 for analyzation, it has been designed.

In this section, the differences of the buildings are shown by comparison of the first mode shape and take-off materials for both buildings.

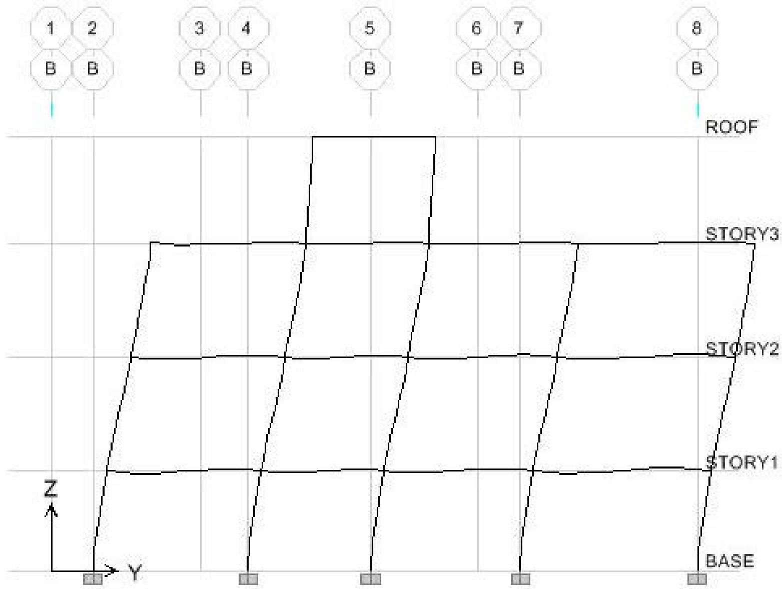


Figure 7.22: First mode shape for middle frame of fixed base building.

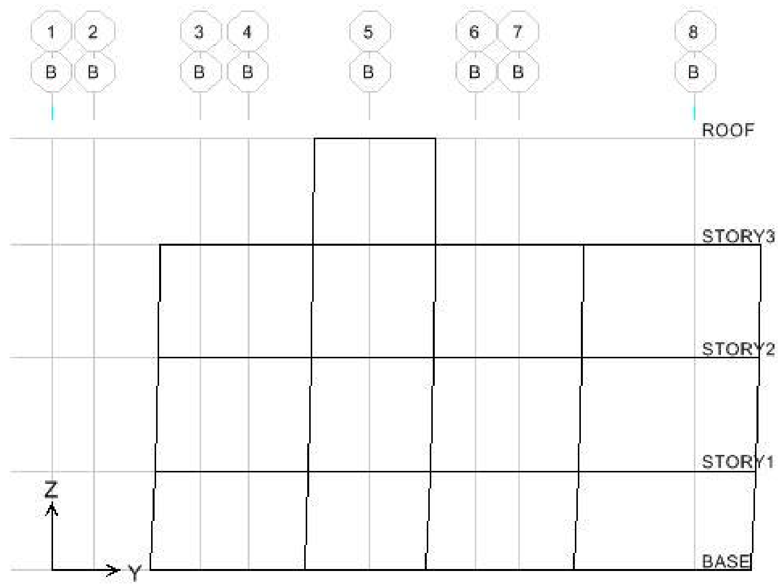


Figure 7.23: First mode shape for middle frame of base isolated building.

The first mode shapes for fixed base and isolated base buildings are shown in the Figure 7.22 and 7.23 respectively (same scale factor is used for both figures). These two Figures show the effectiveness of the isolation system in reducing internal story drifts. When the base isolated building is subjected to an earthquake, the super structure shakes with moving isolators similar to a rigid block.

Table 7.43 shows the comparison of the materials that are used in both buildings. In this table one additional slab is shown for base isolated building.

Table 7.43: Materials take-off by elements.

	Fixed Base Moment Resistance Building		Base Isolated Building	
Structural Elements	**Concrete weight (kg)**	**steel weight (kg)**	**Concrete weight (kg)**	**steel weight (kg)**
Beams	72709.2	3604.02	72709.2	3214.35
Columns	52200	3060	44100	2308
Additional Slab	-------	-------	47793	2115.7
Total	124909.2	6664.02	164602.2	7638.05

As is shown in this table, the total weights of the concrete and steel in the base isolated building (with additional slab) are only a few more than fixed base one. It should be noted, however, that base isolated building is designed for elastic forces (R=2), which reduce damages in structural and non-structural elements to a great degree.

