

AN OVERVIEW OF THE TECHNOLOGY AND DESIGN OF BASE ISOLATED  
BUILDINGS IN HIGH SEISMIC REGIONS IN THE UNITED STATES

by

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## **Abstract**

Seismic hazards are a primary concern in some of the most populous regions in the United States. Performance-based seismic design has brought about new technology advances and introduced an innovative approach towards constructing seismic resistant buildings. Base isolation and structural damping systems are becoming increasingly utilized methods of advanced seismic resistance. This relatively new design approach presents various issues that must be addressed throughout the design and construction processes. A brief background on the origin, dynamics, and hazards of earthquakes and a discussion on designs of traditional, fixed-based structures is presented in this report. A description for selected types of new advanced seismic restraint systems, with an emphasis on base isolation, is also provided. Examples of current applications of buildings equipped with base isolation are presented. This report concludes with a review of the fundamental design methodology for structural base isolation along with additional requirements not addressed by the current building codes.

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## CHAPTER 1 - Introduction

Geological and seismological discoveries during the 20<sup>th</sup> century have helped initiated the development of seismic building codes and earthquake resistant buildings and structures. The improvement in seismic design requirements has led to more robust, safe and reliable buildings. Developments in plate tectonic theories have resulted in a greater understanding for the dynamics of earthquakes which are discussed in Chapter 2 and their occurrences, some of which are mentioned in Chapter 3. These developments also helped establish improved seismic building codes. In the late 20<sup>th</sup> century, the traditional fixed base seismic resistant building designs were taken to another level. Structural base isolation was developed as an alternative method to resist seismic energies and to provide a level of safety beyond what had been traditionally designed for and that of which was established by building codes at the time. Chapter 4 describes the basics behind traditional fixed base seismic design.

Structural base isolation is a modern idea and concept. An introduction to advanced seismic restraint systems is thoroughly discussed in Chapter 4. Technology advancements and building code improvements have helped to introduce such concepts into the field of engineering design. Early applications of base isolation were in the mid 20<sup>th</sup> century. The first building application in the United States was the Foothill Community Law and Justice Center in San Bernardino, California, constructed in 1985. Prior to the Foothill Community Law and Justice Center, many civil structures such as bridges utilized seismic isolation. Much of the development and research advancements for base isolation of buildings are credited to James M. Kelly.

Provisions for the design of seismic base isolated buildings have recently been included in the 2006 *International Building Code (IBC)* by adopting the American Society of Civil Engineers, *Minimum Design Loads for Buildings and Other Structures*, ASCE 7-05 (ASCE 2006). However, with only a few decades of actual application within the United States, the current building codes are not well defined and are based on overly conservative design approaches. ASCE 7 defines two methods of analysis, one of which is a simplified method for more simplistic, regular structures located on geological sites that will not produce irregular or vigorous earthquake energies. Structures with properties beyond the design capabilities of the

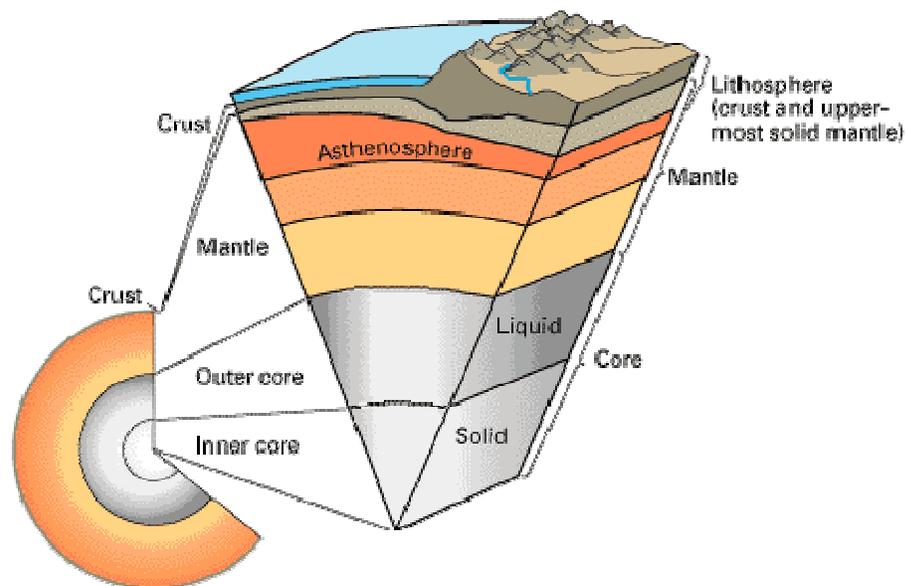
simplified method are required to be designed by use of a more complex dynamic method. The dynamic theories of base isolation are not exactly portrayed through the design procedures under the simplified, equivalent lateral force procedure or the dynamic, response spectrum analysis. This results in overly conservative design methods and the underutilization of the principles that base isolation has been developed upon. Chapter 6 provides information on the design methods and requirements addressed by ASCE 7 for seismically base isolated structures.

Additionally, many considerations should be taken into account when designing base isolated structures that are not directly addressed within the current building code seismic isolation provisions. The gap required to be maintained around the perimeter of the structure and the means of providing fall protection should be considered in addition to those addressed in the building codes. Other items needing to be thoroughly considered include utility line connections, the design of elements cross the isolation interface and movement requirements between building walkways just to name a few. Without prior knowledge to such additional design considerations or previous design experience these issues may be left unidentified. These issues also relate back to the cost of the seismic isolation system and the design expenses. Chapter 7 discusses several design considerations which should be taken into account in addition to those items described within ASCE 7.

Current design provisions are the foundation for the future growth of advanced seismic base isolation. However, many issues still need to be addressed in order to fully utilize the application of structural base isolation. As research develops, the design approach and methodology for base isolation should correspondingly continue to grow and advance. Applications of structural base isolation are mentioned in Chapter 8.

## CHAPTER 2 - Earthquakes

Over the past hundred years, Geophysics and Seismologists have made several important discoveries about the properties of the Earth which has led to improved knowledge about Seismology. The Earth consists of three major layers: the crust, mantle and core. Two types of crust exist; continental crust and oceanic crust. Continental crusts are thin, rigid elements composed of lightweight minerals such as quartz, feldspar and potassium which range from 21.75 miles (35 km) to 43.5 miles (70 km) thick (USGS 1999). Oceanic crust is composed of much heavier and dense materials such as basaltic rock which is rich in iron and magnesium having an average thickness of 4.35 miles (7 km) (Girty 2007). The mantle is defined as a dense, hot layer of semi-solid rock below the crust, approximately 1,802 miles (2,900 km) thick and approximately 82% of the Earth's volume (USGS 1999). The mantle of the Earth is typically divided into two parts: the cool, upper mantle and warm, deeper mantle. Below the mantle is the Earth's core, the densest layer. The earth's solid outer core is mainly composed of iron whereas the liquid molten inner core is composed primarily of nickel-iron alloys. The division of the Earth is illustrated in Figure 2.1.



**Figure 2.1 The Division of the Interior of the Earth. (USGS 1999).**

## Theory of Tectonic Plates and Boundaries

Most earthquakes are a manifestation of the fragmentation and motion of the Earth's outer shell (known as the lithosphere) which is divided into various large and small plates. Each plate varies in thickness and is approximately 50 to 60 miles thick (80 to 100 km). Seismology deals with the Earth's crust and is best described by the theory of plate tectonics. Plate tectonic theories began emerging around the early 1960's (USGS 1999). These theories have helped to establish that the Earth's outer crust is divided into several tectonic plates varying in size and thickness, and drifting relative to one another. Seismologists have been able to locate the tectonic plate boundaries which can be mapped using space orbiting satellites (USGS 1999). Figure 2.2 shows the division of the Earth's crust into the different tectonic plates where the darkened lines depict plate boundaries.

