HASAN KALYONCU UNIVERSITY
GRADUATE SCHOOL OF
NATURAL & APPLIED SCIENCES

SEISMIC BASE ISOLATION IN REINFORCE CONCRETE STRUCTURE

M. Sc. THESIS
IN
CIVIL ENGINEERING

BY

Mustafa Wassef ABDULMAJEED

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Seismic Base Isolation In Reinforce Concrete Structure

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In

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Hasan Kalyoncu University

Supervisor

Assist. Prof. Dr. Dia Eddin NASSANI

By

Mustafa Wassef ABDULMAJEED

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Department of Civil Engineering

Name of the thesis: Seismic Base Isolation In Reinforce Concrete Structure

Name of the student: Mustafa Wassef ABDULMAJEED

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Approval of the Graduate School of Natural and Applied Sciences

Prof. Dr. Mehmet KARPUZCU

Director

I certify that this thesis satisfies all the requirements as a thesis for the degree of Master of Science.

Assist. Prof. Dr. Kasım MERMERDAŞ

Head of Department

This is to certify that we have read this thesis and that in our consensus/majority opinion it is fully adequate, in scope and quality, as a thesis for the degree of Master of Science.

Dia Eddin

Assist. Prof. Dr. Dia Eddin NASSANI

Supervisor

Examining Committee Members	Signature
Prof. Dr. Mustafa Y. KILINÇ	....
Assist. Prof. Dr. Dia Eddin NASSANI	...Dia Eddin...
Assist. Prof. Dr. Kasım MERMERDAŞ	K. Mermerdaş

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Mustafa Wassef ABDULMAJEED

ABSTRACT

Seismic Base Isolation In Reinforce Concrete Structure

M.Sc. Thesis In Civil Engineering

Supervisor: Assist. Prof. Dr. Dia Eddin NASSANI

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Base isolation is one of the most widely accepted seismic protection systems used in building in earthquake prone areas. The base isolation system separates the structure from its foundation and primarily moves it relative to that of the upper structure. The aim of this study is to reduce the base shear and story drifts due to earthquake ground excitation, applied to the superstructure of the building by installing base isolation devices at the foundation level and then to compare the different performances between the fixed base condition and base-isolated condition .The static equivalent lateral force of analysis, response spectrum and time history analysis and used on both of fixed base and base isolated buildings

The key references that are used in this study are the related chapters of IBC2006 code for seismic isolated structures. In this work, this code is used for the design examples of elastomeric bearings. Furthermore, the internal forces develop in the superstructure during a ground motion is determined; and the different approaches defined by the codes towards the 'scaling factor' concept is compared in this perspective.

Keywords: Seismic Isolation, Base Isolation, Earthquake Resistant Design, Seismic Protective Systems

ÖZET

Betonarme Yapılarda Sismik Zemin Yalıtımı

İnşaat Mühendisliği Bölümü Yüksek Lisans Tezi

Danışman: Assist. prof. Dr. Dia Eddin NASSANI

Mayıs 2014, 116 Sayfa

Zemin yalıtımı, deprem riski bulunan alanlardaki yapılar için en çok kabul gören sismik koruma sistemidir. Zemin yalıtım sistemi, yapıyı temelinden ayırır ve yapının öncelikle üst kısma göre hareket etmesini sağlar. Bu çalışmanın amacı, zemin kesmesi kuvvetini ve deprem yer eksitasyonundan kaynaklanan kat ötelenmesini azaltmak için binanın üstyapısına yerleştirilen temel seviyesindeki zemin yalıtımı aletiyle uygulama yapmaktır. Ve daha sonra da sabit zemin durumuyla yalıtılmış zemin durumları arasındaki farklı performanslar karşılaştırılmıştır. Statik eşdeğer yanal kuvvet analizi, tepki spektrumu ve geçmiş zaman analizi sabit zeminli ve yalıtılmış zeminli yapılarda kullanılmıştır.

Bu çalışmada IBC2006 sismik yalıtımlı yapılar kodunun bölümleriyle alakalı olan kısmındaki referanslar kullanılmıştır. Bu çalışmada belirtilen kodlar elastomerik rulman örneklerinin tasarımı için kullanılmıştır. Ayrıca, zemin hareketi boyunca üstyapıda oluşan içsel kuvvetler belirlenmiştir ve kodlar tarafından 'ölçüm faktörleri' konseptine karşı belirlenen farklı yaklaşımlar bu açıdan karşılaştırılmıştır.

Anahtar Kelimeler: Sismik Yalıtım, Zemin Yalıtımı, Depreme Dayanıklı Dizaynlar, Sismik Koruyucu Sistemler.

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A	Cross-sectional area
A_0	Seismic zone coefficient
B_D	Numerical coefficient for effective damping at design displacement
b	Long side length of building in plan
D	Specified design displacement
D_y	Yield displacement
D_D	Displacement corresponding to the ‘design earthquake’
D_{TD}	Total design displacement
D_M	Displacement corresponding to the ‘max. capable earthquake’
d	Short side length of building in plan
e	Eccentricity
E_{loop}	Energy dissipation per cycle of loading
E_c	Instantaneous compression modulus
F^+	Positive force at test displacement
F^-	Negative force at test displacement
$(F_v)_{max}$	Maximum vertical load for one isolator
G	Shear modulus of the elastomer
g	Acceleration of gravity
h	Total height of bearing
h_i	Height above the base to level i
h_x	Height above the base to level x
I	Importance factor of structure
K_H	Horizontal stiffness of bearing
K_V	Vertical stiffness of bearing
K	Bulk modulus
K_1	Elastic stiffness
K_2	Post-yield stiffness

k_{eff}	Effective stiffness
k_D	Lateral stiffness corresponding to the ‘design earthquake’
k_M	Lateral stiffness corresponding to the ‘max. capable earthquake’
M_W	Magnitude
N	Number of storey
Q	Characteristic strength
P_{cr}	Critical buckling load
P_E	Euler buckling load
P_S	Shear stiffness per unit length
R	Seismic load reduction factor
S	Shape factor
S_{D1}	Design 5% damped spectral acceleration at 1 sec. Period
T	Fixed-based period
T_M	Isolated period at maximum displacement
T_{eff}	Effective period
T_D	Isolated period at design displacement
T_V	Vertical fundamental period of vibration
t_r	Total thickness of rubber
t_o	Thickness of single layer of rubber
W_D	Area of hysteresis loop
W_x	Portion of W that is located at or assigned to level x
W_i	Portion of W that is located at or assigned to level i
W	Total weight of superstructure
V_S	Base shear force
β	Damping ratio
β_{eff}	Effective viscous damping ratio
β_s	Damping ratio of superstructure
β_i	Damping ratio of isolator
Δ^+	Positive peak test displacements
Δ^-	Negative peak test displacements
γ_{max}	Maximum shear strain
Φ	Diameter

CHAPTER 1

INTRODUCTION

Recent earthquakes, particularly the 1989 Lorna Prieta and 1994 Northridge earthquakes in California, and the 1995 Kobe earthquake in Japan, have caused significant loss of life and severe damage to the property. Many a seismic construction designs and technologies have been developed over the years in attempts to mitigate the effects of earthquakes on buildings and their vulnerable contents. Attenuating the effects of severe ground motions on the buildings and their contents is always one of the most popular topics in the area of civil and structural engineering and attracts the attention of many researchers and engineers around the world.

The technique of base isolation has been developed in an attempt to mitigate the effects on buildings and their contents during earthquake attacks and has been proven to be one of the more effective methods for a wide range of seismic design problems on buildings in the past two decades. Seismic isolation consists essentially of the installation of mechanisms which decouple the structures and their contents from potentially damaging earthquake-induced ground motions. This decoupling is achieved by increasing the flexibility of the systems, together with providing appropriate damping. Careful studies have been made of structures for which seismic isolation may find widespread application. This has been found to include common forms of new and existing multi storey building structures.

In seismic isolation, the fundamental aim is to reduce substantially the transmission of the earthquake forces and energy into the structure. This is achieved by mounting the structure on an isolation system with considerable horizontal flexibility so that during an earthquake, when the ground vibrates strongly under the structure, only moderate motions are induced within the structure itself.

As the isolator flexibility increases, movements of the structure relative to the ground may become a problem under other vibration loads applied above the level of the isolator, particularly wind loads. (Skinner et al., 1993) indicated that a base isolator with hysteretic force-displacement characteristics can provide the desired properties of isolator flexibility, high damping and force limitation under horizontal earthquake loads, together with high stiffness under smaller horizontal loads to limit wind-induced motions.

Kelly (1990) gave a brief introduction to the response mechanisms of base isolated buildings through a two degrees of freedom linear system. The effectiveness of the isolation system to mitigate the seismic response is through its ability to shift the fundamental frequency of the system out of the range of frequencies where the earthquake is strongest. Also, (Skinner et al., 1993) demonstrated that the most important feature of seismic isolation is that its increased flexibility increases the natural period of the structure. Because the period is increased beyond that of the earthquake, resonance is avoided and the seismic acceleration response is reduced.

The successful base isolation of a building depends on the installation of mechanisms which decouple the structure from potentially damaging earthquake-induced ground motions. Therefore, it is very important to have an adequate understanding of the influence of each parameter in the isolation system and the superstructure on the seismic performance of the base isolated buildings.

The primary function of an isolation device is to support the superstructure while providing a high degree of horizontal flexibility. This gives the overall structure a long effective period and hence lower earthquake generated accelerations and inertia forces. Many kinds of isolation systems have been developed to achieve this function, such as laminated elastomeric rubber bearings, lead-rubber bearings, yielding steel devices, friction devices (PTFE sliding bearings) and lead extrusion devices…. etc.

Andriono (1990) indicated that base isolated structures have the ability to significantly reduce the ductility demands in the superstructure compared with those of unisolated structures.

This makes possible simplification of the structural detailing and other seismic design considerations required by the more conventional approaches. Therefore, a wider choice of architectural forms and structural materials is available to the designer.

Besides the technical feasibility, another key issue that must be addressed early in the design phase of a base isolated building is the economic feasibility. In terms of economic cost, many principal factors should be evaluated. Kelly (1990) indicated that these are construction costs, earthquake insurance premiums, physical damage for repair and disruption costs, loss of market share and potential liability. (Skinner et al., 1993) showed that the current practice for seismic isolation techniques may often reduce the cost of providing a given level of earthquake resistance to buildings.

In the past two decades, base isolation has become an increasingly accepted technique for providing earthquake protection to buildings and their contents. On the other hand, base isolation has often been considered as a technique for problem structures or for equipment which requires a special seismic design approach.

This may arise because of their function (sensitive or high risk industrial or commercial facilities such as computer systems or nuclear power plants); their special importance after an earthquake (hospitals, disaster control centers such as police stations); poor ground conditions (proximity to a major fault); or other special problems (increasing the seismic resistance of existing structures) as described by(Skinner et al., 1993).

Therefore, seismic isolation does indeed have particular advantages over other approaches in these special circumstances, usually being able to provide much better protection under extreme earthquake motions. It is believed that seismic isolation may be used to provide effective solutions for a wide range of seismic design problems.

1.1. Principles of Seismic Isolation

Seismic isolation is a design approach aimed at protecting structures against damage from earthquakes by limiting the earthquake attack rather than resisting it. In seismically base-isolated systems, the superstructure is decoupled from the ground motion by introducing horizontally flexible but vertically very stiff components at the base level of the structure.

Thereby, the isolation system shifts the fundamental time period of the structure to a large value and/or dissipates the energy in damping, limiting the amount of force that can be transferred to the superstructure such that inter story drift and floor accelerations are reduced drastically.

Typical earthquake accelerations have dominant periods of about 0.1-1.0 sec. with maximum severity often in the range 0.2-0.6 sec. Structures whose natural periods of vibration lie within the range 0.1-1.0 sec.

Are therefore particularly vulnerable to seismic attacks because they may resonate. The most important feature of seismic isolation is that its increased flexibility increases the natural period of the structure (>1.5 sec., usually 2.0-3.0 sec.)Because the period is increased beyond that of the earthquake, resonance is avoided and the seismic acceleration response is reduced. The benefits of adding a horizontally compliant system at the foundation level of a building can be seen in Figure 1.1.

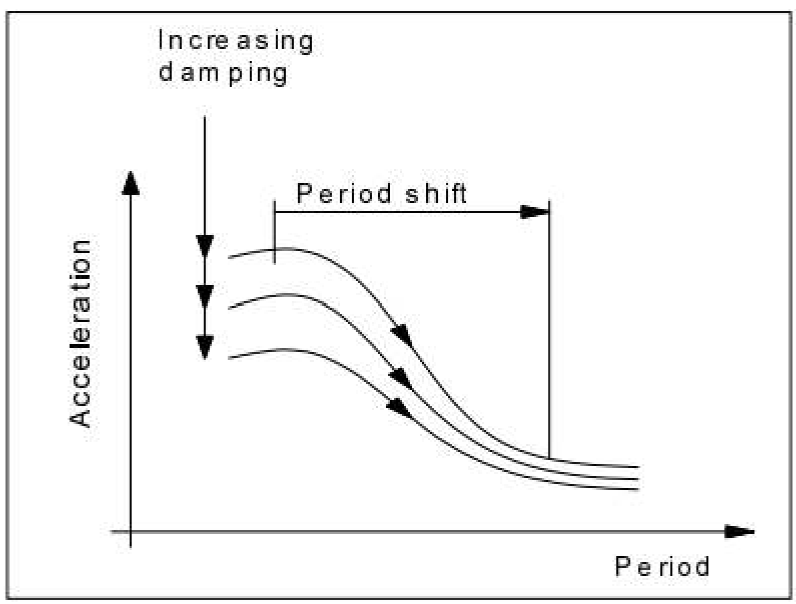

Figure 1.1 Acceleration response spectrum (Skinner et al., 1993)

In Figure 1.1, note that a rapid decrease in the acceleration transmitted to the isolated structure as the isolated period increases. This effect is equivalent to a rigid body motion of the building above the isolation level.

The displacement of the isolator is controlled (to 100-400 mm) by the addition of an appropriate amount of damping (usually 5-20 % of critical). The damping is usually hysteretic, provided by plastic deformation of either steel shims or lead or ‘viscous’ damping of high-damping rubber. For these isolators strain amplitudes, in shear, often exceed 100%. The high damping has the effect of reducing the displacement by a factor of up to five from unmanageable values of ~1.0 m to large but reasonable sizes of <300 mm. High damping may also reduce the cost of isolation since the displacements must be accommodated by the isolator components and the seismic gap, and also by flexible connections for external services such as water, sewage, gas and electricity.

The effect of damping for controlling displacement is shown in Figures 1.1 and 1.2, with increased damping reducing both the displacement and the accelerations of the structure. But damping higher than 0.20 should be avoided since high isolator damping may seriously distort mode shapes, and complicate the analysis. In addition, higher mode responses increase as the damping increases. As a result of this substantial contribution of higher modes, seismic inertia forces increase when compared with those produced by only the first mode.

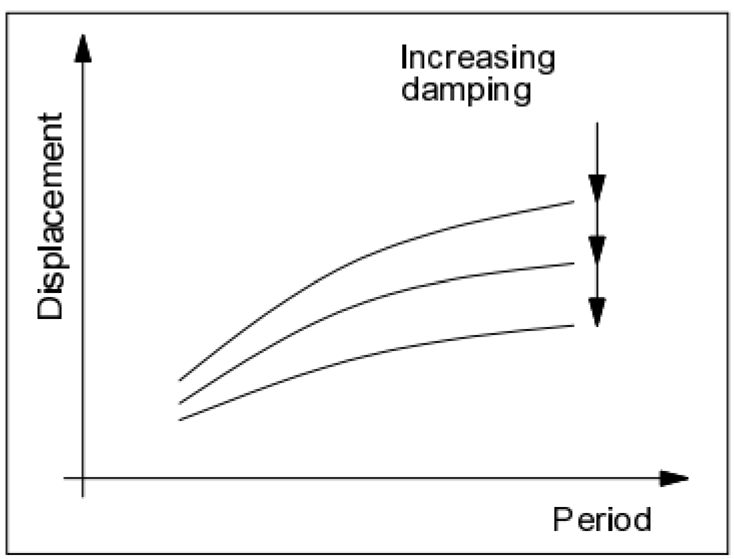

Figure1.2 Displacement response spectrum (Skinner et al., 1993)

1.2 Seismic Isolation Systems

Structural engineers typically employ one of two strategies to protect buildings from the damaging effects of earthquakes. In the first strategy, engineers design the lateral force resisting system to be very stiff, producing a structure with a short natural period (or high natural frequency), small drifts and large accelerations in design earthquake shaking. Such framing systems will protect drift-sensitive components (such as masonry walls) but could damage acceleration-sensitive components (including mechanical equipment, computers, and electrical systems). Historically, engineers of nuclear structures have used this design strategy to make sure that horizontal deformations in nuclear power plants are very small.

Reinforced concrete nuclear power plant structures have been designed using stiff structural systems such as reinforced concrete shear walls or steel braced frames that remain essentially elastic in design earthquake shaking but are detailed per ACI 349 (ACI 2006) to respond in a ductile manner in the event of shaking more intense than the design basis. Nuclear plant structures also commonly use very thick concrete structures to provide radiation shielding. These approaches differ from conventional building design wherein substantial inelastic response and damage is anticipated in design basis shaking.

In the second strategy, engineers design the lateral force resisting system to be flexible, producing a structure with a long natural period (or small natural frequency). Moment frames of structural steel and reinforced concrete are typical flexible seismic framing systems. As a result, earthquake-induced horizontal accelerations and forces are (relatively) small, but transient drifts are large, requiring structural elements and details that allow for large deformation capacity and significant resilience, and residual drifts might be significant.

This design approach is common for steam turbine buildings, particularly for pressurized water reactor (PWR) plants. The turbine-generator may be spring mounted to isolate it vertically and horizontally. Gen IV plants may use compact closed gas cycles, which would likely be mounted on the same base-isolated foundation as the reactor. Vertical isolation may still be required for turbine-generator equipment.

Seismic isolation is a structural design approach that aims to control the response of a structure to horizontal ground motion through the installation of a horizontally flexible and vertically stiff layer of structural isolation hardware between the superstructure and its substructure. The dynamics of the structure is thus changed such that the fundamental vibration period of the isolated structural system is significantly longer than that of the original, non-isolated structure, leading to a significant reduction in the accelerations and forces transmitted to the isolated structure and significant displacements in the deformable, structural base isolation layer.

The structural base isolation devices are designed to sustain such large deformations without damage and to return the structure to its original positions, thus leaving no residual deformation. The structure above the isolation layer is designed to sustain the (reduced, and well-known) horizontal forces transmitted through the isolation layer, usually such that it responds in its elastic response range and remains undamaged .

The design of the isolator system must include sufficient space for access to inspect and replace individual isolators. The components of a horizontally seismically isolated structure are shown in Figure1.3 .

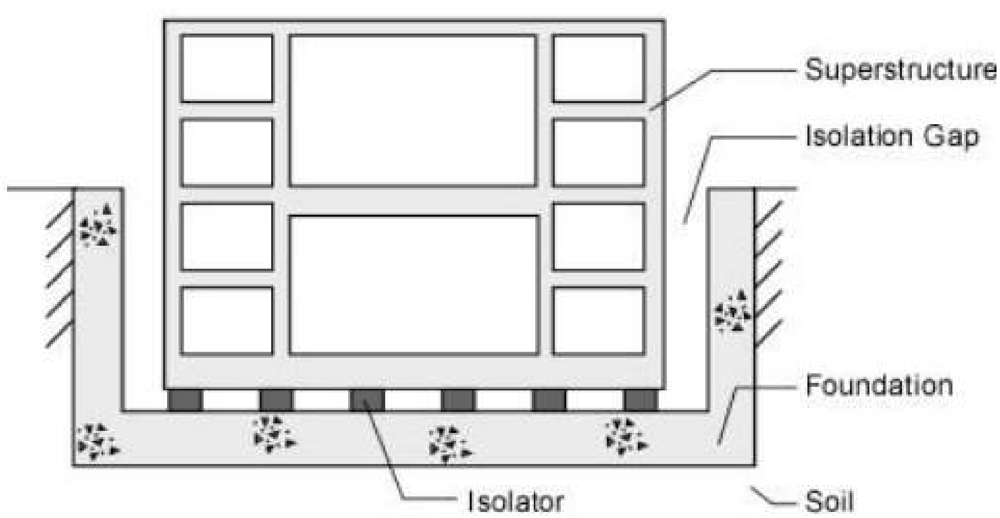

Figure1.3 Components of a base isolated structure shown in elevation.