In order to consider the additional cost of the base isolated building as a percentage of the total cost, the cost of the isolators, concrete and steel should be provided. However, these costs change from one country to another. Therefore, this evaluation process has been performed according to the assumption that this building is considered to be in two countries, North Cyprus and Iran. The unit cost of different materials and the additional cost of the base isolated building for both countries are shown in Table 7.44 and 7.45 respectively.

Table 7.44: Cost of materials per unit.

Item	North Cyprus	Iran
Concrete (dollar/ton)	31.8	23.1
Steel (dollar/ kg)	1.47	0.62
Isolator (unit)	700	700
Total Cost (dollar/m^2)	538.06	416.23
Total Cost of Building (dollar)	246969.54	191049.57

Table 7.45: Additional cost of materials as percentage of total cost.

Additional Cost	North Cyprus	Iran
Concrete (%)	0.51	0.48
Steel (%)	0.57	0.31
Isolator (%)	4.25	5.5
Total (%)	5.33	6.29

As illustrated of Table 7.44, the additional cost of the base isolated building for North Cyprus and Iran are 5.33 % and 6.29% of the total cost respectively. In this evaluation, the cost of damage to the structures after an earthquake, which is considered to be the most important factor in the cost for base isolated buildings, is not considered. Generally, low rise buildings are considered as the most uneconomical buildings for isolating. Nevertheless, for the worst case (low rise building with additional slab) the cost of the structure increases around 5.81 % of the total cost without evaluating damage costs. When increasing the height of the building, since the total cost of structure and number of structural elements increase, the additional cost of the base isolated buildings will be decreased.

CHAPTER 8

CONCLUSIONS AND RECOMENDATIONS

8.1 Conclusions

The primary aim of this research was to assess the general information about seismic isolated buildings. The biggest problem of isolation systems is related to isolate light weight structures. In these structures, due to their low-weight and the high stiffness of the isolators, displacement is reduced, and higher acceleration is transmitted to the structure. If the stiffness of the isolator is reduced by increasing the thickness of the rubber layers and reducing the diameter of the lead cores, the isolator will fail due to the P-Delta effects. Consequently, different types of foundation isolation systems, particularly the Robals and RoGliders are the most useful isolation systems for isolating low-weight structures. Elastomeric bearings and sliders are the most popular and practical ones for isolating medium and high-weight structures.

In the seismic isolated structures, it is more economical to locate isolators as near as possible to the foundation in order to isolate more components of the structure. At this level, mechanical works should be made of flexible materials to endure design displacement.

Seismic Isolators are the best alternative for seismic retrofitting of the structures. However, in the other methods, such as section enlargement, additional shear walls, FRPs and etc, building should be vacated during retrofitting process while in this method, the structure can be strengthened while it is operational.

Designing various isolators and the selection of the appropriate isolation system based on different structural conditions continues to present an enormous complexity in the seismic isolated structures.

In the research, 3 different buildings (3, 6 and 9 story), isolated by 3 various isolators, were analyzed in order to evaluate the optimum one according to the seismic demands. Consideration was given to transmitted acceleration, maximum structural displacement and the seismic coefficient for each building shown in the different graphs. Furthermore, the 3 story optimum isolated building was compared with its conventional fixed base one in performance and material. From the trends of the results of the present study, the following conclusions are drawn:

1. LRBs produce the minimum level of the transmitted acceleration among the other types. This is followed by HDRBs and FPSs respectively.
2. In the rubber bearings, transmitted acceleration to the superstructure is affected by damping of isolation system. On the other hand, transmitted acceleration to the superstructure is reduced in the case of an increase in damping of the isolation system.
3. An isolation system minimizes transmitted acceleration to the superstructure while producing minimum effective stiffness and high damping, two factors that are found in LRBs.
4. FPSs minimize displacement in comparison to other types. It is followed by LRBs and HDRBs respectively.
5. In the rubber bearings, damping affects isolator displacement. Displacement decreases with increased damping of the isolation system.