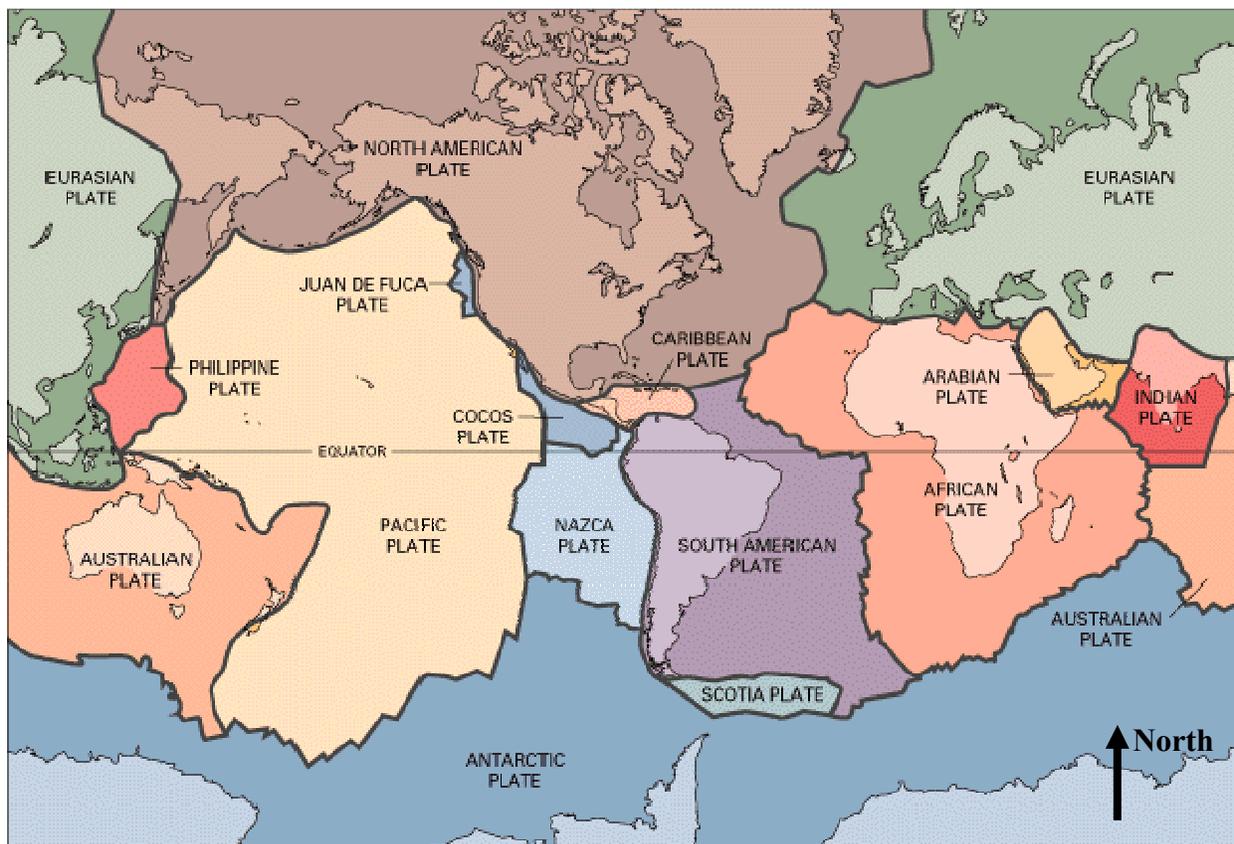


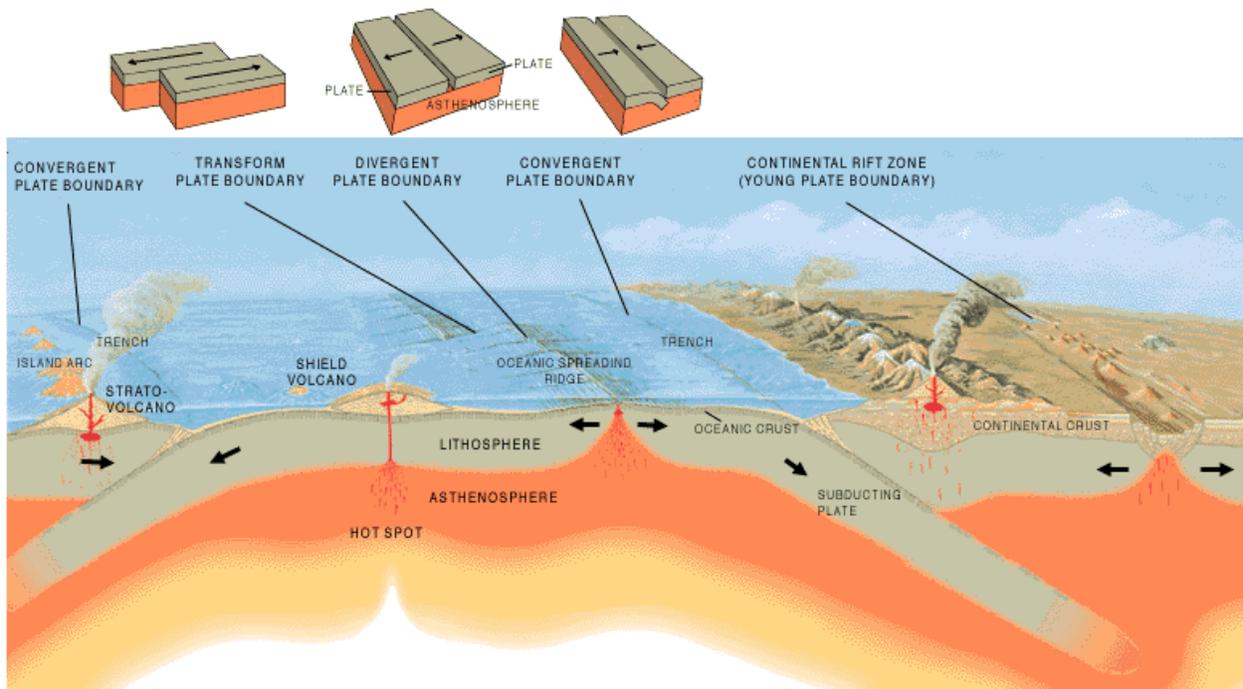
Figure 2.2 Earth's Tectonic Plate Divisions. (Image courtesy of USGS)

## **Types of Tectonic Plate Boundaries**

Four types of interactions can occur at the boundary locations between adjacent tectonic plates; divergent, convergent, transform, and boundary. Beneath the top divided shell of the Earth is a slow moving viscous layer on which the tectonic plates slide. Thin plate regions deform through elastic bending and brittle rupture while thicker regions along a plate yield plastically. The plates themselves tend to be rigid internally and typically only interact at the edges. However, plates can also interact away from the edges or boundaries causing interior plate zones.

### ***Divergent Boundaries***

Divergent boundaries occur when two plates separate from each other and cause cracking, or rifting, at the Earth's surface. New crust is formed from upward moving magma rising from the Earth's mantle. Rising magma can cause high pressure areas which may result in additional rifting at the boundary surface. Most divergent boundaries form along the bottom of the ocean. As the plates move apart the ridge material gradually cools and contracts and its surface sinks. Strike-slip faults form parallel to the direction of the plate motion. Figure 2.3 illustrates the main types of plate boundaries which includes divergent boundaries.



**Figure 2.3 Illustration of Three Main Types of Plate Boundaries, Convergent Plate Boundary, Transform Plate Boundary and Divergent Plate Boundary. (Image Courtesy of USGS)**

### ***Convergent Boundaries***

Convergent boundaries occur when two plates move towards each other. Along convergent boundaries, one plate may override an adjacent plate; this region, known as a subduction zone. When a plate is forced to subside below another plate, it converges with the mantle of the Earth at which point the crust will be heated into magma. Plate convergence occurs commonly between either oceanic and large continental plates, two large oceanic plates or two large continental plates (USGS 1999). When plates converge, strong destructive earthquakes and rapid uplift of the ground are common results. Numerous strong to moderate earthquakes can occur from two converging oceanic plates. The Nazca Plate and the South American Plate shown in Figure 2.2 create a convergent boundary (USGS 1999). Here the Nazca Plate is continuously being pushed beneath the South American Plate (USGS 1999). Figure 2.3 illustrates the interaction of a convergent boundary.

### ***Transform Boundaries***

Transform boundaries occur when two plates move parallel to the plates boundary lines, or horizontally past each other. These fault regions are typically found along the ocean floor and cause earthquakes at shallow depths. Oceanic crusts are mainly involved in transform boundaries (USGS 1999). Unlike convergent boundaries, the plate interactions at transform boundaries do not force crust into the Earth's mantle. The North American Plate and the Juan de Fuca Plate shown in Figure 2.2 are examples of a transform boundary. The interaction of a transform boundary is illustrated in Figure 2.3.