(Bland ford et al., 2009)

1.2.1 Elastomeric Bearings

The system that has been adopted most widely in recent years is typified by the use of elastomeric bearings, the elastomeric is of either natural rubber or neoprene. In this approach, the building or structure is decoupled from the horizontal components of the earthquake ground motion by interposing a layer with low horizontal stiffness between the structure and the foundation. This layer gives the structure a fundamental frequency that is much lower than its fixed-base frequency and also much lower than the predominant frequencies of the ground motion. The first dynamic mode of the isolated structure involves deformation only in the isolation system, the structure above being to all intents and purposes rigid. The higher modes that will produce deformation in the structure are orthogonal to the first mode and consequently also to the ground motion. These higher modes do not participate in the motion, so that if there is high energy in the ground motion at these higher frequencies, this energy cannot be transmitted into the structure. The isolation system does not absorb the earthquake energy, but rather deflects it through the dynamics of the system.

This type of isolation works when the system is linear and even when undamped; however, some damping is beneficial to suppress any possible resonance at the isolation frequency.

1.2.1.1 Low Damping Natural and Synthetic Rubber Bearings

Low-damping natural rubber bearings and synthetic rubber bearings are used in conjunction with supplementary damping devices, such as viscous dampers, steel bars, frictional devices, and so on. The elastomeric used in bearings may be natural rubber or neoprene. The isolators have two thick steel endplates and many thin steel shims. The rubber bearing consists of layer of rubber 5-20 mm thick, vulcanized between steel shims. The rubber layers give the bearing it’s relatively low shear stiffness in the horizontal plane while the steel plates control the vertical stiffness and also determine the maximum vertical load, which can be applied safely. The steel plates also prevent the bulging of the rubber.

The material behavior in shear is quite linear up to shear strains above 100%, with the damping range of 2-3% of critical. The material is not subject to creep, and long-term stability of the modulus is good .Low-damping elastomeric laminated bearings are simple to manufacture (the compounding and bonding processes to steel is well understood), easy to model, and their mechanical response is unaffected by rate, temperature, history, or aging. However in the structures they must be used with supplementary damping systems. These supplementary systems require elaborate connections and, in the case of metallic dampers, are prone to low-cycle fatigue .

1.2.1.2 High Damping Natural Rubber Bearings

The energy dissipation in high-damping rubber bearings is achieved by special compounding of the elastomeric. Damping ratios will generally range between 8% and 20% of critical. The shear modulus of high-damping elastomeric generally ranges between 0.34 MPa and 1.40 MPa. The material is nonlinear at shear strains less than 20% and characterized by higher stiffness and damping, which minimizes the response under wind load and low-level seismic load. Over the range of 20-120% shear strain, the modulus is low and constant. At large shear strains, the modulus and energy dissipation increase.

This increase in stiffness and damping at large strains can be exploited to produce a system that is stiff for small input, is fairly linear and flexible at design level input, and can limit displacements under unanticipated input levels that exceed design levels .

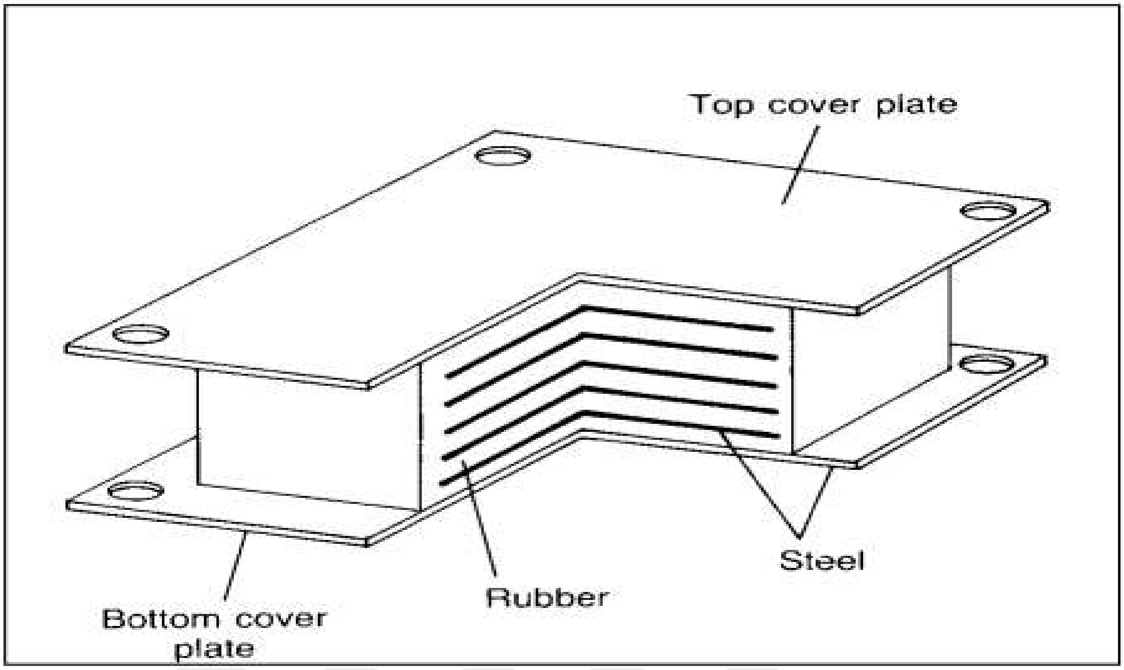

Figure1.4 High damping rubber bearing(Kunde et al., 2003)

1.2.1.3 Lead-Plug Bearings

Lead-plug bearings are generally constructed with low-damping elastomers and lead cores with diameters ranging 15% to 33% of the bonded diameter of the bearing. Laminated-rubber bearings are able to supply the required displacements for seismic isolation. By combining them with a lead-plug insert which provides hysteretic energy dissipation, the damping required for a successful seismic isolation system can be incorporated in a single compact component. Thus one device is able to support the structure vertically, to provide the horizontal flexibility together with the restoring force, and to provide the required hysteretic damping. The maximum shear strain range for lead-plug bearings varies as a function of manufacturer but is generally between 125% and 200%.

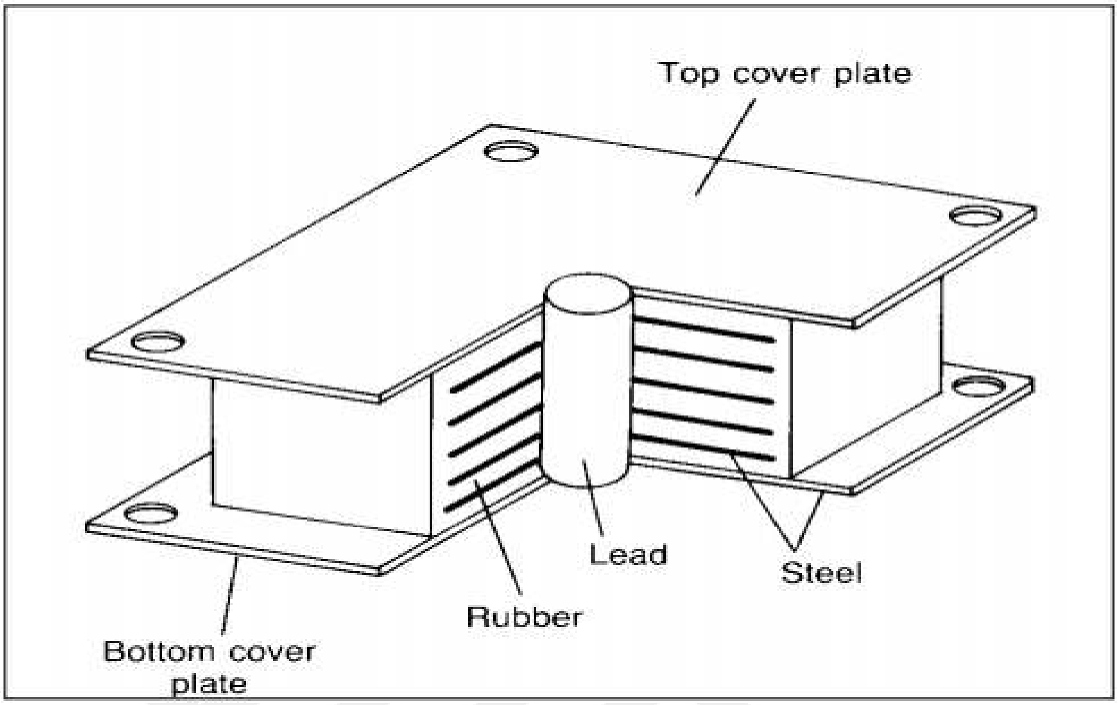

Figure 1.5 Lead-Plug bearing (Kunde et al., 2003)

1.2.2 Isolation Systems Based on Sliding

One of the most popular and effective techniques for seismic isolation is through the use of sliding isolation devices. The sliding systems perform very well under a variety of severe earthquake loading and are very effective in reducing the large levels of the superstructure's acceleration. These isolators are characterized by insensitivity to the frequency content of earthquake excitation. This is due to tendency of sliding system to reduce and spread the earthquake energy over a wide range of frequencies. The sliding isolation systems have found application in both buildings and bridges. The advantages of sliding isolation systems as compared to conventional rubber bearings are:

(i) Frictional base isolation system is effective for a wide range of frequency input

(ii) Since the frictional force is developed at the base, it is proportional to the mass of the structure and the center of mass and center of resistance of the sliding support coincides Consequently, the tensional effects produced by the asymmetric building are diminished.

1.2.2.1 Pure-Friction System

The simplest sliding isolation system is the pure friction system. In this system a sliding joint separates the superstructure and the substructure. The use of layer of sand or roller in the foundation of the building is the example of pure-friction base isolator. The pure-friction type base isolator is essentially based on the mechanism of sliding friction. The horizontal frictional force offers resistance to motion and dissipates energy.

Under normal conditions of ambient vibrations and small magnitude earthquakes, the system acts like a fixed base system due to the static frictional force. For large earthquake the static value of frictional force is overcome and sliding occurs thereby reducing the accelerations. as shown in Figure1.6.

Figure1.6 Low friction bearing device(Source: ISET Journal, paper no.545,2005 by Oliveto & Marletta, university of Catania, Italy)

1.2.2.2 Resilient-Friction Base Isolation System

Mostaghel and Khodaverdian (1987) proposed the resilient-friction base isolation (R-FBI) system as shown in Figure 1.7. This base isolator consists of concentric layers of Teflon-coated plates that are in friction contact with each other and contains a central core of rubber. It combines the beneficial effect of friction damping with that of resiliency of rubber. The rubber core distributes the sliding displacement and velocity along the height of the R-FBI bearing. They do not carry any vertical loads and are vulcanized to the sliding ring. The system provides isolation through the parallel action of friction, damping and restoring force .

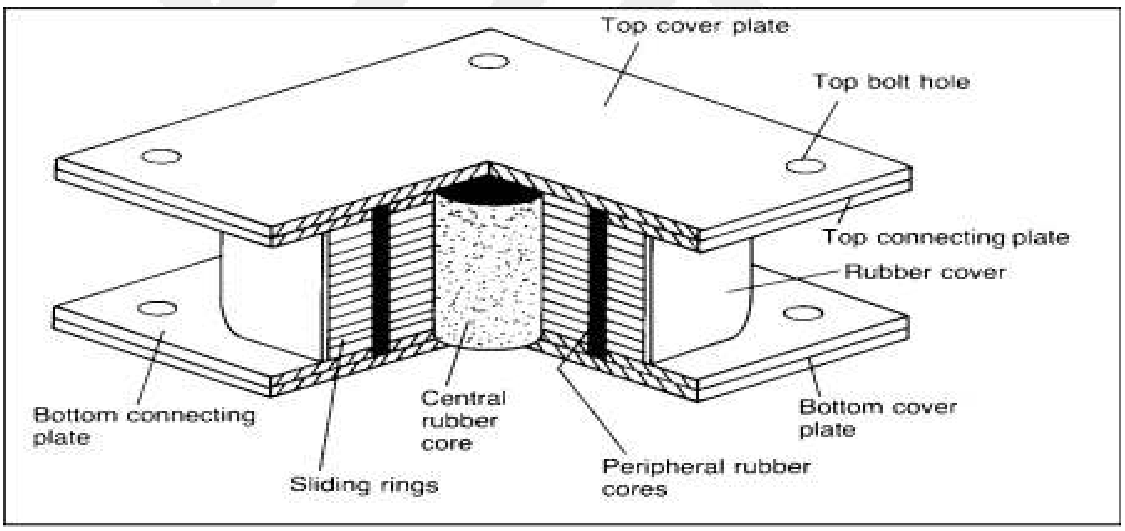

Figure1.7 Resilient-friction base isolation system (Kunde et al., 2003)

1.2.2.3 Electric de France System

An important friction type base isolator is a system developed under the auspices of "Electric de France" (EDF) (Gueraud et al., 1985). This system is standardized for nuclear power plants in region of high seismicity. The base raft of the power plant is supported by the isolators that are in turn supported by a foundation raft built directly on the ground.

The main isolator of the EDF consists of laminated (steel reinforced) neoprene pad topped by lead-bronze plate that is in friction contact with steel plate anchored to the base raft of the structure.The friction surfaces are designed to have a coefficient of friction of 0.2 during the service life of the base isolation system.

The EDF base isolator essentially uses elastomeric bearing and friction plate in series. An attractive feature of EDF isolator is that for lower amplitude ground excitation the lateral flexibility of neoprene pad provides base isolation and at high level of excitation sliding will occur which provides additional protection. This dual isolation technique was intended for small earthquakes where the deformations are concentrated only in the bearings. However, for larger earthquakes the bronze and steel plates are used to slide and dissipate seismic energy. The slip plates have been designed with a friction coefficient equal to 0.2 and to maintain this for the life time of the plant .

1.2.2.4 Friction Pendulum System

The concept of sliding bearings is also combined with the concept of a pendulum type response, obtaining a conceptually interesting seismic isolation system known as a friction pendulum system (FPS) (Zayas et al., 1990) as shown in Figure 1.8. In FPS, the isolation is achieved by means of an articulated slider on spherical, concave chrome surface. The slider is faced with a bearing material which when in contact with the polished chrome surface, results in a maximum sliding friction coefficient of the order of 0.1 or less at high velocity of sliding and a minimum friction coefficient of the order of 0.05 or less for very low velocities of sliding. The dependency of coefficient of friction on velocity is a characteristic of Teflon-type materials (Mokha et al., 1990).

The system acts like a fuse that is activated only when the earthquake forces overcome the static value of friction. Once set in motion, the bearing develops a lateral force equal to the combination of the mobilized frictional force and the restoring force that develops as a result of the induced rising of the structure along the spherical surface.

If the friction is neglected, the equation of motion of the system is similar to the equation of motion of a pendulum, with equal mass and length equal to the radius of curvature of the spherical surface. The seismic isolation is achieved by shifting the natural period of the structure. The natural period is controlled by selection of the radius of curvature of the concave surface. The enclosing cylinder of the isolator provides a lateral displacement restraint and protects the interior components from environmental contamination. The displacement restraint provided by the cylinder provides a safety measure in case of lateral forces exceeding the design values .

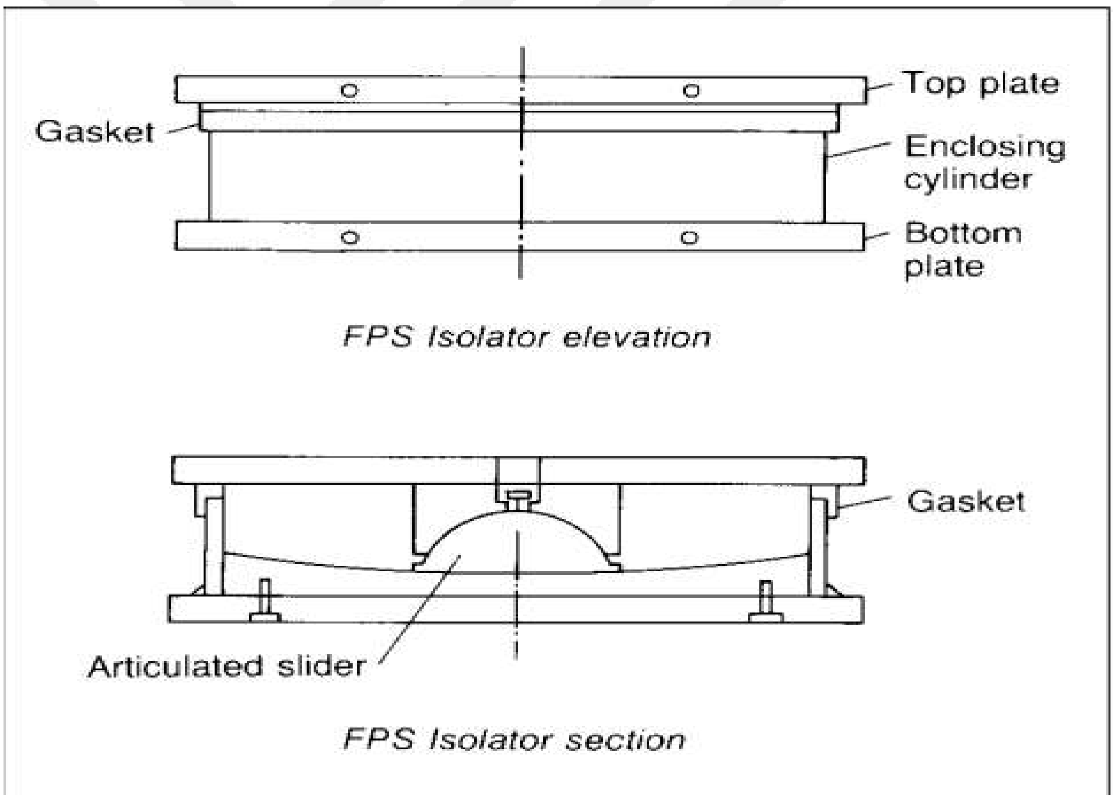

Figure1.8 Friction pendulum system (Kunde et al., 2003)

1.3 Current Applications

The buildings that are designed as base isolated commonly fit into one of only a few categories. Most structures can be classified as essential buildings with an occupancy category of IV as defined by ASCE 7.

In other words essential buildings are structures that are highly important and that the public must rely on to provide critical services. Such buildings are hospitals, fire stations, buildings housing crucial technology, and certain government structures. Another building group commonly retrofitted to an isolated structure is historical buildings. For base isolation to normally be considered buildings must require special performance levels well beyond that which is offered by traditional fixed base designs. This section will discuss why certain building types are isolated and why others are not. A number of examples of base isolated structures are also provided.

When buildings are analyzed for the application of base isolators the first consideration is what the desired level of functionality of the structure is to be after an earthquake. Most buildings that are designed with base isolation are essential facilities, in other words, their expected performance level is immediate occupancy following a seismic event. For example, if a hospital required the evacuation of the facility after an earthquake because the structure was damaged and needed inspected or repair the facility may not be usable during that time.

It may cost the facility a significant amount of money for inspections of damage, making any required repairs, and the relocation of patients and employees. Additionally in the event of an earthquake this could mean that citizens injured because of the seismic event could not be treated in damaged facilities.

This is just one example of why essential facilities may be chosen to be isolated. Another example of a type of structure often base isolated is historical buildings. Historic buildings do not normally serve the public in any essential way such as hospitals, but they have irreplaceable, historic significance.

In high to moderate seismic zones, historic structures are subject to extreme events and movement produced by earthquakes for which they were not initially designed to withstand. Some historic structures may be very weak with respect to resisting seismic forces.

However, in the preservation of historical structures and strengthening them to resist seismic forces, it is unreasonable to completely rebuild the structure based on the up-to-date code or even to modify it in the slightest way that might change its historical features.

Many historical buildings may also be constructed of unreinforced facades, something not allowed in current seismic design for higher seismic areas. Base isolation provides a way that historical structures can be renovated without altering the buildings appearance or historical significance while increasing its probable life expectancy due to a significant seismic event.

Located just 23 miles away from the 1994 Northridge earthquake epicenter in Los Angeles is the University of Southern California (USC) Hospital (Kelly 1991). This was the world’s first base isolated hospital and just the seventh base isolated structure in the United States.

The base isolation system was designed by KPFF Consulting Engineers. The USC Hospital was fully operational during and after the 1994 earthquake and did not receive any damage to the structure or its contents, whereas many adjacent structures were substantially damaged.

A combination of isolator types make up the isolation system of the USC Teaching Hospital. There are 68 lead rubber isolators and 81 elastomeric isolators. The isolated USC Hospital has been believed to have gone through the most extreme seismic ground motions of any isolated structure built thus far in the United States.

The Emergency Operations Center in state of Washington (USA) was designed similarly to other isolated structures to be operational during and after a significant earthquake. There are 33 friction pendulum isolators, shown in Figure 1.9, acting to decouple the structure from the ground motions produced by an earthquake. Construction of the two-story, 28,000 square foot center was completed in 1998 according to the Washington Military Department. Located only 10 miles from the epicenter of the 2001 Nisqually earthquake, the structure received no damage during that event and was fully functional (Olson, 2007).