6. Most of the isolation systems which present minimum displacement, offer high acceleration and base shear coefficient.
7. LRBs produce minimum base shear coefficient, when compared to the other types. This is followed by HDRBs and FPSs.
8. In the rubber bearings, transmitted elastic shear forces are influenced by damping of isolation system. It is noted that with an increase in damping of the isolation system, transmitted base shear coefficient will be decreased.
9. Comparing of fixed base and base isolated structures, it is concluded that internal story drift is minimized in base isolated structures but they, unfortunately, increase first floor drift.
10. In low-rise structures, the cost of base isolated building as compared to the conventional fixed base one, increases 5.8 % of total cost. It should be noted, however, that base isolated structure is designed for elastic forces, which reduces the damage cost significantly.

8.2 Recommendations for Future Studies

The number of base isolation methods is increased year by year. Unfortunately, some of these methods are debatable and unpractical. Therefore, in the near future, so many different acceptable methods will be suggested.

The behavior of the isolators (especially rubber bearings) under tension and torsion forces needs more investigation. Some of the acceptable solutions such as installing tension bolts in the rubber bearings, applying bracing systems to the surrounded frames, using dampers and etc are suggested by engineers, which explained in Chapter 1 section 1.4.

Most of the seismic isolated structures that are located in high seismic zones (near fault) need more investigations, when subjected to the far faults ground motions.

Related to the thesis, the further studies could be done for different types of buildings by increasing the number of stories. It is expected that the additional cost of the base isolated structure, comparing to its conventional fixed base one will be decreased, because of the increment in the total cost of structure and numbers of structural elements.

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APPENDIXES

Appendix A: Tables of Uniform Building Code 1997

Table 16.A : Uniform Load.

use of occupancy		uniform load (psi)
category	**description**	**0.0479 kN/m^2**
Access floor systems	office use	50
	computer use	100
Armories		150
Assembly area and auditorium and balconies	Fixed seating area	50
	Moveable seating and other	100
	Stage area and enclose platforms	125
Cornices and marquees		60
Exit facilities		100
Garages	General storage and/or repair	100
	Private or pleasure-type motor vehicle storage	50
Hospitals	Wards and rooms	40
Libraries	Reading rooms	60
	Stack rooms	125
Manufacturing	Light	75
	Heavy	125
Offices		50
Printing plants	Press rooms	150
	Composing and linotype rooms	100
Residential	Basic floor area	40
	Exterior balconies	60
	Decks	40
	Storage	40
Restrooms		
Reviewing stands, grandstands, bleachers, and folding and telescoping seating		100
Roof decks	Same as area served or for the type of occupancy accommodated	
Schools	Classrooms	40
Sidewalks and driveways	Public access	250
Storage	Light	125
	Heavy	250
Stores		100

Table 16.I: Seismic zone factor Z.

Zone	**1**	**2A**	**2B**	**3**	**4**
Z	0.075	0.15	0.2	0.3	0.4

Table 16.J: Soil profile types.

Soil Profile Type	**Soil Profile Name/Generic Description**
S_A	Hark rock
S_B	Rock
S_C	Very dense soil and soft rock
S_D	Stiff soil profile
S_E	Soft soil profile
S_F	Soil require site specific evaluation

Table 16.Q : Seismic Coefficient C_a.

Soil Profile Type	**Seismic Zone Factor (Z)**				
	Z=0.075	Z=0.15	Z=0.2	Z=0.3	Z=0.4
S_A	0.06	0.12	0.16	0.24	0.32 N_a
S_B	0.08	0.15	0.20	0.30	0.40 N_a
S_C	0.09	0.18	0.24	0.33	0.40 N_a
S_D	0.12	0.22	0.28	0.36	0.44 N_a
S_E	0.19	0.30	0.34	0.36	0.36 N_a
S_F	Site-specific geotechnical investigation shall be performed to determine seismic coefficients for soil profile type S_F.				

Table 16.R : Seismic Coefficient C_V.

Soil Profile Type	**Seismic Zone Factor (Z)**				
	Z=0.075	Z=0.15	Z=0.2	Z=0.3	Z=0.4
S_A	0.06	0.12	0.16	0.24	0.32 N_v
S_B	0.08	0.15	0.20	0.30	0.40 N_v
S_C	0.13	0.25	0.32	0.45	0.56 N_v
S_D	0.18	0.32	0.40	0.54	0.64 N_v
S_E	0.26	0.50	0.64	0.84	0.96 N_v
S_F	Site-specific geotechnical investigation shall be performed to determine seismic coefficients for soil profile type S_F.				