### ***Plate Boundary Zones***

Some types of plate interactions occur at regions where boundaries are not well defined. These areas are represented by large bands. Plate interactions are more difficult to identify in these zones. The region between the Eurasian and African Plates, shown in Figure 2.2, is an example of a plate-boundary zone which involves two large sized plates. Earthquakes that are produced in these regions have a complex pattern because of the complicated plate structures (USGS 1999).

### ***Interior Plate Zones***

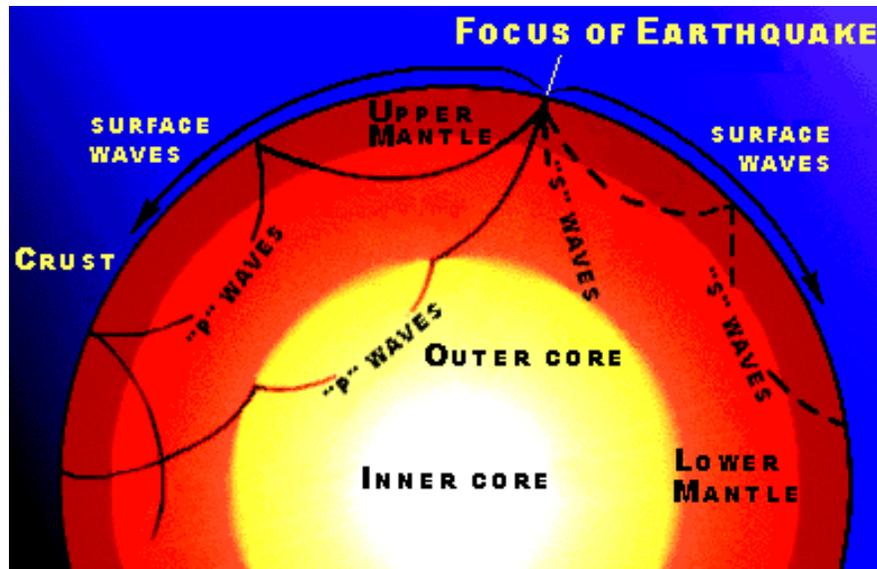
Earthquakes are also capable of occurring within plate regions rather than at the plate boundaries, only 10 percent of all earthquakes occur in such areas (USGS 1997). Interior plate zones are developed over time from continually moving plate boundaries. Such interior regions are weakened by stresses originated at the edges of the plate boundaries (USGS 1997). An example of an interior plate fault was demonstrated in the New Madrid earthquakes in 1811 and 1812 within the North American Plate which can be seen in Figure 2.2 (USGS 1997).

## **Earthquake Seismic Waves**

Earthquakes are best described as the sudden movement of the Earth's crust, or plate boundaries, caused by an abrupt release of strain. The strain within the Earth's crust can build up over anywhere from a few years to several decades. The energy produced from the release of

strain can cause very damaging ground motions to be dissipated through the Earth's surface triggering major earthquakes. The slow, creeping movement between plate boundaries frequently produce small to moderate earthquakes having a magnitude of 5.0 or less on the Richter scale. Small earthquakes are not usually felt at the surface of the earth. Depending on the magnitude, size, and type of plate rupture, earthquakes can be felt anywhere from a few miles to hundreds of miles away from the earthquakes origin. The origin known as the earthquakes focus or hypocenter is the point at which the first earthquake motion occurs. The location directly above the focus is the epicenter which defines the earthquakes origin along the Earth's surface. When an earthquake is caused by a fault rupture energy waves are released and spherically spread, expanding outward from the focus (Girty 2007). Seismic energy is the greatest at the focus and gradually weakens with distance from the earthquakes origin.

Seismic waves are defined as either body waves or surface waves. Body waves travel through the interior of the earth radiating spherically from the focus. These waves are composed of compressional P-waves and shear S-waves. Energy associated with P-waves produce a series of contractions and expansions within the material which they travel (Girty 2007). P-waves are capable of passing through a variety of materials whether liquid, solid or gaseous (Girty 2007). Once compressional waves have passed through the Earth's composition the ground which they traveled returns to its original form without permanent displacement. Shear waves however, radiate outward spherically from the focus producing deformations of the Earth's surface. S-waves cause material to be displaced perpendicular to the direction in which they are moving. However, S-waves do not travel through liquid because they are not capable of horizontal change of such an element which does not have an explicit structural form. P-waves are considered primary waves and travel twice as fast as S-waves and are the first to arrive at seismic recording stations. Figure 2.4 shows how these energy waves pass throughout the Earth's layers from the earthquakes focus.



**Figure 2.4 Movement of Seismic Energy Waves. (Image Courtesy of USGS)**

Surface waves radiate from the earthquake's focus and travel along the surface of the Earth. These types of waves are composed of Love and Rayleigh waves. Rayleigh waves take the form of a shape similar to an ocean wave as it travels along the Earth's surface, displacing it at right angles in the direction that the wave is moving. Displacement caused by Rayleigh waves and S-waves decreases with depth. Love waves displace the Earth's surface in a horizontal shaking motion and, like S-waves, cannot travel through water. Rayleigh and Love waves cause the most damage to structures and buildings because of their potential for extreme vertical and horizontal ground displacements. Rayleigh waves are the most destructive due to the possible magnitude of vertical displacement at the Earth's surface.

Seismic energy waves help to understand how earthquakes transmit energy through the ground and into a structure or building. The characteristics of seismic energy waves, how they are produced and the familiarity of regions most prone to earthquakes can allow engineers to design a structure or building capable of withstanding such induced forces. The understanding of this information is crucial to structural engineers and helps to improve structural designs.

### **Measuring Earthquakes**

Technology has greatly advanced over the past decades. Today special scientific equipment assists seismologists in determining the properties of an earthquake, within seconds of

when it has occurred. A number of ways to quantify the magnitude, intensity and the type of earthquake exist. Each method of measure of earthquake energy is based on several different variables. The types of earthquake recording devices used today, as well as the system used to quantify the magnitude or intensity levels, are discussed within this chapter.

### ***Earthquake Recording Devices***

A seismograph records vibrations produced during an earthquake based on the concept of inertia (Girty 2007). Inertia can be described as an objects resistance to movement. A seismograph consists of a dense, heavy object suspended from a wire. The object has such a large mass that when the wire is extended or contracted the object will move vertically and be horizontally stationary. Attached to the heavy object is a pen which then records the vertical motions, or vibrations, caused by the earthquake onto a slow rotating drum. Seismograms record the amplitude of seismic energy waves and the corresponding time at which the waves occur. These devices can also determine how far from the seismic station an earthquake occurred base on the time interval between the arrival of the P-waves and S-waves. By using the S-P time interval records from a minimum of three different seismic monitoring stations, the epicenter of an earthquake can be located. Hundreds of seismographs are placed around the United States in regions where earthquakes may potentially occur. However, today many seismographs are being replaced with more advanced, computerized, mechanical recorders.

### ***Earthquake Magnitude***

Earthquake magnitudes are a function of both amplitude and frequency. Frequency accounts for the number of waves that pass a given point every second. The amplitude recorded during an earthquake is a measure of the amount of energy released from the epicenter through the Earth's soil. Generally, the magnitude of an earthquake is measured based on the Richter scale. Charles F. Richter is credited for the invention of the Richter scale in 1934 (UPSeis 2007). Richter scales are logarithmic based and report values from 1 to 9. Each unit increase in magnitude has a ten-fold increase in amplitude. Additionally, every unit increase in the Richter scale has a thirty-two fold increase in energy. Another way to determine the intensity of an earthquake is from the seismic moment,  $M_0$ , the product of the faulted rock shear strength, the

area of ruptured surface, and the average displacement of the fault. The moment magnitude scale was introduced in 1979 by Dr. T. Hanks and H. Kanamori and believed to be more consistent than the Richter scale (UPSeis 2007).

### *Earthquake Categories*

Earthquakes can be divided into six categories based on their magnitudes as shown in Table 2.1. The majority of convergent earthquakes typically fall into the two largest magnitude categories; great and major. Transform boundaries on the other hand generally fall into the major and strong categories and divergent boundaries fall into the two lowest magnitude categories; light and minor.

**Table 2.1 Earthquake Categories Corresponding to Recorded Richter Scale Magnitudes.**  
(Table reproduced from Girty 2007)

CLASS	MAGNITUDE
Great	8.0 +
Major	7.0 - 7.9
Strong	6.0 – 6.9
Moderate	5.0 – 5.9
Light	4.0 – 4.9
Minor	3.0 – 3.9

Over 900,000 earthquakes each year throughout the world have a magnitude of less than 2.5 while 30,000 earthquakes per year are along the magnitude of up to 5.5 (Girty 2007). Others occur in smaller numbers (Girty 2007). Only 20 major and 100 strong earthquakes occur every year around the world. However, great earthquakes happen only every 5 to 10 years typically (Girty 2007).