Figure1.9 Washington State Emergency Operation Center Friction Pendulum System (FPS) Isolators. (Photo Courtesy of KPFF Consulting Engineers, 2004)

In Turkey, especially in some of the prestigious projects the application of seismic isolation technique has started to be used in recent years. For example, the new $300million International Terminal at Istanbul's Ataturk Airport is a massive, two hundred and fifty thousand square meters building that serves as the port of entry for14 million international air passengers to Turkey each year.

The main feature of the terminal is the 250m by 225m pyramidal roof space frame with triangular skylights. This delicate roof structure is supported by 130 friction pendulum bearings, placed between the roof frame and the concrete columns that rise 7m above the departure level. The bearings protect the delicate roof glass, glass curtain walls, and the cantilever columns in the event of a major earthquake. Some of the important isolated buildings are listed in Table 1.1.

Table1.1 Current applications of seismic isolation

No	**Buildings**	**Year**	**City**	**Size**	**Description**
1	Emergency Operations Center	1998	Los Angeles, USA	8000 m^2 N = 3	28 high damping natural rubber bearings
2	Flight Simulator Manufacturing Facility	1998	Salt Lake City, USA	10800 m^2 N = 4	High damping natural rubber 6% less cost than conventional design.
3	Auto Zone Office Building	1997	Memphis, USA	23230 m^2 N = 8	43 rubber isolators $27 million
4	M.L. King / C.R. Drew Diagnostics Trauma Center	1995	Willowbro ok Los Angeles, USA	13000m^2 N=5	70 high damping rubber bearings, 12 sliding pad D=100cm
5	Foothill Law and Justice Center, San Bernardino	1985	Los Angeles, USA	15800 m^2 N = 4	21 km distance to the fault, 98 bearings, $38 million high damping natural rubber.
6	Fire Command and Control Facility	1997	Los Angeles, USA	5000 m^2 N = 2	High damping natural rubber 6% less cost than conventional design.
7	Traffic Management Center for Caltrans, Kearny Mesa	1997	San Diego, USA	N = 2	40 bearings t= 60 cm T_d = 2,5 sec., d_D= 25 cm
8	West Japan Postal Computer Center, Sanda	1986	Sanda, Japan	47000 m^2 N = 6	120 elastomeric pads, T_d=3,9 sec.
9	Hayward City Hall, CA	1999	Hayward, USA	14000m^2	Friction Pendulum Isolators

1.4 Literature Survey

The codes and guidelines for design of structures with seismic base isolation have evolved from the design provisions that were developed in the 1980 by a subcommittee of the Structural Engineers Association of Northern California(SEAONC). In 1986 SEAONC published a simple regulation titled “Tentative Seismic Isolation Design Requirements”, also known as the Yellow Book. The Yellow Book, mainly based on equivalent static design methods, implemented in the various editions of the Uniform Building Code (UBC) , the most widely used code for design of earthquake resistant buildings in the United States. The Yellow Book modified and became the 1991 version of the UBC, “Earthquake Regulations for Seismic-Isolated Structures”. The 1994 and 1997 versions of UBC are elaborated. As the UBC-97 has evolved, International Building Code 2006 (IBC2006) has become extremely complex code based mainly on dynamic methods of design.

Abe et al., (2004) performed tests on various bearing materials to determine their properties such as stiffness and multi-directional behavior. The tests performed were the biaxial load test, in which a constant vertical load and a variable horizontal load were applied; a biaxial load test, in which a second variable horizontal loading was applied perpendicular to the biaxial test load; and a small amplitude test, in which the horizontal loading is minimal to determine the resistance behavior of the bearings under small deflections. The test results were then used to ascertain the accuracy of mathematic models that were developed in tandem with the experiments.

The experiments performed by Abe et al., (2004) suggest that the vertical force acting through the bearings affects their stiffness and damping properties. This effect is particularly visible in the response of the lead-plug rubber bearing, due to a closing of the gap between the plug and the rubber.

However, it should be noted that for large deformations the damping ratio and stiffness values become more stable, and less dependent upon the vertical loading. Further research must be undertaken to ascertain a relationship between changes in the vertical loading and the response of the bearings.

Mostaghel and Khodaverdian (1988) wrote a paper on the dynamic response of base-isolated structures. Their paper focused on friction-based isolation systems, and therefore introduced the friction. The work presented in their paper is, however, restricted to unidirectional motion, considering only one horizontal degree of freedom and the vertical ground motion, which is integral to the frictional effect.

Matsagar and Jangid, in their noteworthy paper, present a study about the influence of isolator characteristic parameters on the response of multi-story base isolated structures. The effects of the shape of isolator loop and superstructure flexibility on the seismic response is investigated

1.5 Aims and Scope

The aim of this study is to investigate the effects of seismic base isolation systems, especially rubber bearings, on the response of structures. The study includes analysis of the seismic responses of isolated structures, which is oriented to give a clear understanding of the processes involved and discussion of High Damping Natural Rubber Bearings isolator.

They study illustrate its use in both static and dynamic analyses. The static equivalent lateral force of analysis, response spectrum and time history analysis and compare between base isolation and base fixed are carried out in case study. Design procedures used for base isolated systems are discussed and form the basis for preliminary design procedures. Using a consistent set of design criteria, a commercial computer program SAP2000 demonstrates the ease with which the design for isolated system may be executed.

CHAPTER 2

Base Isolation Design Requirements

The 2006 International Building Code (IBC 2006) indicates that the current edition of the American Society of Civil Engineers, ASCE 7-05, *Minimum Design Loads for Buildings and Other* (ASCE 2006), is to be used for building seismic design. However, regardless of location, designers must abide by the current adopted building codes which may have been amended by local jurisdictions. Since the development of base isolation, ASCE 7-05 has incorporated Chapter 17, entitled “Seismic Design Requirements for Seismically Isolated Structures”. This section gives various requirements for the design of base isolated structures including definitions for many common terms such as displacement, damping, scragging, and several other main isolation components.

2.1 Seismic Design Performance Levels

The primary goal of a building code is to provide a minimum basis of design for life safety and collapse prevention. Base isolation provides a type of safety and structural design beyond what is normally required of life safety. Base isolation can be divided into two seismic design levels, enhanced and superior performance. Structural performance of isolated structures is well above the performance level of any fixed-base seismic design. Fixed-base buildings, are only designed for life safety, in which the structure is capable of withstanding design level ground motions with a rare occurrence of 475-years in all regions. However, this performance level is only a minimum design basis. Structures are not expected, under normal fixed-base designs, to maintain structural integrity or be capable of occupancy after a design based seismic event. ASCE 7 determines the required forces which a structure should be able to resist without collapse and to provide life safety to the buildings occupants and any bystanders. Base isolation can be designed for “superior performance” which has the capabilities to resist very large magnitude seismic hazards which may only occur once over a 2500-year return prediction in high to moderate seismic regions. In all regions with seismic hazards, a very rare seismic event has a 2,500-year reoccurrence probability. Figure 2.1 illustrates the different seismic levels of performance that a structure can be designed for and the type of performance

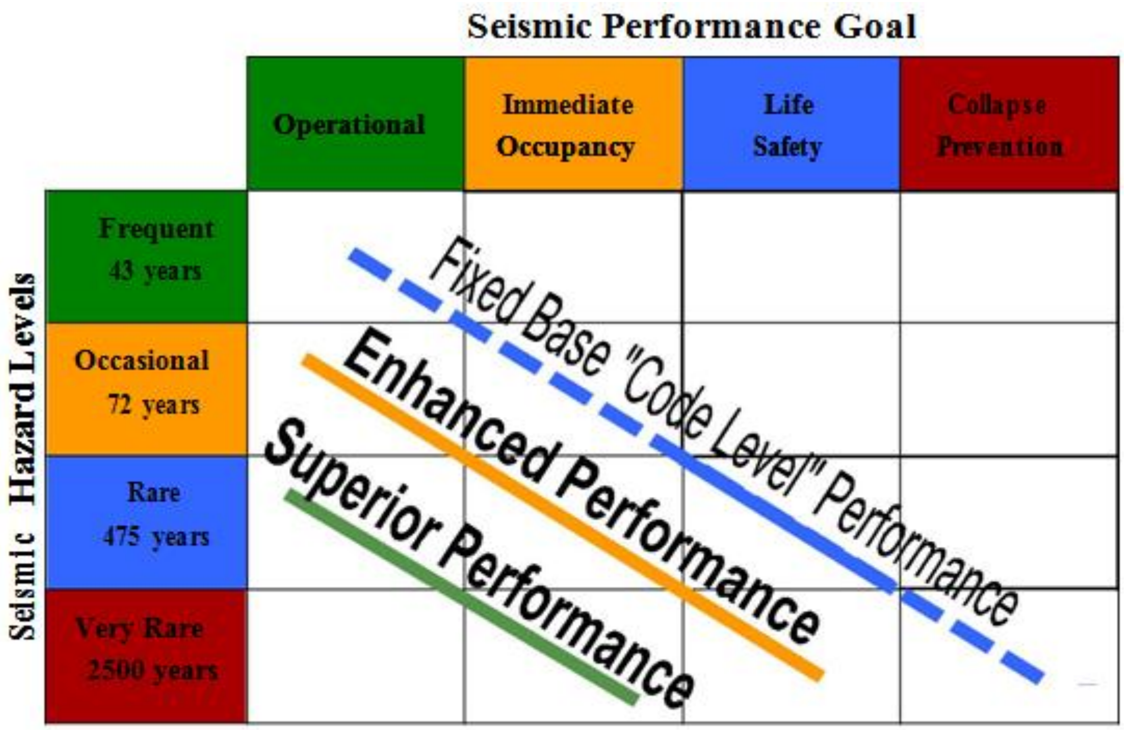

Figure 2.1 Comparison of Seismic Performance Levels for Conventional Fixed-Base Structures and Base Isolated Structures. (Figure Reproduced from Andrew Taylor,1994)

2.2 Static Analysis

Non-isolated and isolated structures use a simplified static procedure or a rigorous dynamic procedure for analysis which has various advantages and disadvantages when employed. ASCE 7 allows isolated structures to be designed using one of three defined methods of analysis. The equivalent lateral force procedure (ELFP) is the most conservative and simplified method of design.

This analysis is based on equations formulated to best represent the performance requirements of a superstructure and its isolation system during an earthquake. ASCE 7 places limitations which are listed below, on the types of structures and site properties that can be used with the ELFP. Since the ELFP does not have any requirement to use structural analysis software based on the actual building configuration that would allow for more accurate results, their equations produce conservative design values. Conservative requirements are ideal for use with small, regular structures on site locations that will not amplify or produce extreme ground motions. ASCE 7 section 17.4 indicates that an equivalent lateral force design application for base isolated structures can be implemented when the following conditions apply:

1. S_1, the mapped maximum considered earthquake spectral response acceleration parameter, is less than or equal to 0.60g
2. Site classification investigation of the soil indicates it is Class A, B, C, or D
3. Height of the superstructure above the isolation interface is less than or equal to four stories or 65ft (20 m)
4. Effective period of the isolated structure is less than or equal to 3.0 sec at maximum displacement, T_M
5. The effective period of the isolated structure at the design displacement, T_D, is greater than three times the elastic, fixed-base period of the superstructure
6. The superstructure is of regular configuration with a symmetrical building plan having no vertical out-of-plane offsets causing a complex structural design
7. The isolation system meets the following criteria:

    - 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

- The isolation system produces restoring forces
- The isolation system does not limit maximum considered earthquake displacement to less than the total maximum displacement

These requirements are shown in Figure 2.2 for ease of determining the appropriate method of design. Even if the structure can be designed using the ELFP, it may be desirable to use a dynamic response history analysis. There are some benefits that are obtained from utilizing a dynamic response history analysis that would not otherwise be realized such as less conservative and more realistic building response values and lower design forces. Additional benefits are described in the following sections.

2.2.1 Equivalent Lateral Force Methodology

The ELFP is useful for isolated structures that are fairly simplistic and have more predictable properties. To determine if the ELFP is allowed to be used for the building and site properties under consideration, designers should follow the flowchart in Figure 2.2. Additionally, the limitations placed on the ELFP by ASCE 7, are listed in the previous section. Figure 2.2 will aid structural engineers determine not only if the ELFP can be utilized, but if that is not allowed, then a dynamic procedure is required. Of course, regardless of the building and site properties a dynamic response history analysis is always a permitted option. Once it is determined which analysis procedure can be utilized, the isolation devices and the superstructure can being designed. Figure 2.3 illustrates the order in which variables and design forces should be computed for the ELFP to obtain the required design values and will be continually referenced in this section.

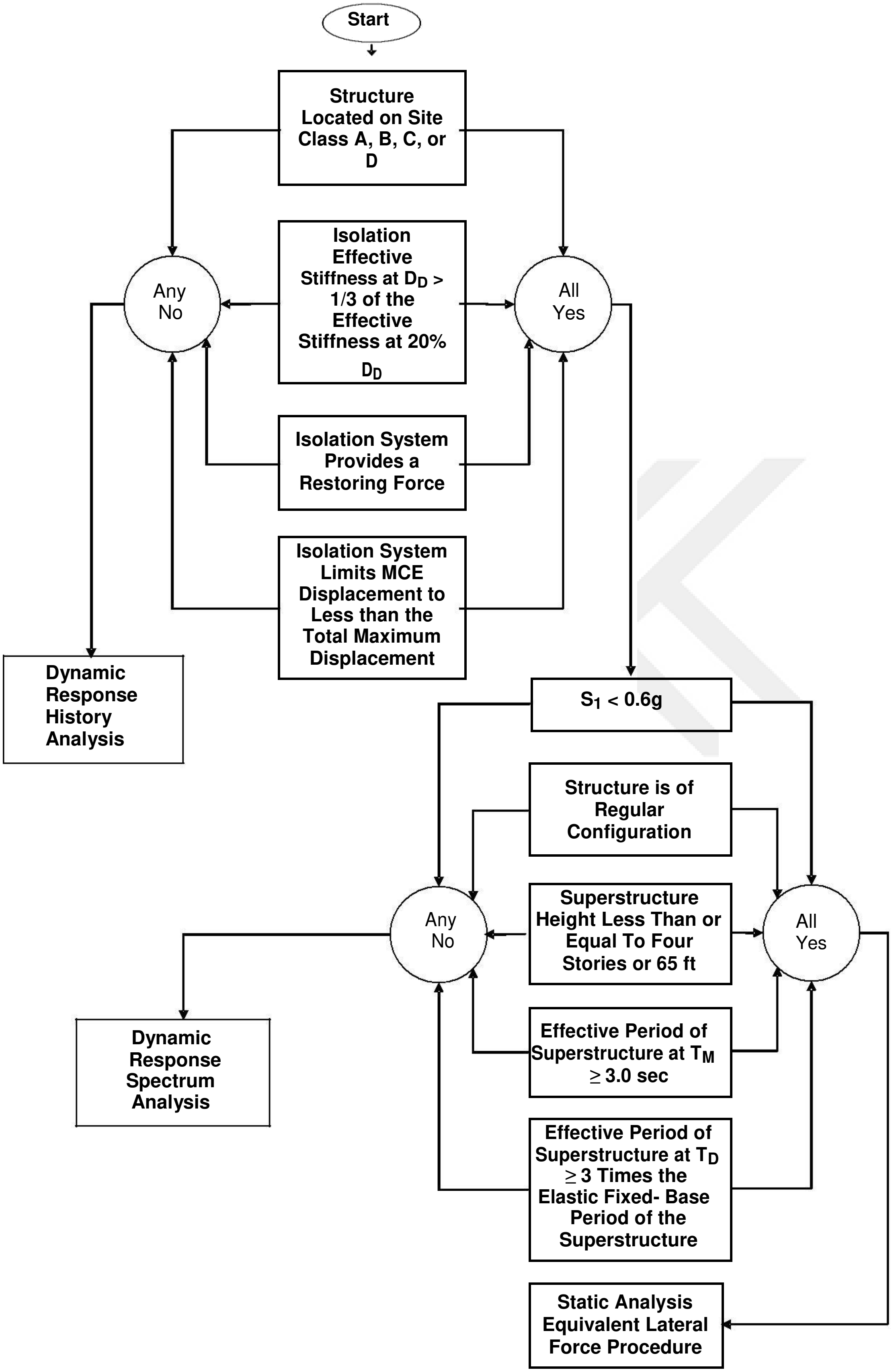

Figure 2.2 Determination of Permitted Analysis Procedure(ASCE2005)

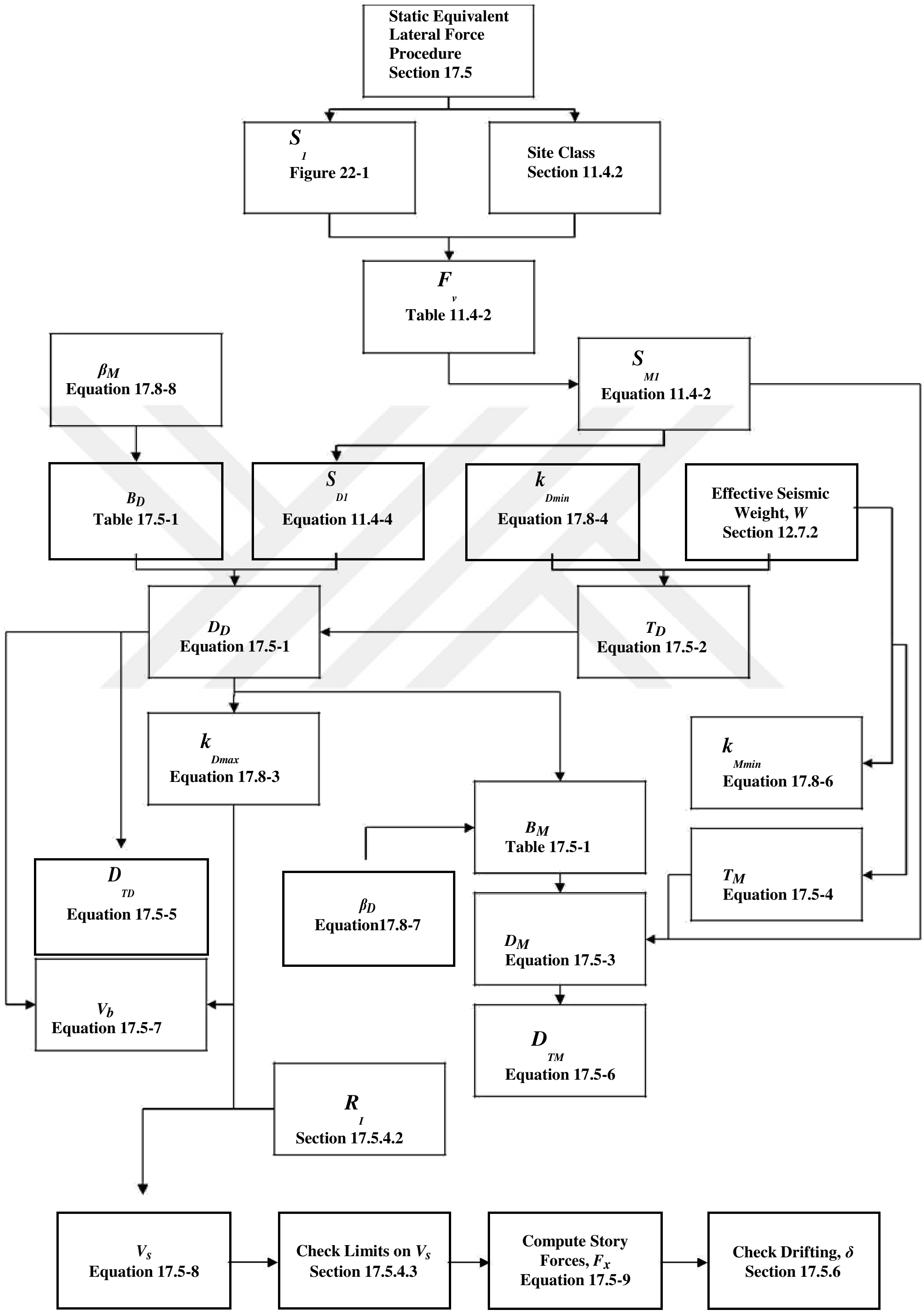

Figure 2.3 Equivalent Lateral Force Methodology(ASCE2005)

2.2.2 Structural Seismic Properties

The first requirement for the ELFP is to determine the site properties for the building being considered. The engineer needs to identify the maximum credible earthquake (MCE), 5 percent damped, spectral response acceleration parameter at a 1 second period, S_1. This value can be easily obtained by knowing the location of the building site and using the U.S. Geological Survey website (www.usgs.gov). Next, the spectral response acceleration parameter at a 1 second period is adjusted for the properties of the soil by a site coefficient, F_v, which depends on the value of S_1 as well as the sites soil classification. The product of the site coefficient and the 1 second spectral response parameter results in the variable S_{MS}, the MCE spectral response acceleration for short periods. Furthermore, the design spectral acceleration parameter for long periods, S_{D1}, is computed as 2/3 the value of S_{MS}. All of these values are determined no differently than if the structure were a fixed base building.