Table 16.N : Structural Systems.

Basic Structural System	Lateral-Force Resisting System Description	R	Height Limitation × 304.8 for mm
Bearing Wall System	Light-framed walls with shear panels		
	a. Wood structure panel walls	5.5	65
	b. All other light framed walls	4.5	65
	Shear walls		
	a. Concrete	4.5	160
	b. Masonry	4.5	160
	Light steel frame bearing walls with tension-only bracing	2.8	65
	Braced frames where bracing carries gravity loads		
	a. Steel	4.4	160
	b. Concrete	2.8	----
	c. Heavy timber	2.8	65
Building frame System	Steel eccentrically braced frame	7.0	240
	Light-framed walls with shear panels		
	a. Wood structures panel walls for structures three stories or less	6.5	65
	b. All other light-framed walls	5.0	65
	Shear walls		
	a. Concrete	5.5	240
	b. Masonry	5.5	160
	Ordinary braced frames		
	a. Steel	5.6	160
	b. Concrete	5.6	-----
	c. Heavy timber	5.6	65
	Special concentrically braced frames		
	a. Steel	6.4	240
Moment-resistance systems	Special moment-resisting frame		
	a. Steel	8.5	N.L.
	b. Concrete	8.5	N.L.
	Masonry moment-resisting wall frame	6.5	160
	Concrete intermediate moment-resisting frame	5.5	-----
	Ordinary moment-resisting frame		
	a. Steel	4.5	160
	b. Concrete	3.5	-----
	Special truss moment frames of steel	6.5	240

Dual systems	Shear walls		
	a. Concrete with SMRF	8.5	N.L.
	b. Concrete with steel OMRF	4.2	160
	c. Concrete with IMRF	6.5	160
	d. Masonry with SMRF	5.5	160
	e. Masonry with steel OMRF	4.2	160
	f. Masonry with concrete IMRF	4.2	-----
	g. Masonry with masonry MMRWF	6.0	160
	Steel EBF		
	a. With steel SMRF	8.5	N.L.
	b. With steel OMRF	4.2	160
	Ordinary braced frames		
	a. Steel with steel SMRF	6.5	N.L.
	b. Steel with steel OMRF	4.2	160
	c. Concrete with concrete SMRF	6.5	-----
	d. Concrete with concrete IMRF	4.2	-----
	Specially concentrically braced frames		
	a. Steel with steel SMRF	7.5	N.L.
	b. Steel with steel OMRF	4.2	160
Cantilever column building systems	Cantilevered column elements	2.2	35
Shear wall-frame interaction systems	Concrete	5.5	160

Table 16.S : Near source factor N_a.

Seismic Source Type	**Closest Distance to Known Seismic Source**		
	= 2km	5 km	=10 km
A	1.5	1.2	1.0
B	1.3	1.0	1.0
C	1.0	1.0	1.0

Table 16.T : Near source factor N_V.

Seismic Source Type	**Closest Distance to Known Seismic Source**			
	= 2km	5 km	10 km	=15 km
A	2.0	1.6	1.2	1.0
B	1.6	1.2	1.0	1.0
C	1.0	1.0	1.0	1.0

Table 16.U : Seismic source type.

Seismic Source Type	Seismic Source Description
A	Faults that are capable of producing large magnitude events and that have a high rate of seismic activity
B	All faults other than Types A and C
C	Faults that are not capable of producing large magnitude earthquakes and that have a relatively low rate of seismic activity

Table A.16.C : Damping coefficients B_D and B_M.

Effective Damping	B_D or B_M Factor
= 2	0.8
5	1.0
10	1.2
20	1.5
30	1.7
40	1.9
= 50	2.0

Table A.16.D : Maximum Capable Earthquake Response Coefficient M_M.

Design Base Earthquake Shaking Intensity, ZN_V	Maximum Capable Earthquake Response Coefficient M_M
0.075	2.67
0.15	2
0.2	1.75
0.3	1.50
0.4	1.25
= 0.5	1.20

Table A.16.E : Structural system above the isolation interface.