### *Earthquake Intensities*

Earthquake intensities can be based on the Mercalli scale which reports values on a Roman numeral scale from I to XII. The Mercalli scale was invented by Giuseppe Mercalli in

1902 (UPSeis 2007). Similar to the Richter scale, the greater the numeral on the Mercalli scale the greater the intensity. The Mercalli scale measures the effects of an earthquake on people with a Roman numeral of VI or lower and the effects of an earthquake on buildings by a Roman numeral VII or higher. However, this scale depends on various factors including the local building codes at the time a building was constructed, the quality of construction, a structures distance from the epicenter, the type of soil beneath the structure and observations at the time of the earthquake (UPSeis 2007). The accuracy of such factors can vary greatly and may cause the Mercalli scale to be inaccurate.

## **Earthquake Hazards**

Earthquakes can result in substantial damage to structures as far as hundreds of miles away from the epicenter. The amount and type of damage depends on several aspects of the earthquake and structural elements. Some hazards associated with earthquakes occur more often than others. A few of the hazards discussed are not typically thought to be associated to with earthquakes. These hazards are essentially caused by manmade structures, not the occurrence of an earthquake. Without buildings, dams, and other structures or equipment, no hazards would exist, no buildings to fall down, dams to fail, or pipe lines to break. Since it is not humanly possible to control earthquakes or their occurrences, engineers must design structures to resist, transfer and withstand the hazards associated with seismic events. Hazards which cause large disarray and damage by earthquakes are further discussed in this section.

### ***Ground Shaking***

The dissipation of energy from the earthquakes focus causes ground shaking inducing extreme forces into the ground supporting buildings and structures. Once the ground is accelerated, energy is transferred into the building as a means of energy dissipation from the soil. When a building is displaced by seismic forces, energy travels into the structural members and connections within a building. These elements must be designed to absorb energy without failing. Additionally, the ground can settle, or subside, below a building causing damage to foundations and, if significant enough, soil subsidence may lead to structural collapse. Soil heaving or lurching, which vertically displaces the ground from its original position is also an

important characteristic to taken into consideration. Significant heaving or lurching may also lead to structural collapse. Another effect of ground shaking is liquefaction of the soil. Liquefaction is caused when ground water and soil are mixed by moderate to strong ground shaking causing soils below a structure to act similar to quicksand. This can lead to shifting, sinking, overturning or collapse of a structure. After ground shaking has subsided, liquefied soil returns to its hardened state. Landslides, avalanches and mudslides can also be a result of extreme or long duration ground shaking. This type of hazard was experienced in the Alaskan earthquake discussed in Chapter 3.

### ***Ground Displacement***

Displacement of soils along fault lines has a more extreme effect than ground displacement caused by ground shaking. This specific type of displacement occurs at or along a fault line at which an earthquake has occurred. Movement in these regions can be quite intensive and produce significant ground displacements. Buildings located along fault lines will experience much greater movement and seismic effects than buildings that are further away. Excessive vertical ground displacements were seen in the 1906 San Francisco earthquake.

### ***Flooding***

Flooding is one of the least severe and least occurring hazards associated with the occurrence of earthquakes. The cause for flooding is typically a result of another type of seismic hazard. Ground shaking and ground displacement may occur in locations near dams and levees or along rivers. These structures can be weakened by the earthquakes ground shaking or displacements and may lead to local failures or an entire collapse. When these critical structures are damaged it can lead to severe breaks allowing water to pass and flood nearby regions requiring evacuation.

### ***Tsunamis***

Oceanic earthquakes result in large, enormous waves, commonly known as tsunamis, which are generated from the movement of the oceans floor. This hazard is least likely to occur

but can be the most detrimental. Coastal regions are most affected by tsunamis, triggering flooding and evacuations. Tsunamis can also destroy structures because of the extreme energy and power released by waves as they hit land. Waves can also form on lakes; these are called seiches rather than tsunamis. This type of hazard occurs primarily in regions of Alaska and Hawaii.

### *Fire*

Fires can erupt when utility pipe lines are broken by extreme ground shaking. Earthquake forces can break main water lines that feed fire hydrants or damage fire hydrants used to put out fires. This hazard is not seen very often but has happened in the past and therefore still considered a potential threat. The 1906 San Francisco earthquake, discussed in Chapter 3, is an example of an event which produced extensive damage as a result of spreading fires.

## **CHAPTER 3 - Historic Earthquakes**

The world's earliest recorded earthquake was in China during the year of 1177 B.C. (USGS 1997). Most early earthquake documentations were not extensively detailed, possibly from the lack of understanding behind their occurrences. By the 17<sup>th</sup> century, the documentation of earthquakes became more detailed and descriptive and often exaggerated (USGS 1997). Earthquakes over the past several decades have become learning tools for building designers and building code writers. Research of seismic events and the corresponding funding grows substantially after a large earthquake occurs. Advancements and updates in building codes are developed based on common or substantial failures that were observed due to the event. Some earthquakes presented in this chapter are seen as significant events within the United States. The damage caused and the types of hazards that resulted are detailed to help convey the earthquakes impact. Other earthquake events were chosen simply because they demonstrate that seismic events can occur in regions typically unassociated with earthquakes or because of the hazards that resulted from the event. Each event was selected to show how the current development in building codes, hazard awareness, preparedness, and seismic mapping has improved. This section will further discuss some of the results of seismic research and what engineers, code officials, and many others within the industry ascertained from these events.

### **New Madrid Earthquake – 1811, 1812**

The New Madrid Fault, near New Madrid, Missouri produced some of the most widely felt and highly notable earthquakes. The first great earthquake in the series occurred on December 16, 1811 and estimated to have a magnitude of 8.0 on the Richter scale (USGS). On January 23, 1812 another earthquake occurred in the same area and on February 7, 1812, the last and most violent earthquake occurred (USGS). Between the sequence of earthquakes and after the last earthquake occurrence, aftershocks were continuously felt for months (USGS). It was recorded that the earthquakes were strong enough to be felt as far away as Denver (approximately 1,002 miles) and Boston (approximately 1,257 miles). The region where the earthquake occurred was not highly populated and did not result in a large loss of human life or

damage to buildings. One most notable effect on the surrounding land was the change in course of the Mississippi River. Today, an earthquake of this magnitude along the New Madrid fault would cause substantial damage and a significant loss of life because there is an increase in the number of residents living around the region. Historically, earthquakes in the Central United States are rare because they are located within the center of the North American Plate. Earthquakes in these regions are considered interplate faults and occur much less often than at the plate boundaries. Structures in regions surrounding those few high seismic prone areas in the Central United States may not have adopted building designs which are deemed substantial enough to withstand such a hazardous event. Additionally, code based design spectrum response curves do not depict actual earthquake ground motions but instead gives an idealized curve that depends on the design based earthquake (DBE) ground motions instead of the maximum consider earthquake (MCE) ground motions.

The New Madrid earthquake proved that earthquakes can happen at that interior of plates away from boundary regions. The CUSEC, or Central United States Earthquake Consortium, was formed in 1983 with funding and support by the Federal Emergency Management Agency (BSSC 2003). The main goals of CUSEC are to improve public awareness about earthquakes, state preparedness in terms of response and recovery aspects and earthquake reduction research. This organization was cofounded by several central states and includes Arkansas, Illinois, Indiana, Kentucky, Mississippi, Missouri, and Tennessee. Later in the 1990's the United States Geological Survey (USGS) and National Earthquake Hazards Reduction Program (NEHRP) suggested the expansion of earthquake research for the central United States (CUSEC 2008). Furthermore, in 1993 and through 1999 the CUSEC, with support from USGS, began establishing regional soil maps that show the amount of seismic hazard and ground shaking that can be expected over a certain period of time (CUSEC 2008). These seismic maps are used to aid in distinguishing practical earthquake regions from those regions of low potential hazards. Since then, substantial efforts have been made to design critical structures based on up-to-date code information and seismic standards to help decrease damage that could otherwise be catastrophic to such regions. Previous versions of the International Building Code (IBC) had based all seismic events in the United States on the same mean reoccurrence with the western United States having greater values than in the central United States. As a result, structures were under designed in high earthquake prone regions in the central North American Plate.