2.2.3 Isolation Properties

The properties of the isolation system need to be identified to further compute the base shear values for which the superstructure is designed to resist. Since many of the isolation properties depend on the next few design values these steps shown in Figure 2.3 become an iterative process. The design and maximum displacement values which depend on the properties of the isolation system should be determined to compute the response of the seismic forces on the isolation system used to design the superstructure. These properties include the effective damping values, β_D and β_M, the damping coefficients, B_D and B_M, and the maximum and minimum isolator stiffness, k_D and k_M. In schematic design, when the exact isolator properties are unknown, it is necessary to assume some initial properties.

For instance, the damping coefficient, B_D and B_M, depend on effective damping properties of the isolation system which are determined through testing of the isolator devices and are computed by Equations 2.1 and 2.2 (ASCE 7 equations 17.8-7 and 17.8-8).

However during the early stages of design, these coefficient values are unknown. Additionally, minimum and maximum isolator stiffness and maximum and design displacement properties are determined through testing requirements, as defined by ASCE 7.

Since the displacement of the isolator can govern many design elements, it may be appropriate to first establish the maximum amount of horizontal displacement feasible and permissible by the system. Or it may be more efficient to just assume values for the isolator stiffness and damping and compute the equations to verify if their results are adequate for preliminary design stages.

$$\beta_D = \frac{1}{2\pi} = \left(\frac{\sum \mathrm{E_D}}{K_{Dmax} \quad D^2{}_D}\right) \tag{2.1}$$

$$\beta_M = \frac{1}{2\pi} = \left(\frac{\sum \mathrm{E_M}}{K_{Dmax} \quad D^2{}_D}\right) \tag{2.2}$$

Where:

$\sum E_D$ = sum of the energy dissipated per cycle of in all isolator units at the test design displacement of D_D (kip-in.)

$\sum E_M$ = sum of the energy dissipated per cycle in all isolator units measured at the test maximum displacement of D_M (kip-in.)

k_{Dmax} = maximum effective stiffness of the isolation system at D_D in the horizontal direction under consideration (kips/in.)

k_{Mmax} = maximum effective stiffness of the isolation system at D_M in the horizontal direction under consideration (kips/in.)

D_D = design displacement at the center of rigidity of the isolation system (in.)

D_M = maximum displacement at the center of rigidity of the isolation system (in.)

2.2.4 Effective Period

For the purpose of understanding the ELFP design variables and respective equations listed in Figure 2.3 and discussed throughout this section, the isolator's stiffness and effective damping will be approximated. Initially assumed effective stiffness values of the isolation system are used in Equations 2.3 and 2.4 (ASCE 7 equations 17.5-2 and 17.5-4) to calculate the effective period of the isolated structure, T_D and T_M, at the corresponding design displacement and maximum displacement.

$$T_D = 2\pi \sqrt{\frac{w}{K_{Dmin} g}} \tag{2.3}$$

$$T_M = 2\pi \sqrt{\frac{w}{K_{Mmin} g}} \tag{2.4}$$

Maximum and minimum stiffness values, k_D and k_M, should be determined initially based on well-established engineering judgment and prior tests on similar components and earthquake responses. All initially assumed values should be later verified. For final designs, these properties should be obtained from testing of the specific isolator(s) designed for the structure. Isolation horizontal stiffness should be obtained through the cyclic testing of the base isolator devices per ASCE 7, Section 17.8.2.2, Items 2 and 3 as indicated in Section 17.8.5.

The minimum effective stiffness should be based on the cyclic forces applied to obtain the largest displacement values. Similarly, the maximum effective stiffness should be obtained from the cyclic forces applied to obtain the smallest displacement values.

The corresponding displacement values along with the induced cyclic forces are used to compute the stiffness of the isolation.

device(s). Figure 2.4 is a theoretical hysteretic loop showing the relationship between force, stiffness and displacement for a lead core elastomeric isolator. A hysteretic loop that represents the actual isolation response would have smooth transitions between the relationship of the maximum displacement and the maximum yield force being applied. The amount of energy dissipated is equivalent to the area of the hysteretic loop.

The variables, k_{eff}, k_d, and k_u, represent the effective stiffness of a lead-plug bearing at horizontal displacement, post-elastic stiffness, and elastic unloading stiffness respectively. Yield force variables, F_y, F_{Max}, and Q_d, represent the isolator yield force, the maximum yield force, and the yield force of a lead plug, respectively. The displacement variables, D_y and D_{Max}, correspond to the yield force displacement and the maximum bearing displacement.

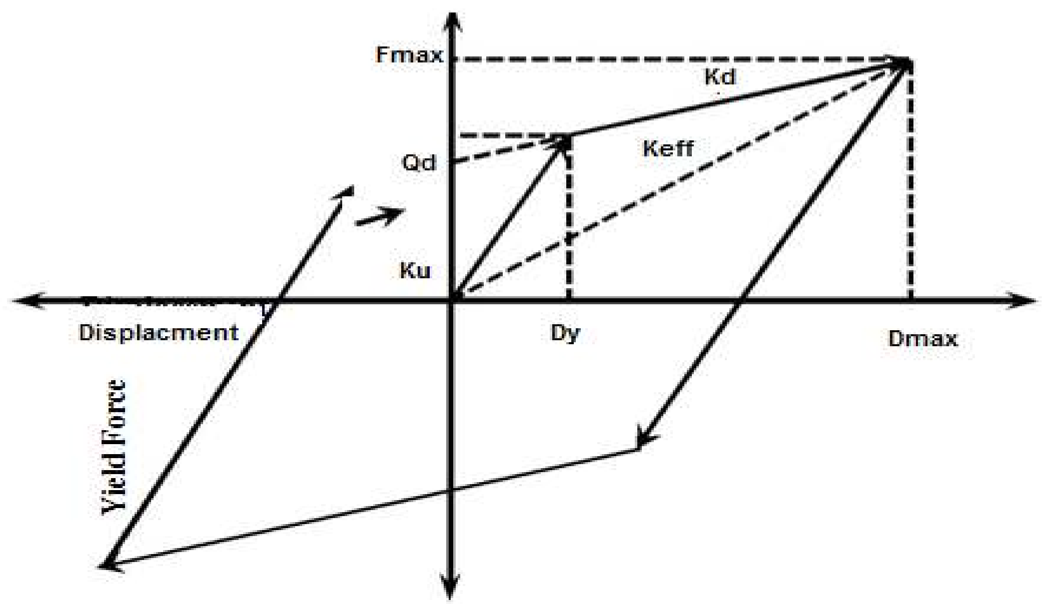

Figure 2.4 Theoretical Hysteretic Loop for Elastomeric Isolator (Kelly& Naeim,1999)

The acceleration of gravity, g, and the seismic weight, W, of the superstructure are also required to determine the effective periods. Total seismic weight is computed in the same manner as for fixed base structures. However, the seismic weight should not include any weight of elements below the isolation interface.

2.2.5 Isolation System Displacement

By assuming an initial value for the effective damping, the damping coefficient, B_D or B_M can be established from Table 17.5-1 of ASCE 7, where the subscripts D and M represent the design and maximum values respectively. Notice from this table, a damping coefficient of 5% effective damping has a value of unity. This value corresponds to the normal assumed damping value of a fixed base structure. As the effective damping of the isolation system increases, the damping coefficient increases. The design displacement and the maximum displacement are then determined by using the damping coefficients.

As the systems effective damping increases displacement decreases and represents the effects of damping on the movement of the overall system. The displacement that the isolation system should be able to withstand is computed as a function of the spectral acceleration parameter, S_{D1} or S_{M1}, the acceleration due to gravity, g, the effective period, T_D or T_M, and the damping coefficient, B_D or B_M. These relationships for D_D and D_M can be shown in the flow chart represented by Figure 2.3. Displacement values are computed using the following Equations 2.5 and 2.6 (ASCE 7 equations 17.5-1 and 17.5-3). However, these equations do not include any inherent or accidental torsion that may be inducted into the system.

$$D_D = \left(\frac{g}{4\pi^2}\right)\frac{S_{D1}T_D}{B_D} \quad (2.5)$$

$$D_M = \left(\frac{g}{4\pi^2}\right)\frac{S_{M1}T_M}{B_M} \quad (2.6)$$

2.2.6 Total Isolation System Displacement

Once the displacement that must be withstood by the isolation system is obtained, the total displacements, D_{TD} and D_{TM}, should then be computed. The total displacement of the isolation system due to any accidental or inherent torsion is taken into account in Equations 2.7 and 2.8 (ASCE 7 equations 17.5-5 and 17.5-6).

$$D_{TD} = D_D\left(1 + y\frac{12e}{b^2 + d^2}\right) \tag{2.7}$$

$$D_{TM} = D_M\left(1 + y\frac{12e}{b^2 + d^2}\right) \tag{2.8}$$

Inherent torsion occurs when the center of mass of the structure does not correspond to the structures center of rigidity of the lateral force resisting system. When the center of mass and center of rigidity coincide, torsion can still be induced accidentally into the structure. Accidental torsion is a result of a number of unpredictable features such as the orientation of ground motions, calculation errors in structural stiffness, and asymmetrical distribution of live loads, to name a few.

Tensional displacement can cause an increase in the displacement of the isolation system beyond what was computed. The variables e, b, d, and y in Equations 2.7 and 2.8 are physical properties of the structure. Variable e represents the actual horizontal eccentricity between the center of mass of the superstructure above the isolation interface and the center of rigidity of the isolation system including accidental eccentricity

Accidental eccentricity is computed as 5% of the longest structural plan dimension which is perpendicular to the direction of the force under consideration as required by ASCE 7.The shortest structural plan dimension and the longest structural plan dimension, measured perpendicular to one another, are represented by variables b and d, respectively.

The distance measured perpendicular to the direction of the ground motions, between the center of rigidity of the isolation system and the structural element under consideration is represented by the variably

Applying these variables as indicated in the design equations and multiplying by the appropriate design or maximum displacement value will result in the total displacement of the isolation system. Total displacement can be a value less than that computed if the isolation system is designed and calculated to resist such torsional displacement.

2.2.7 Displacement Restraints

Displacement restraints are permitted to be utilized, however, they must not engage at a distance less than 0.75 times the total design displacement. If restraints are activated prior to this amount, it must be adequately demonstrated that the isolators will still provide satisfactory performance and is not hindered by their initiation. Restraints can help to limit isolator displacement corresponding to overturning and therefore reduce or eliminate any possible tensile forces being induced at the isolation interface. This limitation assures that the displacement restraint devices only engage at the upper levels of the ground motions that produce the total design displacements. Additionally, these devices can be used to provide resistance to forces that may exceed those used to determine the total design displacement or earthquake ground motions which may exceed the forces for which the base isolation system was designed to withstand.

2.2.8 Isolation Restoring Forces

The isolation system should be able to provide restoring forces to bring the isolation device back to its original position prior to engagement. The isolation restoring force should be determined from the lateral forces induced on the structure. ASCE 7 states that an isolation unit should be capable of producing a value 0.025 multiplied by the seismic weight, greater than the lateral force at 50 percent of the total design displacement, in order to restore the isolator to its original intended position.

2.2.9 Isolation Base Shear

After the total isolation system displacements and the effective periods are computed the lateral forces applied to the structure can be calculated. All elements below the isolation interface, to include the isolators, must be able to resist a minimum base shear, V_b. The minimum base shear for all elements below the isolation interface is equivalent to the product of the maximum effective stiffness, k_{Dmax}, and the design displacement, D_D. Equation 2.9 (ASCE 7 equation 17.57) indicates this relationship. However, ASCE 7 requires the base shear to not be any less than the maximum force induced into the isolation system which produces the corresponding design displacement for an adequately designed isolation device.

$$V_b = K_{Dmax}\ D_D \tag{2.9}$$

2.2.10 Superstructure Base Shear

Once the isolation base shear is known, the shear value induced into the superstructure above the isolation interface is determined. The superstructure must be able to resist the magnitude of shear produced by Equation 2.10 (ASCE 7 equation 17.5-8). This force is computed based on the maximum effective stiffness of the isolation system k_{Dmax}, the design displacement D_D, and the numerical response modification coefficient, R_I.

The response modification coefficient, R_I, for an isolated structure is similar to the response modification coefficient, R, for fixed base structures. ASCE 7 indicates however, that the seismic isolation response modification coefficient shall be the lower value of 2.0 or 3/8 of the response modification coefficient as defined for a fixed base structure. However, R_I, shall be no less than 1.0. The limitations on this value assure that the structure remains essentially elastic during a seismic event.

$$V_s = \frac{k_{Dmax} D_D}{R_1} \tag{2.10}$$

Where:

R_I = numerical response modification coefficient related to the type of seismic force-resisting system of the superstructure

2.2.11 Superstructure Base Shear Limitations

ASCE 7 additionally requires that the value of V_s be checked so that it is no less than the following conditions

1.) The lateral force computed for a fixed base structure with the same seismic weight, W, and isolation period, T_D.
2.) The base shear produced by the factored design wind load.
3.) The lateral force to fully activate the isolation system multiplied by a factor of 1.5.

The limit of condition 1 is to ensure that the structure is designed for forces no less than those induced into the structure when it acts similarly to a fixed base structure for dynamic modes beyond the isolation mode. The second condition is to guarantee that the superstructure is designed to resist loads no less than the design wind loads which are not of significant magnitude to activate the isolation system. Furthermore, the third condition is to provide a minimum design force which may occur in the superstructure when the isolation system is just initially activated and as a safety factor in case there is a delay of activation in the isolation system.

2.2.12 Story Forces

Once the minimum limits for the base shear in the superstructure is determined, the story forces, F_x, can be calculated. The story forces for an isolated superstructure are computed using Equation 2.11 (ASCE 7 equation 17.5-9). This equation is similar to the equation used to determine the story forces for a fixed base structure. The base shear, V_s, is distributed vertically along the height of the structure based on the height of the story under consideration and the weight of the corresponding level.

Story forces are computed for every story level above the base of the structure and elements are designed to appropriately withstand such forces.

$$F_x = V_s \frac{h_x\, w_x}{\sum_{i=1}^{n} w_i\ h_i} \tag{2.11}$$

2.2.13 Drift Limits

The last element to be determined is the drift limits for the structure. These limits are determined by ASCE 7 and are based on the type of analysis being performed. Drift limits for isolated structures differ from those placed on fixed base buildings. Maximum story drift of the structure should not exceed the product of 0.015 and the height of the story level above the isolation system. Additionally, drift values should be calculated using the same equation for non-isolated structures; however, the deflection amplification factor, C_d, should be replaced with R_I, the seismic response modification factor. It is required that the isolation system have a wind restraint system at the isolation interface to limit the lateral displacement in the isolators to a value equal to the drift limit allowed between the superstructure levels.

2.3 Dynamic Analysis

As previously stated, structural designers also have the option, if not otherwise required, to use a dynamic analysis to design an isolation system and corresponding superstructure. Two types of dynamic analyses are used; the response spectrum analysis and the response history analysis. Both methods are better representative of the true seismic response of an isolated structure than the ELFP because they incorporate the use of engineering software and utilize time history earthquake data. Dynamic analysis is encouraged through increases or reductions in design requirements which results in less stringent demands on the structural systems.

2.3.1 Response History Analysis

The first type of dynamic analysis is the response history procedure. Using the response history procedure can lead to significantly more accurate results than those obtained from the response spectrum and ELFP. Similar to the response spectrum analysis, the response history procedure is used to determine the total and maximum displacements, and the base shear values for the isolation system. Additionally, unlike the response spectrum analysis, the response history procedure can be used to determine the superstructure story force distribution. Base shear values at the isolation interface can be reduced to as low as 60 percent of the base shear for a regular building as obtained from the ELFP by use of the response history procedure.

For irregular structures the base shear value of the superstructure can only be as low as 80 percent of the ELFP computed base shear. Irregular structures are more complex and their seismic response is much more difficult to predict even through the use of software programs and dynamic analysis procedures and therefore, are not allowed by ASCE 7 to be much less than the ELFP design values.

To perform a response history procedure a minimum of three relative ground motions are required. Ground motions should be chosen so they accurately represent the site properties under consideration. Time histories are chosen and scaled from individual recorded seismic events. Earthquake ground motions should be based on elements and factors that control the maximum credible earthquake (MCE) to be considered appropriate for use in designs. Those factors include earthquake magnitude, distance from the fault line or seismic region, and specific type of fault rupture. Seismic ground motions are composed of pairs of values commonly measured in directions perpendicular to one another.

Structural building codes, such as ASCE 7, allow for the use of simulated ground motions when recorded ground motions are not available in specific site locations. Selected ground motions should be scaled so that the average of the square root of the sum of the squares (SRSS) spectra for all horizontal ground motion pairs does not drop below the product of 1.3 times the design response spectrum obtained from fixed base procedures by more than 10 percent for each period within the range of $0.5T_D$ and

$1.25T_M$. Additionally, the structure is to be modeled with an eccentricity which results in the most disadvantageous structural response. For all ground motion records the required design values, for example the base shear and displacement values for the isolation system and superstructure, should be computed and analyzed.

It is permissible through ASCE 7 to analyze in excess of three ground motion records. When the number of ground motion records analyzed is at least seven, the average of the response results is permitted to be used for design of the superstructure. However, when less than seven ground motion records are utilized, design values must be computed using the maximum overall response values. It is of substantial benefit to utilize various ground motion records to obtain more accurate results, which correspondingly results in lower design value.

2.3.2 Response Spectrum Analysis

The response spectrum analysis can be used to determine the total design and maximum displacements of the superstructure and isolation devices along with the base shear at the isolation level and isolation interface. This design procedure is best used to approximate the non-linear response of a complex structural system such as base isolated buildings. ASCE 7 allows the base shear as determined by a response spectrum analysis to be no less than 80 percent of the base shear computed using the ELFP for structures that are regular in configuration.

Also, base shear values for structures that are irregular in configuration must not be less than the value obtained through the ELFP, even if the analysis results provides a lower value. This indicates that for irregular structure which has a complex floor layout and vertical off-sets, a response spectrum analysis does not provide results as accurate as if it were a regular structure and designers may want to consider performing a response history analysis. Once the base shear has been obtained it can be used to determine the story forces on the structure.

Despite a more accurate design procedure and a reduction in the base shear forces, the vertical distribution of forces must be computed using Equation 2.11, from the equivalent lateral force procedure which distributes the forces linearly along the height of the superstructure. Linearly distributed vertical forces are required to account for higher mode participation that may occur in the event of a long earthquake which is not taken into account in other design equations. The nonlinearity of the isolation system causes the structure to have higher mode responses (Ryan and York 2007). Structural base isolation theory however, suggests that story forces should be uniformly distributed vertically at story levels rather than in a linear method. Uniform distribution would result in story forces which are essentially the same at each level.

For triangular load distributions, forces are greater with respect to the height above the base of the greatest height of the structure along with the approximate period parameters, C_r and x which are based on the type of structure under consideration. Therefore, the structures seismic weight is not a factor directly affecting the approximate fundamental period for a fixed-base structure as is the case with an isolated structure. The stiffness based on the type of structure is replaced with the stiffness of the isolation system when computing the period of a seismically isolated structure. It can also be observed that the height of an isolated superstructure has no affect on its effective period, indicating that the period of the structure is based on the properties of the supporting isolation system.

The design displacement and maximum displacement values include the affects of accidental torsion, which considers the stiffness distribution and affects of eccentricity within the structure. ASCE 7 formulates the total displacement equations for ease of structure and the weight at the story level under consideration.

This relationship can be seen in Equation 2.11.Isolated structures should also be capable of resisting seismic forces in all directions. To ensure that the structure is adequate in this aspect, the response spectrum procedure should use the combination of 100 percent of the seismic forces induced in one horizontal direction and 30 percent of the seismic forces induced in the perpendicular direction to that of the axis being considered. This results in demands similar to those using the SRSS. This requirement will ensure that the structure is capable of resisting forces in all directions other than just the orthogonal directions.

2.3.3 Effective Fundamental Period

The effective structural period is computed based on the effective seismic weight (which is similar to fixed base structures) the acceleration due to gravity, and the minimum effective stiffness of the isolation system. The effective stiffness is determined for the design displacement, k_D, and the maximum displacement, k_M, of the isolation system. Specific design parameters corresponding to the isolation system and superstructure being considered should be used to compute these values. Fixed base structures have an approximate fundamental period that is computed based on the parameters associated with the type of structure and the height of the building above the base of the building.

Equation 2.3 and 2.4 indicates that the effective period for an isolated building is based on the weight of the structure along with the acceleration of gravity. The approximate fundamental period for a fixed-base structure is calculated based on the computing and values less than determined by these equations are acceptable but must be no less than 1.1 multiplied by the design or maximum computed displacement value.