Basic Structural System	Lateral-Force Resisting System Description	R_i	Height Limitation × 304.8 for mm
Bearing Wall System	Light-framed walls with shear panels		
	a. Wood structure panel walls	2.0	65
	b. All other light framed walls	2.0	65
	Shear walls		
	a. Concrete	2.0	160
	b. Masonry	2.0	160
	Light steel frame bearing walls with tension-only bracing	1.6	65
	Braced frames where bracing carries gravity loads		
	a. Steel	1.6	160
	b. Concrete	1.6	----
	c. Heavy timber	1.6	65
Building frame System	Steel eccentrically braced frame	2.0	240
	Light-framed walls with shear panels		
	a. Wood structures panel walls for structures three stories or less	2.0	65
	b. All other light-framed walls	2.0	65
	Shear walls		
	a. Concrete	2.0	240
	b. Masonry	2.0	160
	Ordinary braced frames		
	a. Steel	1.6	160
	b. Concrete	1.6	-----
	c. Heavy timber	1.6	65
	Special concentrically braced frames		
	a. Steel	2.0	240
Moment-resistance systems	Special moment-resisting frame		
	a. Steel	2.0	N.L.
	b. Concrete	2.0	N.L.
	Masonry moment-resisting wall frame	2.0	160
	Concrete intermediate moment-resisting frame	2.0	-----
	Ordinary moment-resisting frame		
	a. Steel	2.0	160
	b. Concrete	2.0	-----
	Special truss moment frames of steel	2.0	240

Dual systems	Shear walls		
	a. Concrete with SMRF	2.0	N.L.
	b. Concrete with steel OMRF	2.0	160
	c. Concrete with IMRF	2.0	160
	d. Masonry with SMRF	2.0	160
	e. Masonry with steel OMRF	2.0	160
	f. Masonry with concrete IMRF	2.0	-----
	g. Masonry with masonry MMRWF	2.0	160
	Steel EBF		
	a. With steel SMRF	2.0	N.L.
	b. With steel OMRF	2.0	160
	Ordinary braced frames		
	a. Steel with steel SMRF	2.0	N.L.
	b. Steel with steel OMRF	2.0	160
	c. Concrete with concrete SMRF	2.0	-----
	d. Concrete with concrete IMRF	2.0	-----
	Specially concentrically braced frames		
	a. Steel with steel SMRF	2.0	N.L.
	b. Steel with steel OMRF	2.0	160
Cantilever column building systems	Cantilevered column elements	1.4	35
Shear wall-frame interaction systems	Concrete	2.0	-----

Table A.16.F: Seismic Coefficient C_{AM}.

Soil Profile Type	Seismic Zone Factor (Z)				
	M_MZN_a= 0.075	M_MZN_a= 0.15	M_MZN_a= 0.2	M_MZN_a =0.3	M_MZN_a=0.4
S_A	0.06	0.12	0.16	0.24	0.8 M_MZN_a
S_B	0.08	0.15	0.20	0.30	1.0 M_MZN_a
S_C	0.09	0.18	0.24	0.33	1.0 M_MZN_a
S_D	0.12	0.22	0.28	0.36	1.1 M_MZN_a
S_E	0.19	0.30	0.34	0.36	0.9 M_MZN_a
S_F	Site-specific geotechnical investigation shall be performed to determine seismic coefficients for soil profile type S_F.				

Table A.16.G: Seismic Coefficient C_{VM}.

Soil Profile Type	Seismic Zone Factor (Z)				
	M_MZN_v= 0.075	M_MZN_v= 0.15	M_MZN_v= 0.2	M_MZN_v =0.3	M_MZN_v=0.4
S_A	0.06	0.12	0.16	0.24	0.8 M_MZN_v
S_B	0.08	0.15	0.20	0.30	1.0 M_MZN_v
S_C	0.13	0.25	0.32	0.45	1.4 M_MZN_v
S_D	0.18	0.32	0.40	0.54	1.6 M_MZN_v
S_E	0.26	0.50	0.64	0.84	2.4 M_MZN_v
S_F	Site-specific geotechnical investigation shall be performed to determine seismic coefficients for soil profile type S_F.				