## **San Francisco Earthquake - 1906**

The Great earthquake of San Francisco, California in April 18, 1906 is noted as one of the most destructive ever recorded in North America (USGS 1997). The 7.8 magnitude earthquake ruptured along the San Andreas Fault causing large horizontal ground displacements with the greatest magnitude of approximately 18 feet (USGS 1997, BSL 2005). Violent shaking was felt in parts of southern Oregon, central Nevada and southern Las Angeles, nearly 500 miles away (USGS 1997). At that time, San Francisco was the most populous city on the West Coast (The Bancroft Library 2007). The earthquake caused a total loss of 28,188 buildings as recorded by the 1972 National Oceanic and Atmospheric Administration (NOAA) report. Underground pipe lines supplying water to the city were broken which let fires that spread over the city burn for days after the earthquake causing more damage than the earthquake itself (USGS 1997).

The 1906 California earthquake is not purely significant because of its magnitude, however, but for the amount of knowledge about earthquakes that was gained from the resulting research conducted. Large displacements and fault rupture length intrigued geologists and also helped lead to the formulation of the elastic-rebound theory of earthquake cycles by Harry F. Reid (USGS 1997). Geologists later formed the theory of plate tectonics which increased the understanding of the results of the 1906 San Francisco earthquake. After the earthquake occurred, scientists were quick to compile observations of the damaged aftermath. A State Earthquake Investigation Commission was developed to conduct such investigations for the earthquake. During the time, state funding was not available to conduct such research, funding was later provided by Carnegie Institution of Washington. Andrew C Lawson, professor in the geology department from the University of California, was notably know for his preparation and published documentation of the 1906 San Francisco earthquake. The publication of *The California Earthquake of April 18, 1906* set standards for investigating and researching of earthquakes to better understand their complexity (Schwartz 2006). In late 1906, the Seismological Society of America (SSA) was established with the devotion to the advancement of seismology and its applications in understanding the mitigating earthquake hazards and in imaging the structure of the earth.

## **Alaska Earthquake – 1964**

In March 27, 1964, one of the few great earthquakes not originating in California occurred in Alaska as one of the largest ever recorded in North America. This earthquake was ranked as a 9.2 moment magnitude (8.6 surface-wave magnitude or Richter magnitude) by the USGS and occurred along a subduction zone (Sokolowski 2008). Since the earthquake involved an oceanic plate, the sudden motion beneath the ocean caused tsunamis to occur and spread as far as the Hawaiian Islands (USGS 1997). Tsunamis that reached the Alaska lands resulted in nearly all the damage from of earthquake. Damage from the tsunami also occurred in regions of Oregon and California. Earthquake motions were felt over a significant portion of Alaska, parts of the western Yukon Territory and British Columbia in Canada (USGS 1997). The earthquake also resulted in the formation of seiches, landslides, and vertical soil displacements and liquefaction. Many homes and a few large buildings were destroyed or severely damaged. Additionally building facades and public utility services were significantly damaged.

The Alaskan earthquake caused the public unusual concerns not typically associated with earthquakes. The main cause of damage was from successive tsunamis, causing more potential threat than the earthquake itself. State and Federal officials determined after the 1964 earthquake that it was essential to provide tsunami warnings along with information about earthquakes for Alaska and much of the Northern Pacific (Sokolowski 2008). Requirements for tsunami warning devices to be produced by the West Coast & Alaska Tsunami Warning Center (WC&ATWC) for the states of Alaska, California, Oregon, Washington and the British Columbia in Canada were established following the Alaskan earthquake (Sokolowski 2008). The other consequential measures created from the Alaskan earthquake is for immediate report of earthquake information to the general public, media, National and International agencies and other State and Federal disaster preparedness agencies by the WC&ATWC (Sokolowski 2008).

## **San Fernando Earthquake – 1971**

The San Fernando earthquake in February 9, 1971, also known as the Sylmar earthquake, occurred along the San Fernando Fault located in California (SCEDC 2008). The thrust fault resulted in a 6.6 magnitude earthquake (SCEDC 2008). Ground shaking was felt around southern California, western Arizona and southern Nevada roughly 400 miles away (USGS

1997). Deaths were caused by the failure of critical and essential infrastructures. Several hospitals in the region collapsed or had severe damage including the Veteran's Administration Hospital and the newly constructed and supposedly earthquake-resistant Olive View Community Hospital (SCEDC 2008). Additionally, newly constructed freeway overpasses collapsed, two dams were damaged while others receiving minor damage and some buildings subsided or caught fire (USGS 1997). Some of the additional damage was caused by ground fracturing and landslides. One landslide around Van Norman Lakes was so severe that it took out nearly every structure and utility in its path (USGS 1997). It was reported that months after the earthquake aftershocks were still being felt in the region (USGS 1997). This earthquake was not a notably large earthquake on the Richter scale however it is known for the massive damage it did to a heavily populated area.

Building codes were revised in response to the 1971 earthquake. Later, in 1972, the *Alquist-Priolo Special Studies Zone Act* was passed which prohibited buildings for human occupancy to be located on the surface of active fault lines (USGS 1997). According to the State of California Department of Conservation, the Seismic Hazards Mapping Act was later passed in 1990 and addressed non-surface fault rupture hazards, liquefaction, and landslides caused by earthquakes.

### **Loma Prieta Earthquake – 1989**

The 7.1 magnitude Loma Prieta earthquake erupted on October 17, 1989 along the San Andreas Fault, the first major earthquake in the bay area since the 1906 San Francisco earthquake (BSL 2005). Two converging plates, specifically the Pacific plate moving over the top of the North American plate, initiated the earthquake (McNutt 1990). However, since this type of motion is not typical of the San Andreas Fault, the earthquake may have actually occurred on a sub-parallel fault rather than the San Andreas (BSL 2005). The Red Cross estimated that over 23,000 homes were damaged or deemed uninhabitable (McNutt 1990). Reinforced viaducts along the Nimitz Freeway, the Embarcadero Freeway, Highway 101 and Interstate 280 collapsed and liquefaction of the soil occurred in some areas causing damage to various other types of infrastructures (USGS 1997). Amplified seismic accelerations caused extensive damage to structures and contributed to liquefaction of soft, cohesive soils (USGS

1997). Some compressional deformation from the seismic energy traveling through the ground contributed to some of the more extensive structural damage northwest of the epicenter (USGS 1997).

Following the Loma Prieta Earthquake in 1989, extensive efforts have been made to better understand earthquakes and help minimize damage and losses (Page, et. al. 1999). The treat of earthquakes in the San Francisco region and other highly populated regions were reassessed after the 1989 earthquake to help adequately understand the motions of crustal plates and the causes of fault ruptures. This reassessment will help to strengthen future structures to be seismically resistive. The USGS and California Division of Mines and Geology (CDMG) developed state-wide shaking-hazard maps based on past research of faults and earthquakes in 1996. Shaking-hazard maps were composed of data which would produce the maximum magnitude of shaking over a 50 year period (Page, et. al. 1999). Such maps were later presented in the seismic provisions of newer building codes which were later published in 2000. Additionally, many organizations worked together to increase earthquake preparedness of high seismic regions such as the San Francisco Bay and surrounding area.

### **Northridge Earthquake – 1994**

The Northridge earthquake was one of the first earthquakes to occur directly beneath a largely populated area in the Unites States since the 1933 Long Beach earthquake (SCEDC 2008). The 6.7 moment magnitude Northridge earthquake created strong ground shaking on January 17, 1994 in Los Angeles, California along a thrust fault (SCEDC 2008). Wide spread damage to many major facilities and infrastructures occurred. Major freeways, parking structures and office buildings collapsed along with extensive damage that later left structures deemed uninhabitable. Not all buildings withstood the strong shaking and vibrations causing the collapse of building floors. Portions of 11 major roadways leading to downtown Los Angeles were closed due to damages they suffered (EQE 1994). Vertical and horizontal ground motions were noted causing uplift of buildings causing separation of the superstructure from their supporting foundations. The Northridge earthquake triggered many smaller fault ruptures along with thousands of large aftershocks. Landslides, permanent land deformations and fires caused additional damage to the region (USGS 1997). Building damage was reduced because some

buildings were strengthened prior to the earthquake. Additionally, many newer buildings in the area had been built to withstand earthquakes to a higher degree than as seen in earlier earthquakes.