2.3.4 Isolator Displacement

Seismically isolated structures require the computation of the minimum lateral design earthquake displacement (D_D), maximum isolation system displacement (D_M), total maximum displacement (D_{TM}) and the effective period at the design displacement (T_D) and at the maximum displacement (T_M). Minimum lateral earthquake displacement is computed along each main horizontal building axis while the maximum displacement of the isolation system is computed based on the direction of the building that is the most critical. The most critical direction is the one that produces the worst case values of displacement. Displacement values are computed as a function of the acceleration of gravity, effective period of an isolated structure, system damping coefficient, and damped spectral acceleration values. The three displacements that must be considered when designing base isolated structures are illustrated in Figure 2.5.

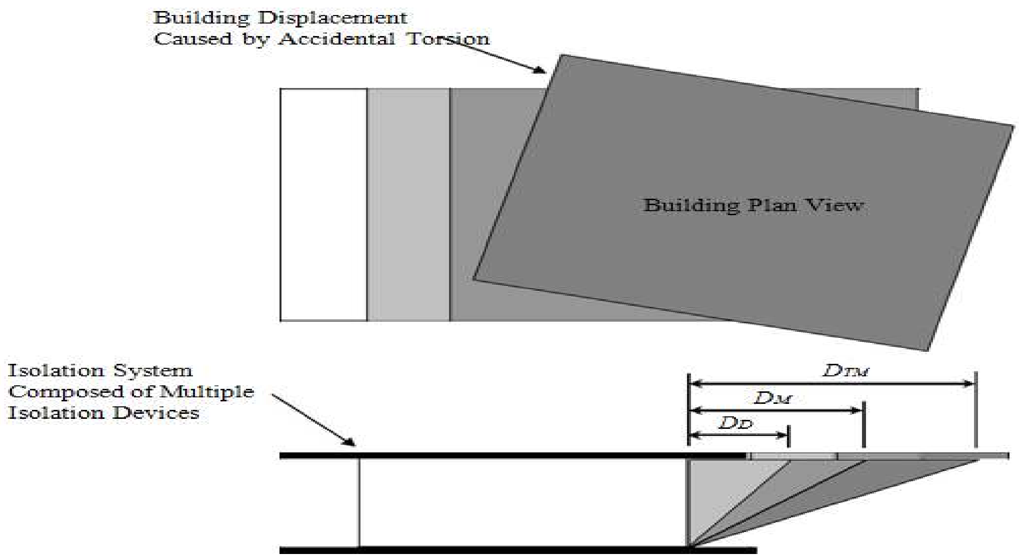

Figure 2.5 Maximum and Minimum Design Displacements for an Isolation System. Figure Reproduced from NEHRP (Cardone et al.,2010)

2.3.5 Story Deflections and Drift

Regardless of the design procedure applied, deflections at each story level are computed in a similar manner to that for a non-isolated building except that the deflection amplification factor, C_d, is replaced with the numerical coefficient, R_I. The numerical coefficient, R_I, is related to the type of seismic force resisting system used for that of a seismically isolated superstructure. By using the response spectrum analysis, story drifts, including any drift induced by vertical deformations of the isolation system, are limited to a distance of 0.015 times the height of the story level under consideration

When using the response history analysis, story drifts are less demanding and are only limited to a distance of 0.020 times the height of the story level under consideration. Also, inter story drifting, the total elastic displacement of the structure under factored, strength-level design earthquake forces, should be computed or obtained from the analysis. Inter story drift limits are more severe for base isolated structures than for non-isolated structures. The deflection amplification factor for a fixed base structure can typically range anywhere from 1.5 to 5.5 and the numerical coefficient that replaces this value for a seismically isolated structure ranges from 1.0 to 2.0. As a result of this, the limiting inter story drift values obtained by Equation 2.12 (ASCE 7 equation 12.8-15) may be much lower and more difficult to design for than drift values for a similar structure with a fixed base.

$$\delta_x = \frac{C_d \ \delta_{xe}}{I} \tag{2.12}$$

Where:
C_d = deflection amplification factor
δ_{xe} = deflections determined by an elastic analysis
I = occupancy importance factor, taken as 1.0 for base isolated structures

The less drifting that is allowed to occur between story levels, the less overturning forces that will be applied on the isolation system. As a result, connections at the isolation interface will be less difficult to establish.

These rigorous requirements also minimize the amount of tension strain in the isolator mechanisms which could contribute to undesirable isolator performance. The minimum lateral earthquake displacement uses the design spectral acceleration parameters at a 1-second period.

Maximum displacements are however computed using the maximum creditable earthquake (MCE) spectral response acceleration at a period of 1-second. A MCE is defined as the largest earthquake that can be expected to occur in a specific region, with peak ground accelerations corresponding to a 2 percent chance of being exceeded in 50 years. The MCE response acceleration is adjusted based on the site class effects as indicated by the geotechnical soil reports.

2.3.6 Elements Located Below the Isolation Interface

At and below the isolation interface all structural and non-structural elements must be designed for the appropriate base shear, or seismic lateral forces appropriate for the region where the structure is to be built. In a fixed base structure, the seismic base shear is a factor of the effective weight of the building and the seismic response coefficient. Seismic response coefficients are a function of the response modification factor.

The response modification factor depends on the strength and type of lateral force system used in the building, the occupancy importance factor, and the spectral response acceleration values. The occupancy importance factor for all isolated structure types and uses is equal to unity. This is because base isolation satisfies a design criterion well above that of the basic code minimum standards. For a seismically isolated building, the base shear for the isolation system and the structural elements below the isolation interface is equal to the product of the isolation systems maximum effective horizontal stiffness and the isolators design displacement.

2.3.7 Elements above the Isolation Interface

Any part of the structure above the isolation system must be designed for the base shear of the isolators divided by a coefficient representing the type of seismic force-resisting system used in the superstructure. This coefficient is similar to the response modification factor for traditional fixed base structures. Fixed base buildings can have a response modification normally in the range of 5 to 8.

A common opinion of base isolation is that it reduces the design base shear in the superstructure. The coefficient for isolated buildings is equal to 3/8th of the modification factor for a building with a fixed base, which seems as though it would be reducing the base shear. However, ASCE 7 indicates that the coefficient shall not be greater than 2.0 or less than 1.0. A structure having a response modification factor of 1.0 represents an elastic structure. This essentially results in a maximum superstructure base shear equal to the exact base shear of that for the isolation system and not less than half of the base shear at the isolation interface.

2.3.8 Seismic Base Shear

Structural building codes, such as ASCE 7, indicate that the base shear should not be less than the lateral seismic forces for a fixed base structure with the same seismic weight computed using the isolated period of vibration. Additionally, the base shear should not be less than the horizontal design wind loads, typically this will not govern for a seismically isolated structure. Minimum base shear forces should be greater than the lateral force that it takes to activate the isolators multiplied by an arbitrary factor of 1.5.

These requirements are enforced for several reasons. First, the superstructure must be able to withstand the horizontal wind forces applied to the structure. With respect to the lateral wind loads, an isolated building will react similar to a fixed base structure. The isolation system is not designed to engage in response to small amounts of lateral forces. This keeps the building from constantly moving under non-hazardous typical wind loads

The maximum wind load does not necessarily correspond to the magnitude of isolator engagement. Therefore, the difference in force levels up to the engagement magnitude must be resisted through the superstructure as if it were a fixed base structure. The lower bound engagement magnitude is represented by product of the isolator activation load multiplied by a factor of 1.5. Isolated superstructures must be designed for forces no less than the same structure designed as a fixed base where properties of the isolated structure only minimally reduce the values of base shear.

The common conceptions about base isolation under the most current building codes are slightly misleading. Strict design equations have been determined as acceptable design procedures for isolated buildings without utilizing the full capacity and intent of the isolation devices. This may change with time, testing, and application within the next few building code cycles. Such stringent design equations may be in part from the lack of verification for current buildings that employ base isolation devices. Base isolations true response to earthquake forces is still not completely understood. There is also a lack of data publications for isolated buildings that have been activated during an earthquake; reasons for this seem generally unknown.

2.3.9 Structural Elasticity and Ductility

Base isolated superstructures are required to be designed assuming the building remains elastic during what is considered the maximum considered earthquake for a particular region (Sattary et al., 1993). Designing for an essentially elastic structure results in little to no relationship between the forces used to design the isolation system and the superstructure. Superstructure forces are reduced by the response modification factor, R_I, which ranges between 1.0 and 2.0 as determined by the requirements of ASCE 7.

The isolation system is designed using the design basis earthquake level (DBE) values while the systems stability is based on the maximum considered earthquake (MCE) values. The MCE has a 2 percent probability of being exceeded within 50 years, or a 975 year return period in high to moderate seismic regions and 2500-year return period in low seismic regions (Clark et al., 1993). The DBE however, is based on a 10 percent probability of being exceeded within a 50 year time frame or a 475-year return period.

Building codes allow buildings to be seismically designed for the DBE since providing designs for MCE values would be costly and inefficient for those buildings with low occupancy importance factors. Additionally, the purpose for designing for the DBE is primarily to ensure life safety of the occupants and prevent instantaneous structural collapse. Structural ductility is less significant in structures that are base isolated because less seismic forces are induced into the structure above the isolation interface. Isolation devices are designed so that the initial response to ground movement is significantly small. As the forces at the ground level tend to increase with magnitude, the isolators' stiffness decreases proportionally (Sattary et al., 1993).

2.4 Isolator Design Procedures

Basic procedures for design of the high damping and low damping rubber isolators (HDR, LDR), lead-rubber isolators (LRB), and the friction pendulum isolators (FPS) are presented in this section. The primary purpose of this information is to aid design engineer in preliminary sizing of the isolators needed for a given project. For is information The reader is encouraged to read the recent textbook by Kelly (1996) for a very detailed coverage of mechanical characteristics and modeling of HDR and LRB isolators. A less exhaustive but more practical coverage of the same topics may be found in a recent textbook by Naeim and Kelly (1999). Further instructions and details for design of FPS isolators may be obtained from the patent-holder, Earthquake Protection Systems of Berkeley, California and from Reference (zayas et al .,1987) The need for an isolation system which is stiff under low levels of lateral load (e.g. wind) but flexible under higher levels (i.e. earthquakes) necessarily leads to a nonlinear system. The properties of most isolator systems are characterized as bilinear. Although a tri-linear model with stiffening at large horizontal displacements better represents the performance of HDR isolators. Any complete design procedure should insure that (i) the bearings will safely support the maximum gravity service loads throughout the life of the structure and (ii) the bearings will provide a period shift and hysteric damping during one or more design earthquakes. The steps to achieve these aims are:

1. The minimum required plan size is determined for the maximum gravity loads at each bearing location.
2. The total rubber thickness or dimensions of the FPS isolator is computed to give the period shift during earthquake loadings.
3. The damping characteristics of the isolator system is calculated to ensure proper value of the hysteric damping and wind resistance required.
4. The performance of the bearings as designed is checked under gravity, wind, thermal, earthquake, and any other load conditions.

2.4.1 Elastomeric Isolators

One of the most important parameters in design of elastomeric bearings is the shape factor, *S*, defined as

$$S = \frac{\text{loaded area}}{\mathit{Force-free\ area}}$$

For a circular pad with a diameter of Φ and a single layer rubber thickness, *t*

$$S = \frac{\Phi}{4t} \tag{2.13}$$

Generally a good design tries to keep the value of *S* to somewhere between 10 and 20.

The horizontal stiffness of a single isolator is given by

$$K_H = \frac{GA}{t_r} \tag{2.14}$$

where *G* is the shear modulus of the rubber, *A* is the full cross-sectional area of the pad, and t_r is the total thickness of rubber.

The maximum shear strain, γ, experienced by the isolator is the maximum horizontal displacement, *D*, divided by the total rubber thickness, t_r.

$$\gamma = \frac{D}{t_r} \tag{2.15}$$

The vertical stiffness of a rubber bearing is given by

$$K = \frac{E_c A_s}{t_r} \tag{2.16}$$

where E_c is the compression modulus of the rubber-steel composite and A_s is the area of a steel shim plate. For a circular pad without any holes in the center

$$E_c = 6GS^2 \tag{2.17}$$

For bearings with very large shape factors the compressibility of rubber affects value of E_c. In such cases a more accurate estimate of E_c may be obtained from

$$E_c = \frac{6GS^2\, k}{6Gs^2 + K} \tag{2.18}$$

where K is the bulk modulus of rubber and generally varies from 145,000 psi to 360,000 psi

depending on the type of rubber being used. The value of 290,000 psi is most commonly used

2.4.2 Lead-Rubber Isolators (LRB)

The lead-rubber bearings is a nonlinear system which may be very effectively idealized in terms of a bilinear force—deflection curve with constant values throughout many cycles of loading (Figure 10). Formulas developed in the previous section are also applicable here with some additional equations that model the lead core properties.

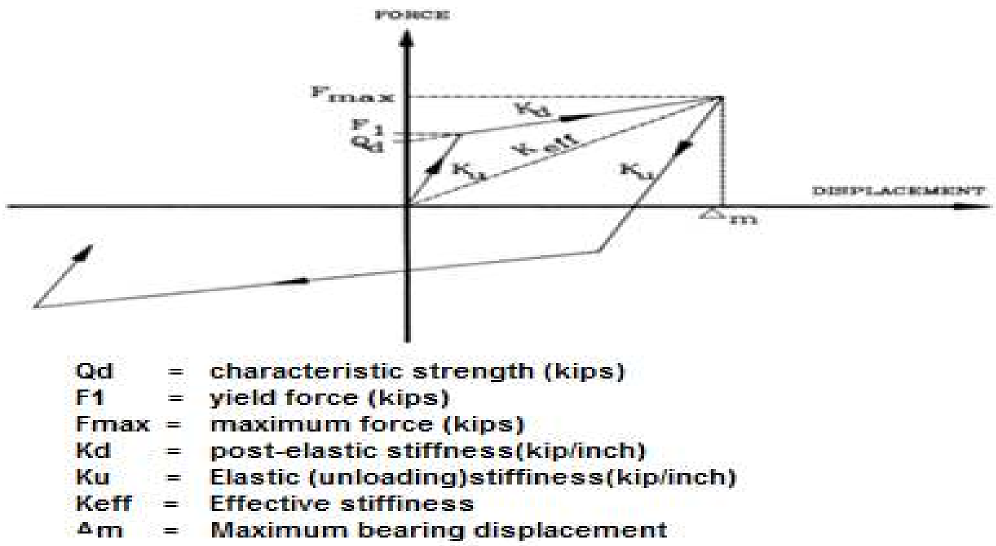

Figure 2.6. Typical linear hysteresis loop(Kelly& Naeim,1999)

The characteristic strength, Q_d, can be accurately estimated as being equal to the yield force of the lead plug. The yield stress of lead is about 1,500 psi. The effective stiffness of the lead-plug bearing, K_{eff}, at a horizontal displacement D larger than the yield displacement D_y, may be defined in terms of the post-elastic stiffness, K_d, and characteristic strength, Q_d, as

$$K_{eff} = K_d + \frac{Q_d}{D} \qquad D \geq D_y \tag{2.19}$$

the natural period is given as

$$\mathrm{T} = 2\pi\sqrt{\frac{\mathrm{w}}{\mathrm{k_{eff}}\ \mathrm{g}}} \tag{2.20}$$

As a rule of thumb for lead-rubber isolators K_u is taken as $10K_d$. Kelly has shown that with this

assumption, the effective percentage of critical damping provided by the isolator, β_{eff}, can be obtained from

$$\beta_{\mathrm{eff}} = \frac{4\mathrm{Q}\left(\mathrm{D} - {}^{\mathrm{Q_d}}/_{9\mathrm{K_u}}\right)}{2\pi(\mathrm{K_u}\mathrm{D} + \mathrm{Q_d})\mathrm{D}} \tag{2.21}$$

2.4.3 Friction Pendulum System

If the load on an FPS isolator is W, and the radius of curvature of the FPS dish is R, then the horizontal stiffness of the isolator may be defined for design purposes as

$$K_H = \frac{W}{R} \tag{2.22}$$

The natural period of and FPS isolated system is only a function of R

$$T = 2\pi\sqrt{\frac{R}{g}} \tag{2.23}$$

The effective (peak-to-peak) stiffness of the isolator is given by

$$\mathrm{k_{eff}} = \frac{\mathrm{W}}{\mathrm{R}} + \frac{\mu\mathrm{W}}{\mathrm{D}} \qquad (2.24)$$

where $\boldsymbol{\mu}$ is the friction coefficient and all other terms are defined previously. The friction coefficient has been shown to be independent of velocity for pressures of 20 ksi or more on the articulated slider. The damping provided by the system, β, is a function of horizontal displacement and may be obtained from

$$\beta = \frac{2\mu}{\pi\mu + {}^{\mathrm{D}}/_{\mathrm{R}}} \qquad (2.25)$$

An estimate of the rise of the structure (vertical displacement) as a result of movement along the curved surface of the isolator may be obtained from

$$\delta \cong \frac{1}{2}\frac{D^2}{R} \qquad (2.26)$$

2.5 Equivalent Linear Model of Isolators

The non-linear force-deformation characteristic of the isolator can be replaced by an equivalent linear model through effective elastic stiffness and effective viscous damping. The equivalent linear elastic stiffness for each cycle of loading is calculated from experimentally obtained force-deformation curve of the isolator and expressed mathematically as “International Building Code”, IBC2006

$$K_{eff} = \frac{(F^{+} - F^{+})}{(\Delta^{+} - \Delta^{+})} \qquad (2.27)$$

where F+ and F – are the positive and negative forces at test displacements Δ+andΔ–respectively. Thus, the K_{eff} is the slope of the peak-to-peak values of the hysteresis loop as shown in Figure 2.7

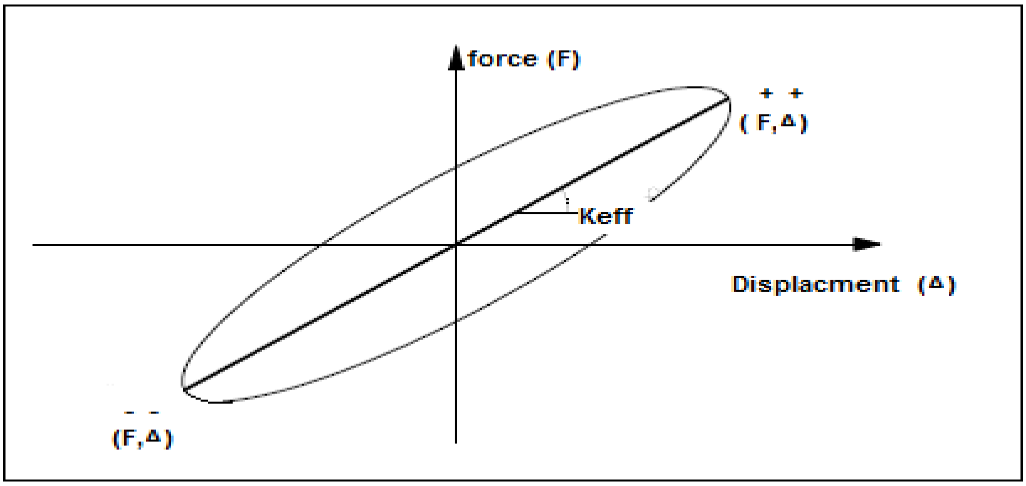

Figure 2.7 Force displacement relationship of equivalent linear model (Vasant et al. 2004)

The effective viscous damping ratio of the isolator calculated for each cycle of loading is expressed as “International Building Code”, IBC 2006

$$\beta_{eff} = \frac{(2E_{loop})}{[\pi\, K_{eff}\, (|\Delta^{+}|\, |\Delta^{+}|)^{2}]} \tag{2.28}$$

where E loop is the energy dissipation per cycle of loading

2.6 Isolation Design Analysis Software

Isolated structures commonly require the use of non-linear time history analysis software. The types of software available today are becoming useful tools for such designs. However, during the early stages of the development of base isolation, software with such capabilities was rare. Traditional fixed base structures do not require such analysis tools since only a linear analysis is necessary for design. Regularly configured isolated structures also do not require a dynamic, non-linear history analysis. In fact, the superstructure of an isolation structure also only requires a linear analysis rather than a non-linear analysis. The level of difficulty in design varies based on the type of analysis performed. In many instances, design analysis software can help to simplify processes; however, there may be a few cases in which the difficulty may actually increase with the use of such software. In this research , a commercial computer program SAP 2000 has been used to design the structure system .

2.7 Isolator Placement and Location

Locating isolators within a building is a rather simplistic concept. Commonly, isolators are placed beneath each column supporting the superstructure. Ultimately, it is ideal to place isolators in a region which they will receive the greatest vertical weight. This insures that the axial load from the superstructure helps maintain isolator balance without concern for overturning. Isolators are then placed at a level of the building that is below, or at the level of exterior grade or placed at a basement sub grade level. The quantity of isolators used in a building can vary significantly depending on the size of the building, the number of columns supporting the structure, and the weight of the building (Buckle and Liu 1993). It is ideal to have isolators that are all of the same type. However, it is possible to have various types of isolators under the same structure.

If different isolators are desired and incorporated into a single isolated structure they must be rigidly attached to a common interface, or diaphragm. A common interface allows the compatibility of structural displacements amongst the different isolator reactions (Buckle and Liu 1993). Figure 2. 8 illustrates various methods of vertical isolator placement.

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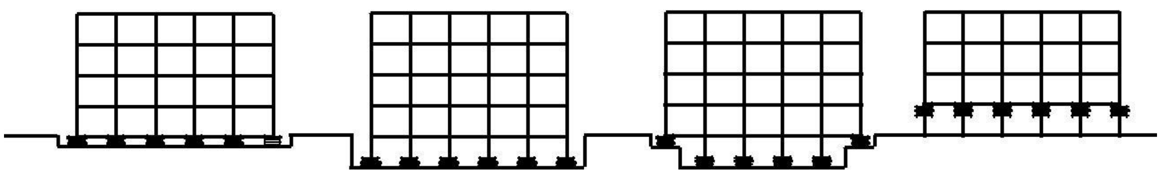

Figure 2.8 Vertical Location and Positioning of Isolation Devices.(Graham 1994).

CHAPTER 3

CASE STUDIES

3.1 Introduction

The aim of the case studies is to investigate the performance features and response mechanisms of base isolated structures for IBC2006 specifications. Major characteristics of seismic isolation systems are introduced and studied in the previous chapters. Four different types of buildings are used in the analyses. The isolated buildings are analyzed by using Static Equivalent Lateral Force, Response Spectrum and Time History methods. A commercial computer package, namely SAP2000, is used for 3D analysis of the structures.

3.2 Description of the Structures

The structures, used for the analyses, are assumed to be serving as school buildings. The detailed descriptions of the buildings are as follows:

3.2.1 three-Storey Symmetrical Building (Type-I)

The three-storey building has a regular plan (33m x 13m) as shown in Figure 3.1. The section view is shown in Figure 3.2. The structural system is selected as concrete frames with identical columns of 55/55 centimeters in size, and beams of dimension 30/70 centimeters. Each floor slab has 16 centimeters thickness and the story height is 3 meters. The critical damping ratio of superstructure is taken as 2% for isolated cases.

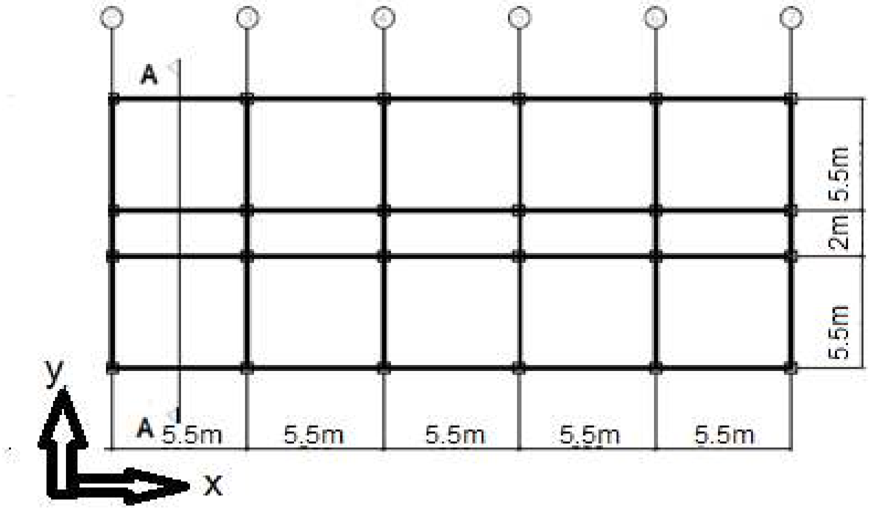

Figure 3.1 Plan view of symmetrical building types.

24 units High Damping Rubber bearings (HDR) are used for the isolation of the building. The detailed calculations of isolation system design are explained in Section 4.1. The bearings have the following linear properties accordingly:

βi = 0.15 (isolator damping ratio)

G = 550 kN/m^2

K_h = 417 kN/m

K_v = 200000 kN/m

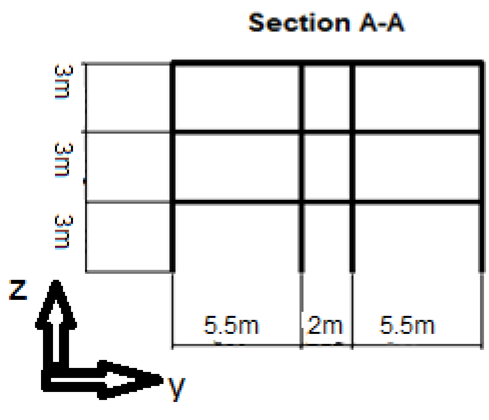

Figure 3.2 Section view of building Type-I

3.2.2 five-Storey Symmetrical Building (Type-II)

The five-storey building has a regular plan (33m x 13m) as shown in Figure 3.1. The section view is shown in Figure 3.3. The structural system and seismic isolation system is identical with three-storey symmetrical building.

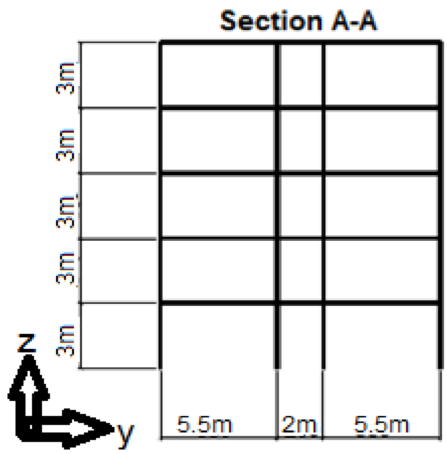

Figure 3.3 Section view of building Type-II

3.2.3Eight -Storey Symmetrical Building (Type-III)

The eight-storey building has a regular plan (36m x 13m) as shown in Figure 3.1. The section view is shown in Figure 3.4. The structural system and seismic isolation system is identical with three-storey symmetrical building.

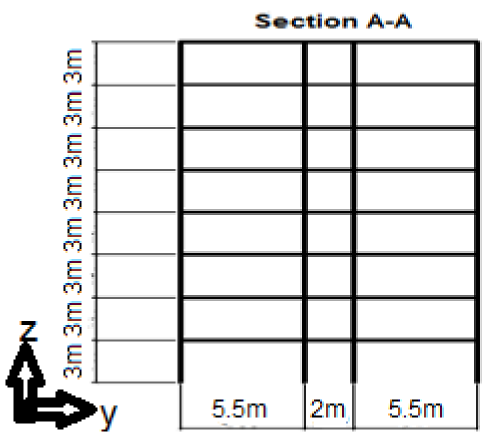

Figure 3.4 Section view of building Type-III

3.2.4 Non-symmetrical Building (Type-IV)

The plan view of five storey non-symmetrical building is shown in Figure 3.5 below. The section view is shown in Figure 3.6. The structural system is selected as concrete frames with identical columns of 55/55 centimeters in size, and beams of dimension 30/70 centimeters. Each floor slab has 16 centimeters thickness and the story height is 3 meters. The critical damping ratio of superstructure is taken as 2% for isolated cases.

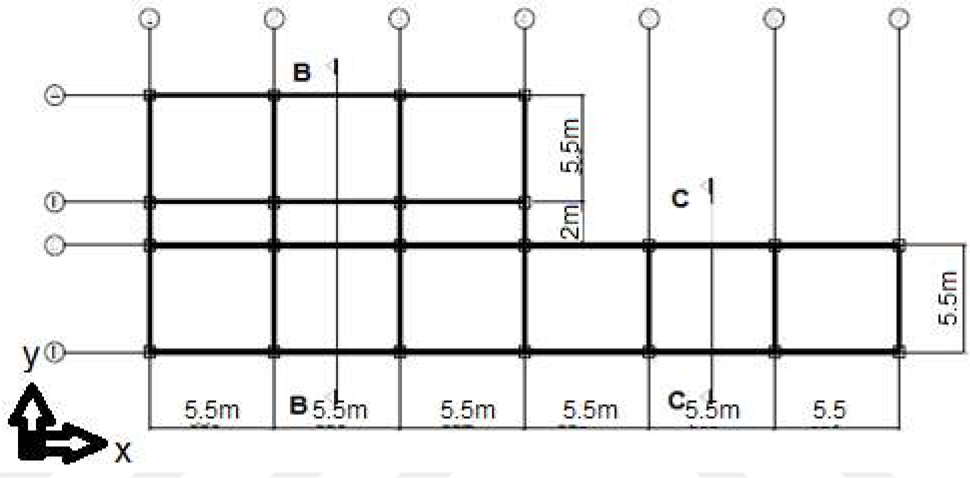

Figure 3.5 Plan view of building Type-IV

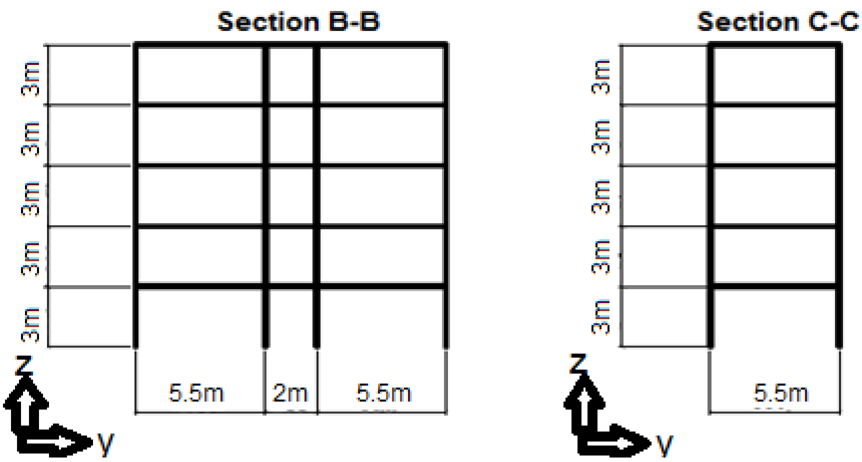

Figure 3.6 Section views of building Type-IV

20 units High Damping Rubber bearings (HDR) are used for the seismic isolation of the building. The linear properties of the selected isolators are as follows:

βi = 0.15 (isolator damping ratio)

G = 550 kN/m^2

K_h = 417 kN/m

K_v = 200000 kN/m

3.3 Analysis Methods

In this section, static equivalent lateral force procedure, response spectrum analysis and time history analysis are discussed.

3.3.1 Static Equivalent Lateral Force Procedure

The isolation system should be designed to withstand minimum lateral earthquake displacements, D_D, that act in the direction of each of the main horizontal axes of the structure in accordance with the following (Kunde et al., 2003)

$$D_D = \left(\frac{g}{4\pi^2}\right)\frac{S_{D1}T_D}{B_D} \tag{3.1}$$

where:

B_D = Numerical coefficient related to the effective damping of the isolation system at design displacement(IBC2006), as set forth in Table 3.1

g = Acceleration of gravity

S_{D1} = Design 5% damped spectral acceleration at 1 sec. period

T_D = Isolated period at design displace

Table 3.1 Damping coefficient "International Building Code", IBC2006

EFFECTIVE DAMPING β_i	B_D
<2%	0.8
5%	1.0
10%	1.2
20%	1.5
30%	1.7
40%	1.9
>50%	2.0

For damping values other than the one specified in Table 3.1, linear interpolation can be done to find corresponding B_D value. Alternatively, a very close approximation to the table values is given by;

$$\frac{1}{B_D} = 0.25(1 - Ln\beta) \qquad (3.2)$$

The effective period of the isolated structure, T_D, is determined as:

$$T = 2\pi\sqrt{\frac{W}{k_{eff}\ g}} \qquad (3.3)$$

where

W is the total dead load weight of the superstructure.

The total design displacement, D_{TD}, of elements of the isolation system shall include additional displacement due to actual and accidental torsion calculated considering the spatial distribution of the lateral stiffness of the isolation system and the most disadvantageous location of mass eccentricity. D_{TD} must satisfy the following condition.(IBC2006).

$$D_{TD} = D_D\left(1 + y\left(\frac{12e}{b^2 + d^2}\right)\right) \quad (3.4)$$

where:

d = Shortest plan dimension

b = Longest plan dimension

e = The actual eccentricity measured in plan between the center of mass of the structure and the center of stiffness of the isolation system, plus the accidental eccentricity taken as 5% of the longest plan dimension of the structure perpendicular to the direction of seismic loading under consideration (Figure 3.7).

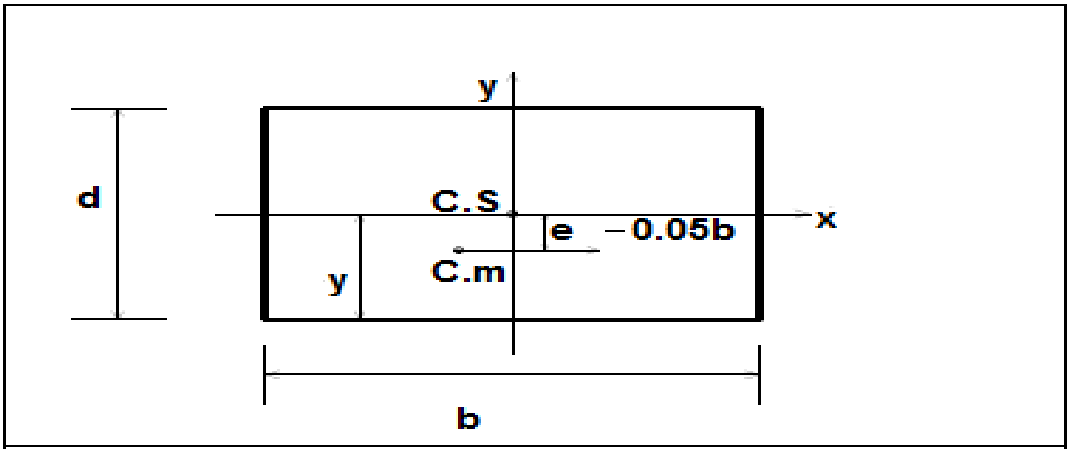

Figure 3.7 Plan dimensions for calculation of D_{TD} (Naeim and Kelly .1999)

The structure above the isolation system must be designed to withstand a minimum total shear force, V_S:

$$Vs = \frac{K_h D_D}{R} \quad (3.5)$$

where:

R = Seismic load reduction factor.

ASCE 7-05 assume R as if the structure goes into inelastic range ($1.0 \leq R < 2.0$). In this study R is assumed be equal to one, thus the lateral EQ force, applied to the building is not reduced. One can easily calculate the corresponding design values of R=2.0 if it is desired. The total shear force, V_S, is distributed over the height of the structure as given by:

$$F_x = \frac{Vs\ h_x\ w_x}{\sum_{i=1}^{n} w_i\ h_i} \tag{3.6}$$

where:

h_i = Height above the base to level i.

h_x = Height above the base to level x.

W_x = Portion of total dead load that is located at or assigned to level x.

W_i = Portion of total dead load that is located at or assigned to level i.

3.3.2 Response Spectrum Analysis

The 5% damped spectrum, given in IBC 2006, is used for the analysis of building and the spectrum is modified for each of the soil types (C). Spectrum is assumed to be acting on the building from both directions (X-Y) simultaneously. While the component applied from one axis is multiplied by 1.00 the orthogonal component is multiplied by 0.30. According to this logic, two different E.Q. combinations are applied to the structure and the results are examined for each case. In the results, the one, which causes the most critical condition, is taken into account.

3.3.3 Time History Analysis

Time history analysis shall be performed with at least three appropriate pairs of horizontal time history components. If three time history analyses are performed, the maximum response of the parameter of interest shall be used for design. If seven or more time history analyses are performed, the average value of the response parameter of interest shall be used for design. Each pair of histories shall be applied simultaneously to the model.(IBC 2006).

The used ground motions and their properties are given in Table 3.2 below:

Table 3.2 Ground motions used in time history analyses

	COYOTE LAKE EARTHQUAKE	LOMA PRIETA EARTHQUAKE	EL CENTRO EARTHQUAKE
Place	COYOTE LAKE U.S.A	LOMA PRIETA U.S.A	EL CENTRO U.S.A
Date	06.08.1979	17.10.1989	18.05.1940
Magnitude	5.7	7.1	6.9
Closest Distance to Fault Rupture	3.20	18.00	4.5
Number of Date points	5764	2000	5000
Time Interval	0.005	0.02	0.02

3.4 Design Codes

In this study the design procedure for isolated buildings is primarily based on International Building Code IBC 2006 and ASCE7-05. The applicable methods in codes for the design of an isolated building and necessary formulas of these methods are mentioned above. In addition, some important limiting regulations in the design procedure.

In the IBC 2006 and ASCE 7-05 it is stated that the conditions of site where the structure is located must be reflected into the design procedure. Consequently, the calculated response of the structure is modified. A coefficient called as scaling factors involved in design calculations for this purpose.

3.4.1 IBC 2006 and ASCE 7-05

The base shear, V_S, is taken as not less than the following threshold values for static equivalent lateral force procedure:

_The lateral seismic force required for a fixed-base structure of the same weight and a period equal to the isolated period.

_The base shear corresponding to the factored design wind load.

_The lateral seismic load required to exceed the isolation system (the yield level of a softening system, the breakaway friction level of a sliding system factored by 1.5).

When the factored base shear force on structural elements, determined using either response spectrum or time history analysis, is less than the minimum prescribed below, response parameters, including member forces and moments, is adjusted proportionally upward.

_The total design displacement of isolation system is taken as not less than 90% of D_{TD} as specified in Equation 3.4.

_The design base shear force on the structure above the isolation system, if regular in configuration, is taken as not less than 80% of V_S as prescribed by Equation 3.5.

In codes there are two exceptions for the above conditions:

1. The design base shear force on the structure above the isolation system, if regular in configuration, is permitted to be taken as less than 80%, but not less than 60% of V_S, provided time history analysis is used for design of the structure.
The design base shear force on the structure above the isolation system, if irregular in configuration, must be taken as not less than V_S.
2. The design base shear force on the structure above the isolation system, if irregular in configuration, is permitted to be taken as less than 100%, but not less than 80% of V_S, provided time history analysis is used for design of the structure. The scaling limits on displacements specified above must be evaluated using values of D_{TD} determined in accordance with Equation 3.4 except that $D_{D'}$ is permitted to bused instead of D_D where $D_{D'}$ is prescribed by the following formula:

$$D_D' = \frac{D_D}{\sqrt{1 + \left(\frac{T}{T_D}\right)^2}} \tag{3.7}$$

where:

T = Elastic, fixed-based period of the structure above the isolation system.

T_D = Effective period, in seconds, of the seismically isolated structure at the design displacement in the direction under consideration as prescribed by Equation 3.3.

Table 3.3 Scaling limits for IBC2006

		If Response Spectrum is used	If Time History Analysis is used
Regular	$V_{analysis}$	≥ 0.8 Vs	≥ 0.6 Vs
Regular	$D_{analysis}$	≥ 0.9 D_{TD}	≥ 0.9D_{TD}
Irregular	$V_{analysis}$	≥ 1.0 Vs	≥ 0.8 Vs
Irregular	$D_{analysis}$	≥ 0.9 D_{TD}	≥ 0.9D_{TD}

CHAPTER 4

ISOLATION SYSTEM DESIGN

4.1 Design of High Damping Rubber Bearing

As an example of isolation system design, the symmetrical building, introduced in Section 3.2.2, is considered. The designed HDR bearings are used also for the other building types analyzed in the case studies.

Twenty-eight HDR bearings are used for the isolation of the building. The selected strategy for the design is to use one type of bearing for the whole system. The basic structural data to be used for the design is as follows:

$T_D = 2.1$ sec. (Target period for 'Design Level' earthquake)

$T_M = 2.5$ sec. (Target period for 'Max. Capable Level' earthquake)

$R = 1$ (Seismic load reduction factor)

$G = 550$ kN/m^2 (For large shear strains)

$G = 650$ kN/m^2 (For small shear strains)

$K = 2{,}000{,}000$ kN/m^2 (Bulk modulus)

$\beta_i = 15\%$ (Damping ratio of isolator)

$W_T = 14068$ kN (Total weight of the structure)

$\gamma_{max} = 150\%$

The site is assumed to be located in seismic zone 1 with C soil type.

Naeim and Kelly,(1999) represented the seismic faults are grouped into three categories based on the seriousness of the hazard. Faults capable of producing large magnitude earthquakes (M>7.0) and have a high rate of seismic activity (annual average seismic slip rate, SR, of 5 mm or more) are classified as type A sources. Faults capable of producing moderate magnitude earthquakes (M<6.5) with a relatively low rate of seismic activity (SR<2 mm) are classified as type C sources. As a result of these conditions S_{D1} is equal to 0.65 shown in appendix A .