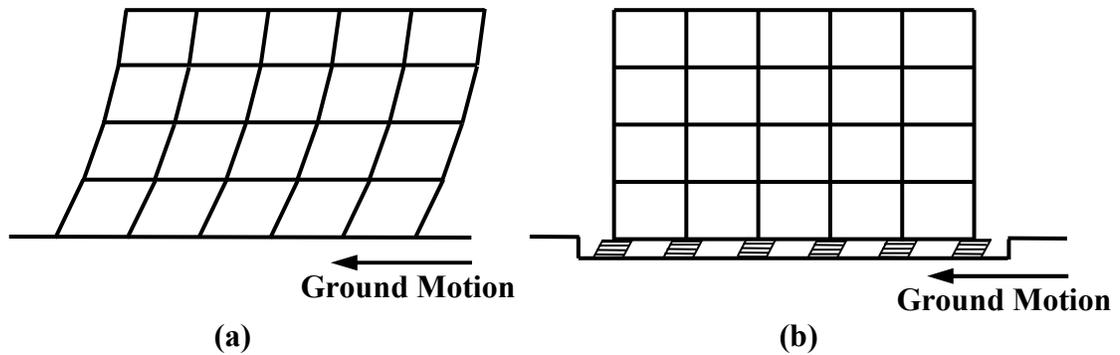
The Northridge earthquake, as a result of the type of damage caused instigated some of the most significant changes in the design code for buildings located in high seismic regions along the western coast line in the United States as well as small portions of central and eastern United States. Research of buildings performance during the earthquake provided a great deal of information about how buildings behave during such a significant seismic event. Prior to this earthquake, the primary focus of seismic design was the prevention of loss of life, not post-seismic structural sustainability. Many buildings were deemed unsafe to reenter and required demolition. A large number of buildings with significant damage were older and had not been designed or retrofitted to withstand such ground shaking. As a result the State of California has developed a program for older buildings requiring seismic upgrading for public buildings with large occupancies and/or high risk of failure (EQE 1994). ASCE 7 and IBC code revisions began offering levels of seismic design based on a structures importance for functionality after an earthquake and the type of occupancy in the building. Many failures were seen in flexible buildings, concrete framed buildings, concrete parking structures and concrete tilt-up structures (EQE 1994).

## **CHAPTER 4 - Traditional Non-Isolated Structures**

Non-isolated structures, known as fixed base structures are the most common building type. Current and past building codes for fixed base structures are established from many years of research which has resulted in the development of extensive analysis procedures and limitations. Earthquake damaged buildings have been extensively analyzed and their findings utilized to improve upon previous building design codes. Seismic design procedures vary for fixed base structures, based on the type of load distribution, location of the structure, type of occupancy, importance of the buildings contents and serviceability requirements of the structure after a damaging event, just to name a few. When it comes to seismic events, strict design requirements and a variety of codes and specifications must be cross referenced and often simultaneously satisfied. Structural engineers must verify their designs are adequate, and at a minimum, conform to the local building code. Currently, the requirements of the American Society of Civil Engineers, *Minimum Design Loads for Buildings and Other Structures*, ASCE 7-05 (ASCE 2006) are referred to in most model building codes for seismic design of fixed base structures. All references within this chapter, unless noted otherwise, refer to ASCE 7-05 with supplement No. 1.

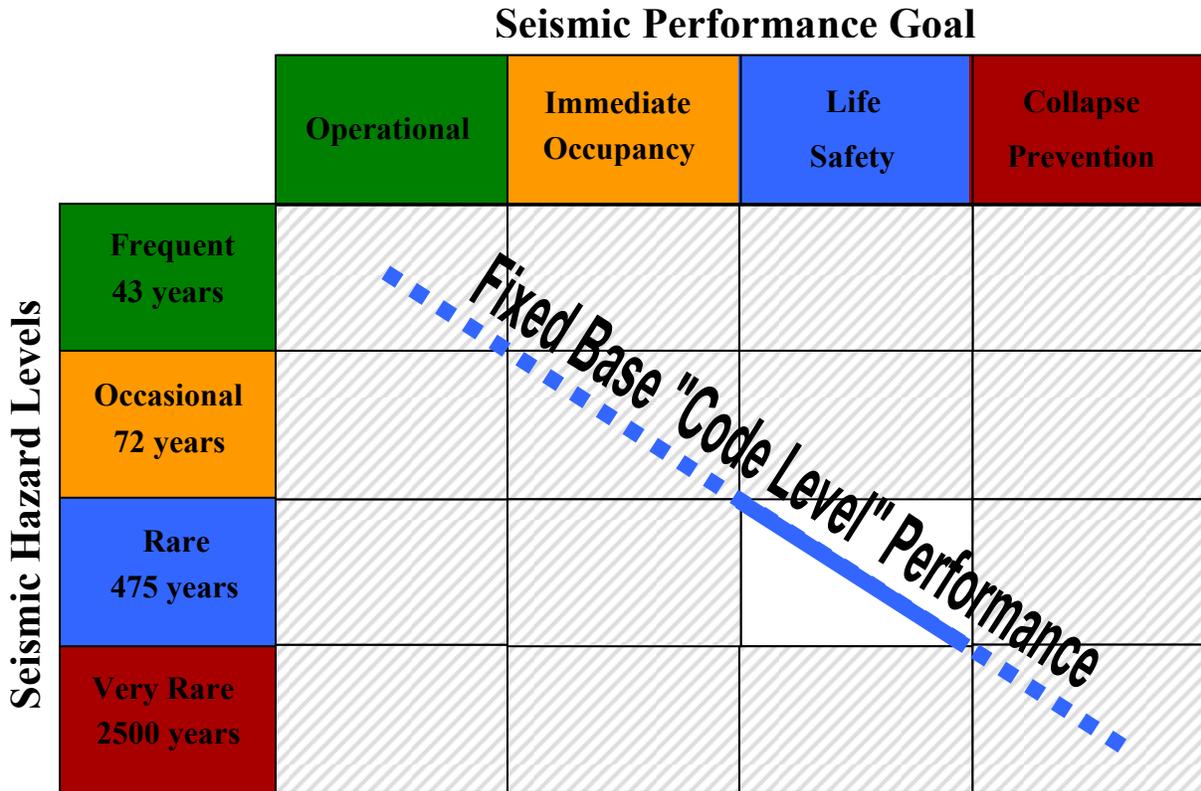
### **Seismic Force Transfer**

Typical fixed base buildings are continuous from top to bottom having a foundation bearing on the ground. Figure 4.1 illustrates the term “fixed-base” structure. Seismic waves traveling through the soil beneath a fixed base structure transmit energy directly into the superstructure through the foundation. Depending on the type of soil a building is founded upon, forces may be amplified or reduced. Energy waves are weakened when traveling through solid soils such as rock and amplified when traveling through soft, sandy materials. As seismic waves are transferred up from the ground and into the superstructure, forces are dissipated through deformation of the buildings rigid elements and connections. If seismic forces are extreme, excessive deformations can lead to building failure.



**Figure 4.1 (a) Superstructure Response of a Traditional Fixed Base Structure to Lateral Seismic Ground Motions (b) Base Isolated Structure to Lateral Seismic Ground Motions**

To resist intensive seismic forces, structural elements and connections must be specially designed in accordance with the parameters defined within ASCE 7. Traditionally, fixed base structures designed to withstand large ground forces produced by earthquakes consist of heavy, stiff elements. The more rigid the elements are the greater amount of force that is attracted to those elements and the stronger the connections must be in order to transfer forces between members. Large inelastic deformations are allowed to occur during a rare, significant seismic event. It has been deemed acceptable by ASCE 7, in these rare events, to allow actual forces to exceed the members' elastic capacity as long as the structure does not collapse (Kelly 1991). During low and moderate earthquakes, the structure is intended to remain elastic during a seismic event with a mean reoccurrence of 475 years or greater. The strength of a structure undergoing significant seismic forces is primarily dependent on its ability to withstand these inelastic deformations without immediate collapse (Sattary, Walters and Elsesser 1993). The building code only requires that a structure be designed for life safety and not collapse prevention as indicated in Figure 4.2.



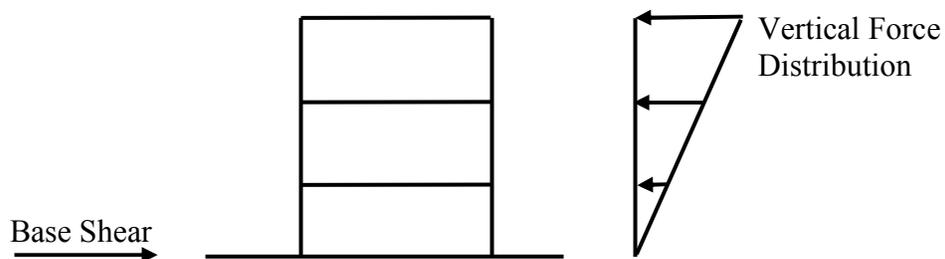
**Figure 4.2 Seismic Code Level Performance for Fixed-Base Structures. (Figure Reproduced from Andrew Taylor)**

Another way to deal with intense seismic loads is through overstrength in structural members causing loads to be attracted to areas with greater capacity and away from weaker elements where potential failures could occur (Sattary, Walters and Elsesser 1993). In high to moderate seismic regions, material specific (i.e. American Concrete Institute (ACI) 318, American Institute of Steel Construction (AISC), National Design Specification (NDS) for Wood Construction) building standards have certain types of connections and design guidelines that must be followed. The coined term, strong column weak beam, is supported by the building code to avoid the collapse of such critical members. If a beam were to fail, only the area directly above and below the beam would be damaged. The collapse of a column could cause the progressive collapse of all floors and elements that single element supported. Seismic procedures try to avoid progressive collapse, where failure of one element causes the failure of another element, similar to a chain reaction.

## Lateral Force Resisting Systems

Fixed base structures can be constructed using any type of structural materials; reinforced concrete, steel, timber, reinforced masonry, as long as their design meets current seismic code requirements. Lateral force resisting systems must be designed to connect to the remainder of the structure so that during a seismic event energy collected in the structure can be transferred to the lateral force resisting system. Lateral force resisting systems can be composed of braced frames, moment frames, shear walls or a combination of systems. Certain attributes for each type of lateral force resisting system may justify them as being more appropriate than others. Each system may have certain physical characteristics that deem them to be better suited than others to resist seismic forces. For instance, concrete shear walls must have special reinforcement detailing as defined by ACI 318 to encourage the wall to behave in a ductile manner. The lateral force resisting system selection may also depend on the type of structural materials being utilized. As an example, shear walls are not commonly designed using steel and braced frames are not commonly constructed of reinforced concrete. Other features such as the required rigidity of the lateral force resisting system may contribute to the design selection. Overall, each system and material has their ideal use and functionality.

Lateral force resisting systems are designed by computing the seismic base shear based on the site location, soil type and structural properties of the building materials. These design properties and limitations are commonly based on the specifications of ASCE 7. Base shear forces are then distributed at each story level above the ground as a factor of the weight at the floor and height above the ground for the level under consideration. The seismic forces increase significantly at a linear rate based on the height above the ground as shown in Figure 4.3. The greatest seismic forces are induced at the highest story level, and therefore, structural components must be relatively stiff to minimize interstory drift.



**Figure 4.3 Vertical Distribution of Base Shear Force at Each Floor Level.**

## **Diaphragm Design**

Another consideration in the design of a fixed base structure is the transfer of forces from the diaphragms through chords or drag elements and into the lateral force resisting system. Since seismic forces are first induced into the floor diaphragms, the forces must be transferred out of the diaphragms and into the lateral force resisting system. Since large accelerations can be imposed in the structure, these essential connections in the lateral force load path are required to allow inelastic deformations without extreme deformations that may lead to collapse or failure. This is accomplished so through overstrength in the structural members composing the lateral force resisting system to which the diaphragm is connected to or chord elements that transfer forces between the lateral force resisting systems.

## **Foundation Design**

Foundation design for structures having a fixed base may become very large in high or moderate seismic zones. The lateral forces transferred through the foundations require that they have a large enough surface area and weight to counter the applied shear forces through friction along the soil without sliding. Also, since structural elements for the lateral force resisting system in a fixed base design are significantly larger in size for high seismic regions the weight of the structure increases. This addition of structural weight also increases the gravity forces that the foundations must be designed to resist. In locations with weak soils, pile foundations or other such means are required to transfer vertical gravity loads into deeper soils that usually have greater strength. Since the greatest seismic forces will occur at the highest story, an enormous moment is created, causing possible overturning of the structure during a seismic event. Foundations must be designed to resist such overturning and sliding effects and are directly influenced by the type and strength of the supporting soils.

Connections between the foundation and the structure must also be capable of resisting the seismic base shear and uplift forces cause by overturning. Forces are transferred by a number of methods depending on the type of structural materials.

## **CHAPTER 5 - Introduction to Advanced Seismic Restraint Systems**

This chapter provides a basic description of base isolators, dampers, and hybrid systems commonly used to aid in the control of seismic energy. Although a variety of isolation and damping devices exist, only the most common are discussed. Each of these devices provides a type of advanced seismic restraint unlike that of a traditional, fixed base, design. Base isolators and damping devices require special considerations that are well advanced above what is traditionally observed as a means to reduce structural damage caused by earthquakes in fixed base building designs. Special considerations and design methodology is discussed in subsequent Chapters 6 and 7.

### **Base Isolation**

After nearly four decades of perfecting, structural base isolation is becoming a more common method for seismic design. In 1967 at a New Zealand Physics and Engineering Laboratory of the Department of Scientific and Industrial Research engineers began researching seismic isolation devices (Jacobs 2008). In the United States, development of elastomeric base isolators is primarily a result of the work of James M. Kelly at the University of California Berkeley (Jacobs 2008).

Base isolation seismic control devices vary in size, shape, element composition, degree of seismic resistance and many other properties. There are a number of different types of base isolation devices, however, only a few have been actually designed and implemented because of their complexity and/or unfamiliarity and lack of distributed research within the design field. Many other devices are under research, have not been extensively and adequately tested, or are purely envisioned ideas yet to be fulfilled. Primarily three isolator types are in use to date; elastomeric isolators, flat sliding isolators, and friction pendulum isolators. A few less common, but noteworthy systems will also be briefly described in this section to present further advances in seismic restraint technology. The fundamentals of base isolation along with their design and performance considerations are examined and discussed in depth in Chapter 6.

### *Advantages of Base Isolation*

A number of advantages to using base isolated devices exist. First and foremost, the degree of life safety and building protection associated with isolation. Occupants will be safe and less concerned knowing that the structure will not collapse beneath or on them. For owners and businessmen, it is crucial to know that sensitive equipment will not be damaged and that production will not cease during or after a seismic event. From the general public's point of view, it is relieving to know that in times of need, critical facilities such as hospitals will be functioning and operational. Base isolation offers an extensive degree of security and safety.

Another benefit is when seismic building codes are significantly changed and updated, a base isolated structure, which provides a design well above code minimums, will not require extensive seismic upgrades unless an extreme event demonstrates failure of such structures. This will eliminate any future costs needing to either replace any damaged parts of a building or make the structure capable of withstanding future seismic events.

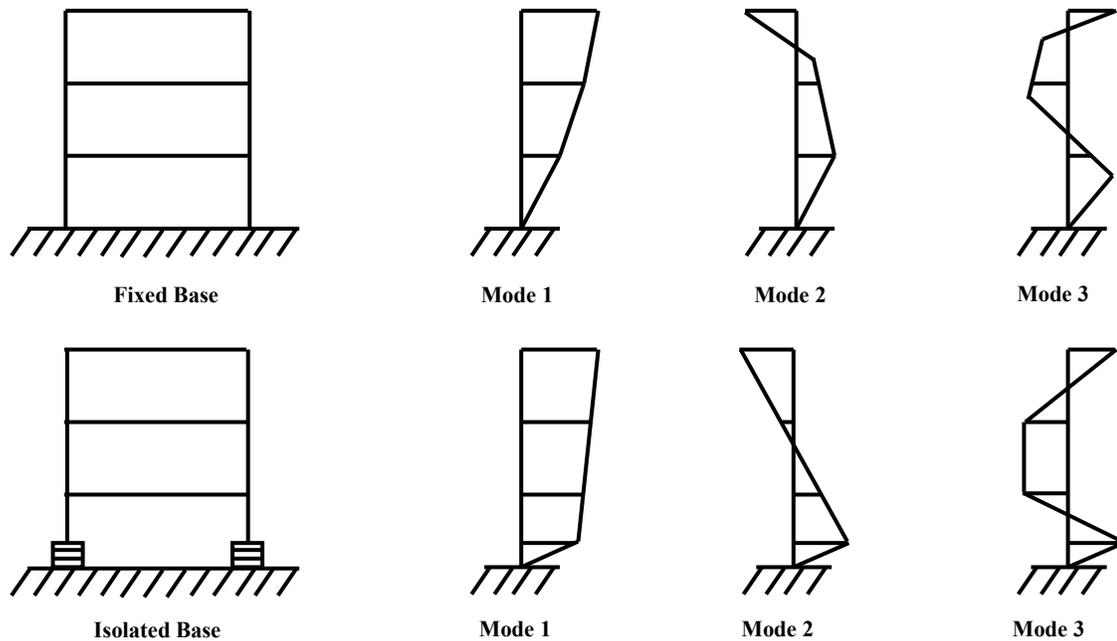
Base isolation can extend the predicted life of a structure after an earthquake. By having the capabilities of performing well during a seismic event, a structure's risk of collapse and extensive damage are greatly reduced. Some structures which are not base isolated may have such degradation after an earthquake that they must be demolished and rebuilt. The time and related costs associated with this are enormous in most cases and are not budgeted for prior to the seismic occurrence. However, an isolated structure could survive a seismic event with little to no damage. Such success results in decreased production down times and increased profit margins for operational facilities.