4.2 Performance Comparison of Isolation Systems

The design of a seismic isolation system involves many interrelated variables such as targeted period of the isolated structure, base shear and damping of isolation system. To illustrate an overview of how these variables affect the structural design, the IBC2006 requirements for isolated buildings is used.

In the code, the displacement across the isolators (D_D), the isolation period (T_D) and the base shear (V_S) are given by Equations 3.1, 3.3, and 3.5. It is clear from Equations 3.1 and 3.5 that the damping provided by the isolation system reduces both the base shear and isolator displacement.

In order to compare the impact of the variables, involved in design procedure, on the performance of isolation systems with different damping values; a basis must be established. There are three alternatives cases;

- The seismic isolation systems have the same base shear.
- The seismic isolation systems have the same displacement.
- The seismic isolation systems have the same isolated period.

Each of these alternatives is evaluated with regard to their overall impact. In order to be able to easily follow, numerical comparison of a 10% damped (B_D = 1.2), a 15% damped (B_D = 1.35), and a 20% damped (B_D = 1.5) systems is discussed. 10%, 15%, and 20% damping values are selected for the comparison since they are in the practical range of currently available systems.

<u>Equal Base Shear</u>

When the base shear of two isolated systems is equal, the relationships for isolation period and displacement can be derived from Equations 3.1, 3.3 and 3.5.

$V_1 = V_2$

$$T_1 = \frac{T_2}{r}$$

$D_1 = \frac{D_2}{r^2}$

where $r = \frac{B_1}{B_2}$

B1 and B2 are the damping factors for the two isolation systems. To state these relations numerically:

V10 = V20	V15 = V20
$T_{10} = 1.25\ T_{20}$	$T_{15} = 1.11\ T_{20}$
$D_{10} = 1.56\ D_{20}$	$D_{15} = 1.24\ D_{20}$

From Figure 4.2 it is seen that for an equal base shear, 10% and 15% damped systems will require 25% and 11% longer periods and will produce 1.56 and 1.24 times the displacements of a 20% damped system

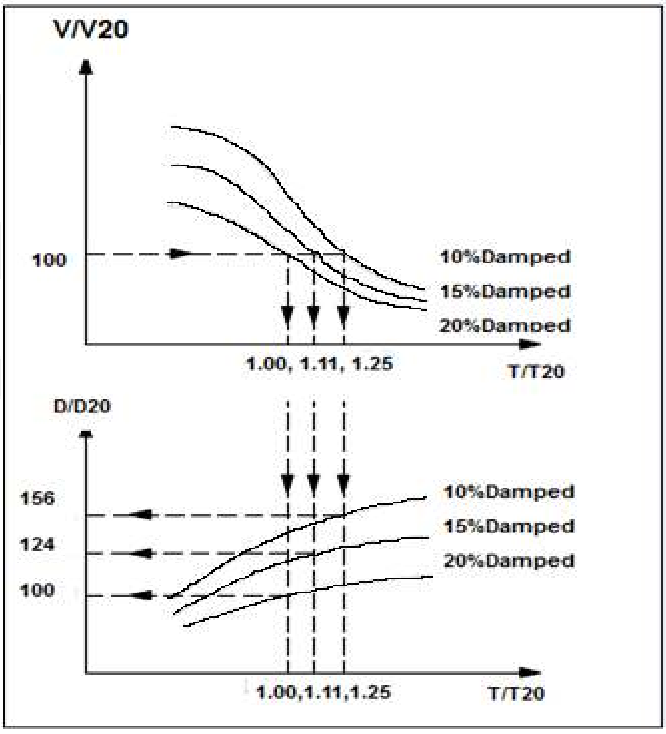

Figure 4.1 Effect of damping for equal base shear case

<u>Equal Isolated Periods</u>

When the isolated period of two systems is equal, the relationships for base shear and

displacement can be derived from Equations 3.1, 3.3 and 3.5.

$$T_1 = T_2$$

$$V_1 = \frac{V_2}{r}$$

$$D_1 = \frac{D_2}{r}$$

To state these relations numerically:

T10 = T20	T15 = T20
$V_{10} = 1.25\ V_{20}$	$V_{15} = 1.11\ V_{20}$
$D_{10} = 1.25\ D_{20}$	$D_{15} = 1.11\ D_{20}$

Thus, for an equal displacement, 10% and 15% damped systems will require 20% and 10% lower periods and will produce 1.56 and 1.24 times the base shear of a 20% damped system .

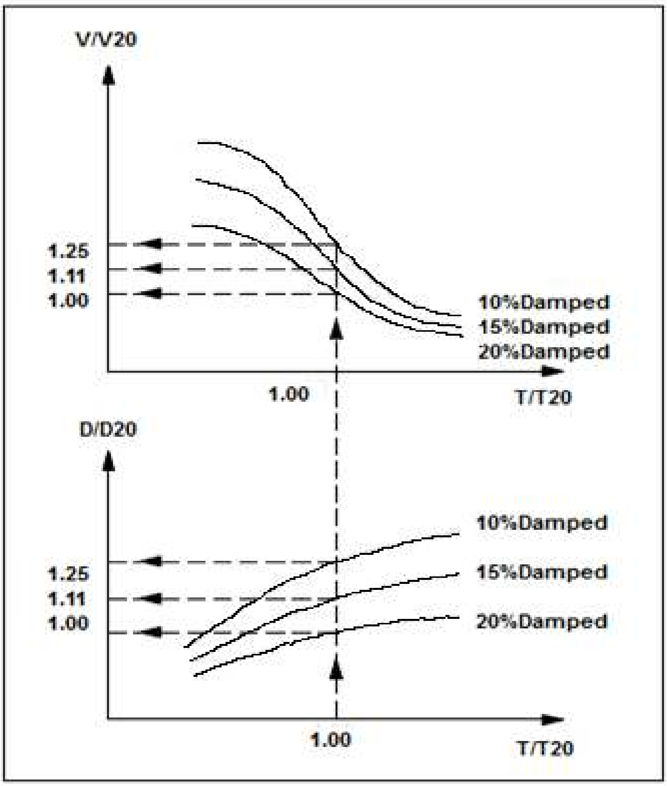

Figure 4.2 Effect of damping for equal period case.

For an equal isolated period, while 10% damped system will produce 25% greater displacement and base shear, 15% damped system will produce 11% greater displacement and base shear when compared to 20% damped system.

<u>Equal Displacements</u>

When the displacements of two isolated systems are equal, the relationships for isolation period and base shear can be derived from Equations 3.1, 3.3 and 3.5.

D_1=D_2

$T_1 = r.T_2$

V_1=$\frac{V_2}{r^2}$

To state these relations numerically:

D10 = D20	D15 = D20
T_{10} = 0.8 T_{20}	T_{15} = 0.9 T_{20}
V_{10} = 1.56 V_{20}	V_{15} = 1.24 V_{20}

Thus, for an equal displacement, 10% and 15% damped systems will require 20% and 10% lower periods and will produce 1.56 and 1.24 times the base shear of a 20% damped system.

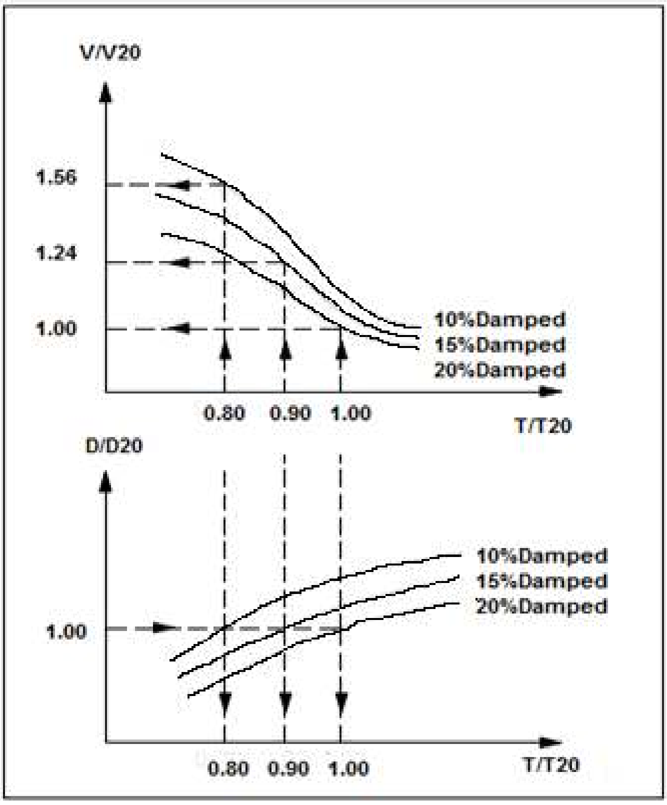

Figure 4.3 Effect of damping for equal displacement case

CHAPTER 5

ANALYSIS & DISCUSSION OF RESULTS

5.1 Analysis of the Base Isolated Symmetrical Building

In this section analysis and design building using IBC2006 code are performed for symmetrical building . To facilitate a study of the code provisions, building Types I, II and III are selected. The description of the buildings is introduced in Section 3.2. The high damping rubber bearings, HDR, designed in Section 4.1 are used for the isolation of the all buildings. Response spectrum analysis, described in Section 3.3.2, is carried out on building Types I, II and III. In addition, static equivalent lateral force and time history analyses, described in Section 3.3.1 and 3.3.3, are performed for Type II which typifies the class of symmetrical structure that is encountered in design. The analyses of the isolated buildings are done for soil type (C) that is given in the IBC 2006 code.

5.1.1 Scaling of the Results

The results of the analyses is scaled according to IBC2006 as mentioned in Section 3.4. The detailed calculation of scaling factors for each analysis method is given below.

5.1.1.1 Scaling for Static Equivalent Lateral Force Procedure

The limits of scaling mentioned in IBC2006 for static equivalent lateral force: The base shear must be greater than the lateral seismic force required for a fixed-base structure of the same weight and a period equal to the isolated period. Building Type-I is analyzed with static equivalent lateral force procedure and the results of the analysis are scaled according to the mentioned limit. For the calculation of the lateral seismic force, V, the procedure described in IBC2006 Seismic Code is used.

Occupancy Category Use Group: III

Occupancy Importance Factor, I: 1.25

Site Class: C

From Maps:

S_S = 1.89g

S_1 = 0.76g

Values of site coefficient:

Fa = 1.0

Fv = 1.3

Spectral Response Acceleration

S_{MS} = FaSs = 1.89 g

S_{M1} = FvS_1 = 0.988 g

Damped Design Spectral Response 5%

$S_{Ds}=\frac{2}{3}\,S_{MS}$=1.26

$S_{D1}=\frac{2}{3}\,S_{M1}$=0.658

Seismic Design Category: D

Natural Period:

$T_a\;=C_T\;hn^x$

h_n = 29.5 ft

T_a= 0.0466*$(29.5)^{0.9}$

Ta=0.9 sec

$$V\;=C_S\,W \tag{5.1}$$

$$C_S = \frac{S_{Ds}}{\left(\frac{R}{I}\right)} \tag{5.2}$$

R = the response modification factor

I = the occupancy importance factor

C_S = the seismic response coefficient

W = the effective seismic weight

V = total design lateral force

R=8 (Seismic load reduction factor for non-isolated building)

W=14068 KN

$$C_S = \frac{1.26}{\left(\frac{8}{1.25}\right)} = 0.19$$

$$C_{Smax} = \frac{S_{D1}}{\left(\frac{R}{I}\right) * Ta}$$

$$C_{Smax} = \frac{0.65}{\left(\frac{8}{1.25}\right) * 0.9} = 0.1142$$

$$C_{Smin} = \frac{0.5 * S_1}{\left(\frac{R}{I}\right)}$$

$$C_{Smin} = \frac{0.5 * 0.76}{\left(\frac{8}{1.25}\right)} = 0.05$$

$$V = 0.1142 * 14068 = 1606 \text{ kN}$$

- Calculating the base shear for base isolation

$$\mathrm{Vs} = \frac{\mathrm{K_h D_D}}{\mathrm{R}}$$

$\mathrm{K_h} = 417$ ($for\ section$ 4.1)

$\mathrm{D_D} = 0.24$ ($for\ section$ 4.1)

R'=1 (Seismic load reduction factor)

$$\mathrm{Vs} = \frac{417 * 0.24}{1} = 100.08 \qquad for\ one\ bearing$$

$100.08 * 24 = 2401$ kN/m

Table 5.1 Calculation of scaling factor for Type-I according to IBC2006

S_{D1}	V (kN) (Equation 5.1)	VS (kN) (Equation 3.5)	Scaling Factor
0.658	1606	2401	no need to scale

5.1.1.2 Scaling for Response Spectrum Analysis

Building Types I, II and III are analyzed with response spectrum analysis and the results of the analysis are scaled according to IBC2006. To be comprehensible, the parameters needed for the calculation of scaling factors are given below. The damping coefficient, B_D, is taken as 1.38 for the calculations since the HDR, bearings designed in Section 4.1, are used for the isolation. As a result of the modal analysis the fixed based period T, and isolated period T_D of the buildings are determined as:

Table 5.2 Fixed and isolated periods of buildings

	Type-I	Type-II	Type-III
T (sec.)	0.39	0.65	1.08
T_D(sec.)	2.1	2.7	3.5

Scaling according to IBC2006

When IBC2000 is considered, the design displacement determined by response spectrum analysis, $D_{analysis}$, must be greater than 90% of D_{TD}' as specified in Equation3.4.On the other hand, the design base shear force on the structure above the isolation system must be greater than 80% of V_S as prescribed by Equation 3.5.Otherwise, all response parameters, including inertial forces and deformations, must be adjusted proportionally upward.

When the results of the analyses are examined, it is seen that the first scaling limit, $D_{analysis}$, >0.9×D_{TD}', is more critical than the second one and results in greater scaling factors. Therefore, displacement dependent scaling limit is used in the scaling factor calculations.

Table 5.3 Calculation of scaling factor for Type-I according to IBC2006

S_{D1}	D_D (cm) (Equation 3.1)	D_{TD} (cm) (Equation 3.7)	0,9*D_{TD} (cm) (Equation 3.4)	$D_{analysis}$ (cm)	Scaling Factor
0.658	24	30	27	30	no need to scale

Table 5.4 Calculation of scaling factor for Type-II according to IBC2006

S_{D1}	D_D (cm) (Equation 3.1)	D_{TD}(cm) (Equation 3.7)	0,9*D_{TD} (cm) (Equation 3.4)	$D_{analysis}$ (cm)	Scaling Factor
0.658	31.9	39.9	35.9	37	no need to scale

Table 5.5 Calculation of scaling factor for Type-III according to IBC2006

S_{D1}	D_D (cm) (Equation 3.1)	D_{TD} (cm) (Equation 3.7)	0,9*D_{TD}(cm) (Equation 3.4)	$D_{analysis}$ (cm)	Scaling Factor
0.658	41.3	51.7	46.5	40	1.15

5.1.1.3 Scaling for Time History Analysis

Building Type II is analyzed with time history analysis and the results of the analysis are scaled according to IBC2006. The parameters needed for the calculation of scaling factors are given below. The damping coefficient, B_D, is taken as 1.38 for the calculations since the HDR, bearings designed in Section 4.1, are used for the analysis. The fixed based, T, and isolated periods, T_D, of the building are given in Table 5.14

Scaling according to IBC2006

The displacement dependent scaling limit for time history analysis is same with the one given for response spectrum analysis. However, the design base shear force on the structure above the isolation system must be greater than 60% of V_S as prescribed by Equation 3.5. Otherwise, all response parameters, including component actions and deformations, must be adjusted proportionally upward. When the results of the analyses are examined, it is seen that the first scaling limit, $D_{analysis}$ > 90% of D_{TD}', is more critical than the second one and results in greater scaling factors. Therefore, it is used in the scaling factor calculations.

Table 5.6 Calculation of scaling factor for Type-III according to IBC2006

S_{D1}	D_D (cm) (Equation 3.1)	D_{TD} (cm) (Equation 3.7)	0,9*D_{TD}(cm) (Equation 3.4)	$D_{analysis}$ (cm)	Scaling Factor
0.658	41.3	51.7	46.5	40.2	1.12

5.1.2 Results of the Analyses

The results of the analyses of building Type-II, representing the typical symmetrical structure, are given in Table 5.7,5.8 and 5.9 below; also in order to be able to compare different analysis methods a comparison table, is prepared. It can be concluded from the comparison table that response spectrum analysis, scaled according to IBC 2006 provisions, results in the most conservative design values.

Table 5.7 Results of the analyses for Type-I

	methods earthquake	T_D *(sec)*	*Base Shear in X Direction* VX (kN)	*Base Shear in Y Direction* VY (kN)	*Base Moment in X Direction* MX (kN.m)	*Base Moment in Y Direction* MY (kN.m)	*Max. Interstory Drift Ratio*	*Max. Isolator Displacement (cm)*	
IBC 2006	Time History	2	2497.285	2474.747	8001.544	8182.8948	0.0008	32.1	no need to scale
	Response Spectrum	2	2650.708	2618.028	7604.981	7708.5819	0.0009	30	no need to scale
	Equivalent Lateral	2	2769.723	2769.723	10923.073	10923.073	0.001	32	no need to scale

Table 5.8 Results of the analyses for Type-II

	methods earthquake	T_D *(sec)*	*Base Shear in X Direction* VX (kN)	*Base Shear in Y Direction* VY (kN)	*Base Moment in X Direction* MX (kN.m)	*Base Moment in Y Direction* MY (kN.m)	*Max. Interstory Drift Ratio*	*Max. Isolator Displacement (cm)*	
IBC 2006	Time History	2.7	3136.478	3223.928	19177.1875	18502.4648	0.0008	37.6	no need to scale
	Response Spectrum	2.7	3259.722	3231.486	19414.2672	19509.9159	0.001	37	no need to scale
	Equivalent Lateral	2.7	3231.942	3231.942	19177.1875	18502.4648	0.0012	38	no need to scale

Table 5.9 Results of the analyses for Type-III

	methods earthquake	T_D *(sec)*	*Base Shear in X Direction* VX (kN)	*Base Shear in Y Direction* VY (kN)	*Base Moment in X Direction* MX (kN.m)	*Base Moment in Y Direction* MY (kN.m)	*Max. Interstory Drift Ratio*	*Max. Isolator Displacement (cm)*	
IBC 2006	Time History	3.5	3912.82	3815.81	40864.1829	40899.3651	0.0011	46.2	no need to scale
	Response Spectrum	3.5	3866.13	3820.57	41342.2816	41455.758	0.0011	46.3	results scaled
	Equivalent Lateral	3.5	3387.47	3387.47	49783.7935	49783.7935	0.0017	48	results scaled

5.2 Analysis of the Base Isolated Non-symmetrical Building

To identify the effect of "scaling" phenomenon mentioned in IBC2006 and make a "scaling" point of view, building Type-IV, introduced in Section 3.2.4, is analyzed. The high damping rubber bearings, HDR, designed in Section 4.1 are used for the isolation of the buildings. Static equivalent lateral force, response spectrum and time history analyses, described in Sections 3.3.1, 3.3.2 and 3.3.3, are carried out on the model building which typifies the class of non-symmetrical structure that is encountered in design. The analyses of the isolated buildings are done for soil type (C) .

5.2.1 Scaling of the Results

The results of the analyses are scaled according to IBC2006 as mentioned in Section 3.4. The detailed calculation of scaling factors for each analysis method is given below.

5.2.1.1 Scaling for Static Equivalent Lateral Force Procedure

The limits of scaling mentioned in IBC2006 for static equivalent lateral force .The base shear must be greater than the lateral seismic force required for a fixed-base structure of the same weight and a period equal to the isolated period. Building Type-IV is analyzed with static equivalent lateral force procedure and the results of the analysis are scaled according to the mentioned limit. For the calculation of the lateral seismic force, V, the procedure described in IBC 2006 Code is used.

Occupancy Category Use Group: III

Occupancy Importance Factor, I: 1.25

Site Class: C

From Maps:

$S_S = 1.89g$

$S_1 = 0.76g$

Values of site coefficient:

Fa = 1.0

Fv = 1.3

Spectral Response Acceleration:

S_{MS} = FaSs = 1.89 g

S_{M1} = FvS_1 = 0.988 g

Damped Design Spectral Response 5%

$S_{Ds}=\frac{2}{3}S_{MS}$=1.26

$S_{D1}=\frac{2}{3}S_{M1}$=0.658

Seismic Design Category: D

Natural Period:

$T_a = C_T\, h_n{}^x$

h_n = 49.21 ft

Ta = 0.0466*$(49.21)^{0.9}$

Ta=1.5sec

$$V = C_S\, W \quad (5.1)$$

$$C_S = \frac{S_{Ds}}{\left(\frac{R}{I}\right)} \quad (5.2)$$

R = the response modification factor

I = the occupancy importance factor

C_S = the seismic response coefficient

W = the effective seismic weight

V = total design lateral force

R=8 (Seismic load reduction factor for non-isolated building)

W=18671 KN

$$C_S = \frac{1.26}{\left(\frac{8}{1.25}\right)} = 0.19$$

$$C_{Smax} = \frac{S_{D1}}{\left(\frac{R}{I}\right) * Ta}$$

$$C_{Smax} = \frac{0.65}{\left(\frac{8}{1.25}\right) * 1.5} = 0.068$$

$$C_{Smin} = \frac{0.5 * S_1}{\left(\frac{R}{I}\right)}$$

$$C_{Smin} = \frac{0.5 * 0.76}{\left(\frac{8}{1.25}\right)} = 0.05$$

Therefore use C_S = 0.068

$V = 0.068 * 18671 = 1269$ kN

- Calculating the base shear for base isolation

$$\mathrm{Vs} = \frac{\mathrm{K_h D_D}}{\mathrm{R}}$$

$\mathrm{K_h} = 417$ ($for\ section$ 4.1)

$\mathrm{D_D} = 0.26$ ($for\ section$ 4.1)

R'=1 (Seismic load reduction factor)

$$\mathrm{Vs} = \frac{417 * 0.26}{1} = 108.42 \qquad for\ one\ bearing$$

$108.42 * 20 = 2168$ kN/m

Table 5.1 Calculation of scaling factor for Type-II according to IBC2006

S_{D1}	V (kN) (Equation 5.1)	VS (kN) (Equation 3.5)	Scaling Factor
0.658	1269	2168	no need to scale

5.2.1.2 Scaling for Response Spectrum Analysis

Building Types IV is analyzed with response spectrum analysis and the results of the analysis are scaled according to IBC2006. To be comprehensible, the parameters needed for the calculation of scaling factors are given below. The damping coefficient, B_D, is taken as 1.38 for the calculations since the HDR, bearings designed in Section 4.1, are used for the isolation. As a result of the modal analysis the fixed based period T, and isolated period T_D of the buildings are determined as:

Table 5.11 Fixed and isolated periods of buildings

	Type-I
T (sec.)	0.42
T_{D1} (sec.)	2.2

When IBC2000 is considered, the design displacement determined by response spectrum analysis, $D_{analysis}$, must be greater than 90% of D_{TD}' as specified in Equation3.4.

On the other hand, the design base shear force on the structure above the isolation system must be greater than V_S as prescribed by Equation 3.5.Otherwise, all response parameters, including inertial forces and deformations, must be adjusted proportionally upward.

When the results of the analyses are examined, it is seen that the first scaling limit, $D_{analysis}$, >0.9×D_{TD}', is more critical than the second one and results in greater scaling factors. Therefore, displacement dependent scaling limit is used in the scaling factor calculations.

Table 5.12 Calculation of scaling factor for Type-IV according to IBC 2006

S_{D1}	D_D (cm) (Equation 3.1)	D_{TD} (cm) (Equation 3.7)	0,9*D_{TD} (cm) (Equation 3.4)	$D_{analysis}$ (cm)	Scaling Factor
0.658	26	32.5	29.2	43	no need to scale

5.2.1.3 Scaling for Time History Analysis

Building Type IV is analyzed with time history analysis and the results of the analysis are scaled according to IBC2006. The parameters needed for the calculation of scaling factors are given below. The damping coefficient, B_D, is taken as 1.38 for the calculations since the HDR, bearings designed in Section 4.1, are used for the analysis. The fixed based, T, and isolated periods, T_D, of the building are given in Table 5.14 The displacement dependent scaling limit for time history analysis is same with the one given for response spectrum analysis.

However, the design base shear force on the structure above the isolation system must be greater than 60% of V_S as prescribed by Equation 3.5. Otherwise, all response parameters, including component actions and deformations, must be adjusted proportionally upward. When the results of the analyses are examined, it is seen that the first scaling limit, $D_{analysis}$ > 90% of D_{TD}', is more critical than the second one and results in greater scaling factors. Therefore, it is used in the scaling factor calculations.

Table 5.13 Calculation of scaling factor for Type-IV according to IBC2006

S_{D1}	D_D (cm) (Equation 3.1)	D_{TD} (cm) (Equation 3.7)	0,9*D_{TD} (cm) (Equation 3.4)	$D_{analysis}$ (cm)	Scaling Factor
0.658	26	32.5	29.2	23.7	1.23

5.2.2 Results of the Analyses

The results of the analyses of building Type-IV, representing the typical non-symmetrical structure, are given in Table 5.13 below; also in order to be able to compare different analysis methods a comparison table. It can be concluded from the comparison table that response spectrum analysis scaled according to IBC 2006 provisions, results in the most conservative design values.

Table 5.14 Results of the analyses for Type-II

	methods earthquake	T_D *(sec)*	*Base Shear in X Direction* VX (kN)	*Base Shear in Y Direction* VY (kN)	*Base Moment in X Direction* MX (kN.m)	*Base Moment in Y Direction* MY (kN.m)	*Max. Interstory Drift Ratio*	*Max. Isolator Displacement (cm)*	
IBC 2006	Time History	2.2	3900.367	3176.053	19103.3614	1408.4091	0.008	29.2	results scaled
	Response Spectrum	2.2	2731.677	2573.9	15201.3544	16082.0361	0.0005	43	no need to scale
	Equivalent Lateral	2.2	2573.707	2573.707	20887.097	20887.097	0.0007	34	no need to scale

5.3 Comparison between base –fixed and base isolation

Building Type II is analyzed with time history analysis Comparison between base –fixed and base isolation about base shear and moment shear and displacement and the results of the analysis are not scaled according .

Table 5.15 Results of the analyses for Type-II Time Period for different Mode shape

NUMBER	MODE SHAPE	TIME PERIOD (Sec.) FIXED BASE	TIME PERIOD (sec) RUBBER ISO
1	Mode	0.658218	3.569026
2	Mode	0.649716	3.234535
3	Mode	0.627309	3.210074
4	Mode	0.555953	1.104608
5	Mode	0.481895	0.716263
6	Mode	0.475950	0.693995
7	Mode	0.391598	0.497831
8	Mode	0.345471	0.414016
9	Mode	0.212128	0.367000
10	Mode	0.207803	0.338713
11	Mode	0.207291	0.332619
12	Mode	0.202469	0.328168

Table 5.16 Results of the analyses for Type-II displacement for different floors

FLOORS	DISPLACEMENT FIXED BASE m	DISPLACEMENT RUBBER ISO m
1	0.019620	0.396366
2	0.042210	0.402285
3	0.062072	0.406728
4	0.077233	0.410260
5	0.086344	0.412882

Table 5.17 Results of the analyses for Type-II

FIXED BASE					RUBBER ISO				
Base Shear in X Direction VX (kN)	*Base Shear in Y Direction* VY (kN)	*Base Moment in X Direction* MX (kN.m)	*Base Moment in Y Direction* MY (kN.m)	*Max. Interstory Drift Ratio*	*Base Shear in X Direction* VX (kN)	*Base Shear in Y Direction* VY (kN)	*Base Moment in X Direction* MX (kN.m)	*Base Moment in Y Direction* MY (kN.m)	*Max. Interstory Drift Ratio*
13940	14393	106042	102504	0.003	3557	3506	21247	21266	0.0007

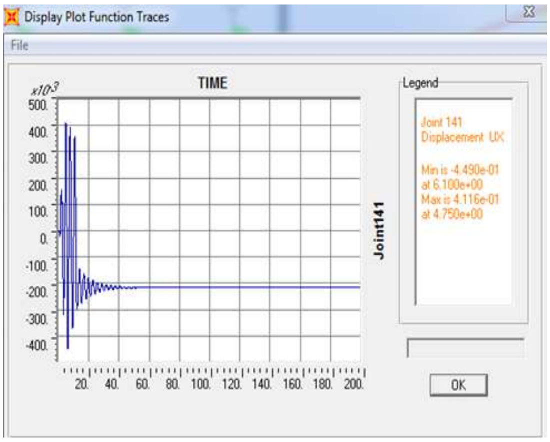

Figure 5.1 time history for displacement with rubber isolation along x direction

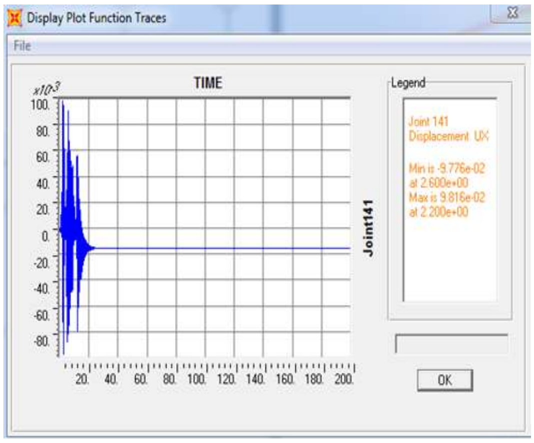

Figure 5.2 time history for displacement with base- fixed along x direction

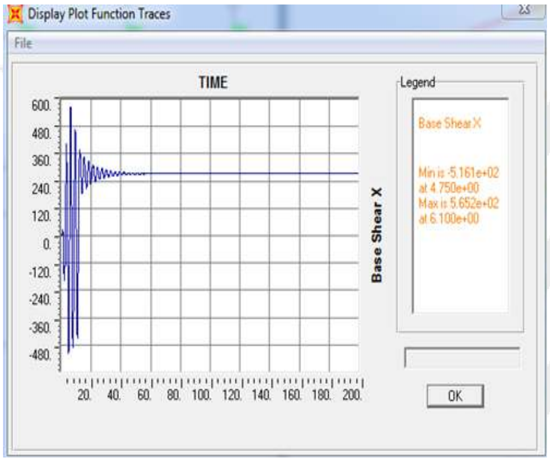

Figure 5.3 time history for base shear with rubber isolation along x direction

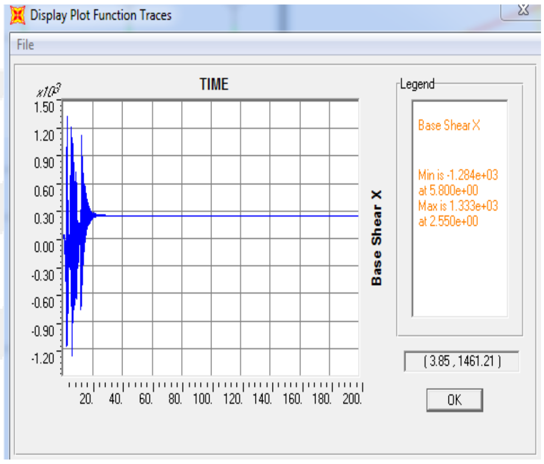

Figure 5.4 time history for base shear with base -fixed along x direction

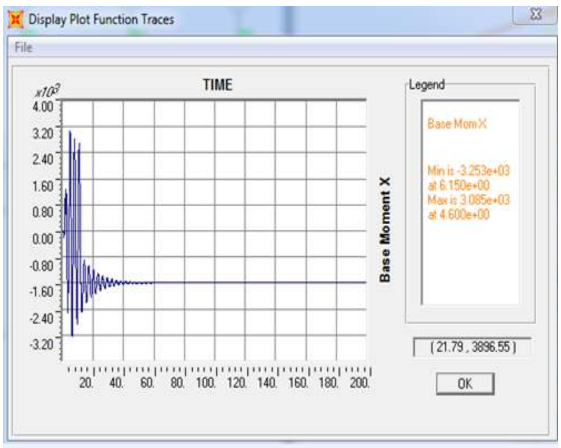

Figure 5.5 time history for base moment with rubber isolation along x direction

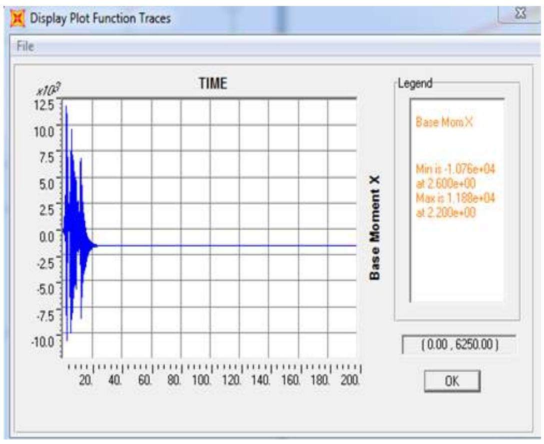

Figure 5.6 time history for base moment with base –fixed along x direction

CHAPTER 6

CONCLUSIONS AND FUTURE WORK

6.1 Conclusions

In this study, the design of seismic isolation systems is explained and the influence of base isolation on the response of structure is examined in details. Various types of isolators are introduced and one of the most commonly used type, high damping rubber bearing, is used in the case studies. In this study alternatives of modeling an isolator for design purposes, linear, is discussed; also advantages and disadvantages of is stated. The analyses of isolated buildings, symmetrical and non-symmetrical in plan, are performed according to the related chapters of the design IBC2006. According to these analyses, the codes are compared for each type of analysis method.

In the light of the results obtained from the case studies, the following conclusions

can be stated

- When compare between three methods Equivalent Lateral Force, Response Spectrum and Time History Analysis in four building type I,II,III and IV. The conclusion that the results of Response Spectrum analysis and Time History Analysis are more accurate than the Equivalent Lateral Force Analysis.
- In the section 5.3 Comparison between base–fixed and base isolation and conclusion Base isolation method has proved to be a reliable method of earthquake resistant design. The success of method is largely attributed to the design of isolation devices and proper planning and placing the isolators.
- The results of the investigation show that the response of the structure can be dramatically reduced by using base isolation
- The base shear in x-direction is (3557) with base isolation, less than the base shear of fixed base (13940).
- The base shear in y-direction is (3506) with base isolation, less than the base shear of fixed base (14393).

- The base moment in x-direction is (21247) with base isolation, less than the base moment of fixed base (106042).
- The base moment in y-direction is (21266) with base isolation, less than the base moment of fixed base (102504).
- The drift ratio is (0.0007) with base isolation, less than the drift ratio of fixed base (0.003).
- The results of displacement shows that the displacements are increased with the period and with the storey height in the base isolated building a large proportion when compared to that of fixed base.

6.2 FUTURE WORK

My recommendations are to study non-linear effect of the base isolation and also to study two different types of base isolation like Lead-Rubber Isolator (LRB) and Friction Pendulum System.

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Appendix A

Design of High Damping Rubber Bearing

A.1 Lateral Stiffness of Base Isolators

By using Equation 3.3 for 'Design Level Earthquake':

$$2.1=2\pi\sqrt{\frac{14068}{k\ total\quad 9.81}} \quad\rightarrow\quad k_{total}=11697\ KN/M$$

$$K_D=\frac{11697}{28}=417KN/M \qquad (for\ one\ bearing)$$

For 'Maximum Capable Earthquake Level':

$$2.5=2\pi\sqrt{\frac{14068}{k\ total\quad 9.81}} \quad\rightarrow\quad k_{total}=6290\ KN/M$$

$$K_M=\frac{6290}{28}=224\ KN/M \qquad (for\ one\ bearing)$$

where k_D and k_M are the minimum lateral stiffness of base isolation bearings Corresponding to the 'design earthquake' and 'maximum capable earthquake', Respectively.

A.2 Estimation of Lateral Displacements

From Equation 3.1

$$D_D=\left(\frac{9.81}{4\pi 2}\right)\frac{0.65\times 2.1}{1.38}=0.24M$$

$$D_M=\left(\frac{9.81}{4\pi 2}\right)\frac{0.65\times 2.5}{1.38}=0.29M$$

Where D_D and D_M are the displacements of the isolation system corresponding to the 'design earthquake' and 'max. capable earthquake', respectively. The damping reduction factor B=1.38 is obtained from Equation 3.2.

A.3 Estimation of Disc Dimensions

Thickness of the disc can be calculated by,

$t_r = \frac{D_D}{\gamma_{max}}$

Therefore

$$t_r = \frac{0.24}{1.5} = 0.16m$$

Take

$t_r = 20\ cm$

Disc diameter, Φ ,is estimated by using equation 2.1

$$A = \frac{417 \ \times 0.2}{550} = 0.151M^2$$

$$\Phi = \sqrt{\frac{4A}{\pi}} \quad = 0.438M$$

Take

$\Phi = 44\ CM$ $\qquad A = 0.151\ M^2$

A.4 Actual Bearing Stiffness & Revised Fundamental Period

$$K_D = \frac{0.151\times 550}{0.2} = 415.2\ KN/m$$

$$K_{total} = 415\times 28 = 11627\ KN/M$$

$$T_D = 2\pi\sqrt{\frac{14068}{11627 \times 9.81}} = 2.20\ sec$$

It is seen that the calculated revised value of T_D = 2.20 sec

A.5 Bearing Detail

For compressive stresses under vertical loads, the isolators undergo relatively smaller shear strain on older $\gamma = 0.2$. therefore G= 650 KN/m should be used. Shape factor, S, is selected as 8. The compression modulus, Ec, from Equation 2.4 is :

$$E_c = \frac{6 \times 8^2 \times 650 \times 2000000}{6 \times 8^2 \times 650 + 2000000} \quad = \quad 221906.11 \ KN/m^2$$

where the total vertical stiffness is determined from Equation 2.2 as,

$$k_v = \frac{0.151 \times 28 \times 221906.11}{0.2} = 4{,}691{,}095.24 \, KN/m$$

The vertical displacement, Δt_v, becomes;

$$\Delta t_v = \frac{W}{k_v} = \frac{14068}{4{,}691{,}095.24} = 2.99 \times 10^{-3} \, m$$

The vertical fundamental period of vibration is given by;

$$T_v = \frac{T_H}{\sqrt{6}\, S}$$

gives,

$$T_v = \frac{2.20}{\sqrt{6} \times 8} = 0.112 \, sec$$

As a result S = 8 is adequate

$$S = \frac{\Phi}{4t_o}$$

where to is the thickness of single layer of rubber.

$$t_o = \frac{440}{4 \times 8} = 13.75mm$$

$n \times t_o = 200 \, mm$ $\qquad\qquad$ $n = 14$ layers

Consequently, the design of the bearing is completed as shown in Figure 4.1. The end plates are 25mm thick, and the steel shims are 2mm each. The total height is:

$$h = (2 \times 25) + (14 \times 20) + (10 \times 2) = 350mm$$

the steel shims will have a diameter Φ_s = 440 mm, giving 5mm cover.

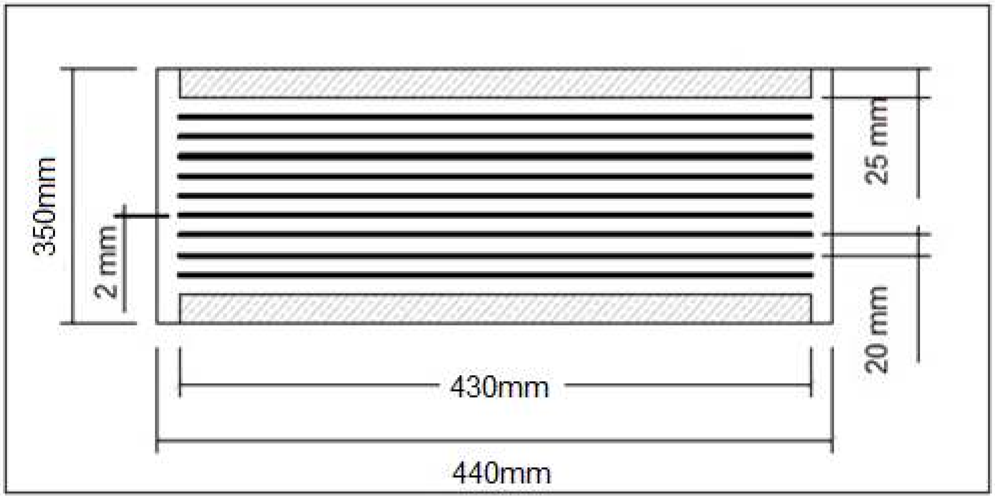

Figure A.1 Detail design of isolator.

A.6 Buckling Load

$$p_{cr} = \sqrt{p_s \times p_E}$$

$$P_S = G\,A$$

$$p_E = \frac{\pi^2\,(EI)_{eff}}{t_r^2}$$

$$(EI)_{eff} = \frac{1}{3}\,E_c I$$

$$\rightarrow \quad p_{cr} = \sqrt{\frac{0.151 \times 550 \times \pi^2 \times 221906.11 \times \pi \times 0.34^4 \times 0.25}{3 \times 0.2^2}} = 3988\,KN$$

Maximum vertical load for one isolator is calculated from tributary area:

$$(fv)_{max} = \frac{5.5 \times 3.75}{33 \times 13} \times 14068 = 676.18\,KN$$

Factor of Safety ,$FS = \frac{3988}{676.18} = 5.8$