As a result of enhanced structural performance and life safety, insurance costs can potentially decrease for an isolated structure. The costs associated with endangering the lives of the structure's occupants are very costly when it comes to insurance. It would seem reasonable for insurance providers to recognize that there is less risk of personal injuries and be more apt to lower insurance premiums. This could become a cost savings if analyzed for the life of the structure. In addition, knowing that a structure will not collapse and threaten the lives of bystanders or damage adjacent buildings gives insurance agents reason to decrease insurance costs.

### ***Fundamentals of Base Isolation***

Isolators pose a fairly simplistic design concept, separating or decoupling the superstructure from the ground through which earthquakes induced energy waves are transferred to the structure. Figure 4.1 previously illustrated the general difference between the traditional fixed base structure and base isolated structure.

The main purpose of base isolation is to lengthen the structures fundamental period forcing significant deformations to occur at the level of isolation instead of within the superstructure (Sattary, Walters and Elsesser 1993). The component separating the structural elements from the ground surface is known as an isolator. Isolators decouple the structure from the ground and change the superstructures dynamic properties as well as its response to a seismic event. Throughout the duration of large seismic events, isolators inhibit energy from entering critical structural components. During the first dynamic mode the isolation system undergoes lateral displacement while the superstructure acts as a rigid element. The first mode is considered the isolation mode (Chopra 2007). The second mode, coined as the first structural mode, primarily entails displacement in the superstructure above the isolation interface. For a fixed base and isolated structure, Figure 5.1 illustrates the first three modal shapes. Mass participation during the second mode can be essentially neglected and does not directly affect the forces induced into the structure because nearly 100 percent of the mass participates in the isolation mode (Chopra 2007). A high percent of mass participation indicates that the structure is easily excited. When the total mass participation is around 90 to 95 percent or greater, it indicates that the vibrations are significant enough to depict the structures true reaction to the applied ground motions. Typically in most low rise structures, the mass participation decreases with increase in number of modes.

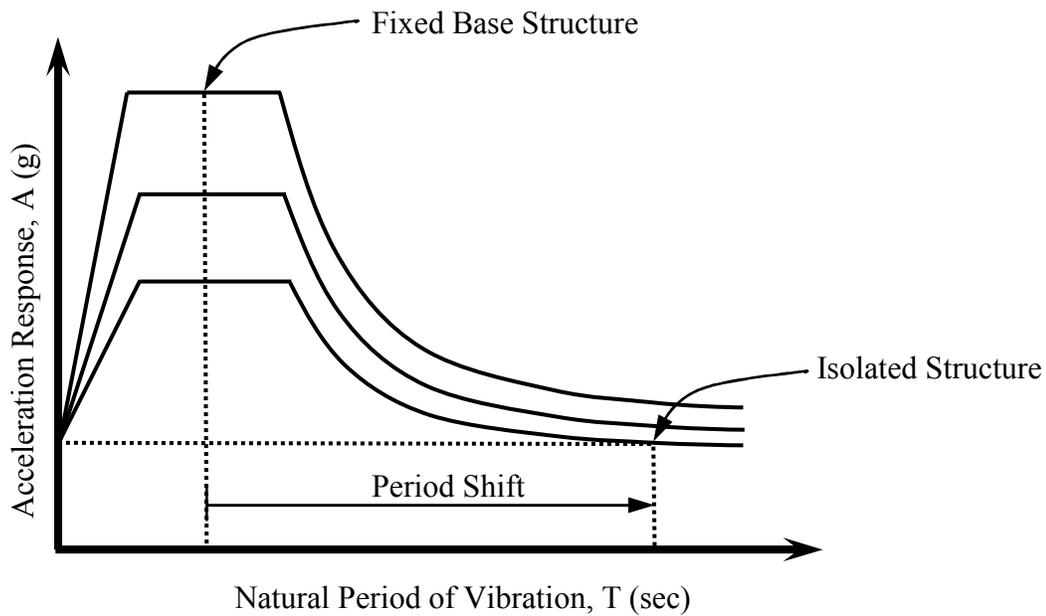


**Figure 5.1 Fixed Base and Isolated Base Natural Vibration Modes. (Image reproduced from Chopra)**

When the structure acts essentially rigid during the first dynamic mode, the natural period of the mode will only minimally differ from the isolation systems period. The slight change in the period of the first mode is due to flexibility in the superstructure even when it is acting as essentially rigid (Chopra 2007). This results in modal shapes reflecting those more similar to a fixed-base structure. Base isolation largely effects the natural period of the isolation mode (mode 1) and the structural mode (mode 2). However, base isolation has less effect on higher mode periods. During higher modes the motions at the base of the structure are comparatively less than the motions of the superstructure. As the motion of the superstructure increases during higher modes and the base motion decreases, the isolation system may no longer be engaged. This results in the base acting as though it were fixed instead of isolated. Chapter 9 of this report describes in detail the engagement requirements for the design of the base isolation system.

The first mode results in the lengthening of the fundamental period of vibration of the superstructure (Chopra 2007). By lengthening the fundamental period of the structure, the acceleration and seismic forces of the isolated structure are significantly reduced. Consequently, the building receives less seismic force, improving structural performance and decreasing non-structural damage. Isolators shift the structure from the peak response range of the acceleration

spectrum to the low response range. This shift reduces seismic forces induced into the structure (Guh and Youssef 1993). Figure 5.2 shows the change in response between a fixed base and isolated building. A fixed base structure normally has a natural period of vibration within the peak portion of the design spectrum. Isolation systems tend to lengthen the structures natural period to a region beyond the peak region, where earthquake motions provide lower accelerations and as a result, smaller seismic forces in the structure. The amount of shift in period and the degree of pseudo-acceleration reductions depend significantly on the shape of the earthquake design spectrum. Realistic earthquake design spectrums are plots of ground motions produced by an earthquake for certain regions. Code based design spectrums have been adjusted to eliminate any significantly large energy waves that may require impractical design requirements. If large spikes of ground motion occur at lower ranges of acceleration, the isolation system may not act as efficiently, since the design spectrum curve no longer fits such an ideal curve.



**Figure 5.2 Comparison of Fixed Base Structure Period of Vibration and Pseudo-Acceleration versus an Isolated Structure. (Image reproduced from Chopra)**

As previously stated, to reduce the base shear forces transmitted into the structure, the natural vibration period of the isolated building should be longer than the natural period of the

same building analyzed as a fixed-base structure as shown in Figure 5.2. If the frequency of the ground motions and the superstructure are equivalent, the structure will be induced with resonance. When the buildings natural frequency, or period, is relatively the same as or equal to the frequency of the seismic waves, the building may reach resonance, and possibly collapse. Coinciding frequencies, or periods, result in violent oscillations that move in harmony and amplify seismic energies. As the difference between the fixed-base and isolation vibration period increases the greater the isolation performance. However, at a certain range, as the natural period of vibration of the superstructure continues to increase, the structure will begin to behave less rigidly during the first mode. Larger periods of vibration are commonly associated with tall and flexible structures. If the superstructure behaves in a flexible manner during the first mode, the second mode effects of the structure are no longer negligible because the amount of mass participation in the first mode is reduced and a greater participation than for a rigid superstructure occurs in the second mode (Chopra 2007).

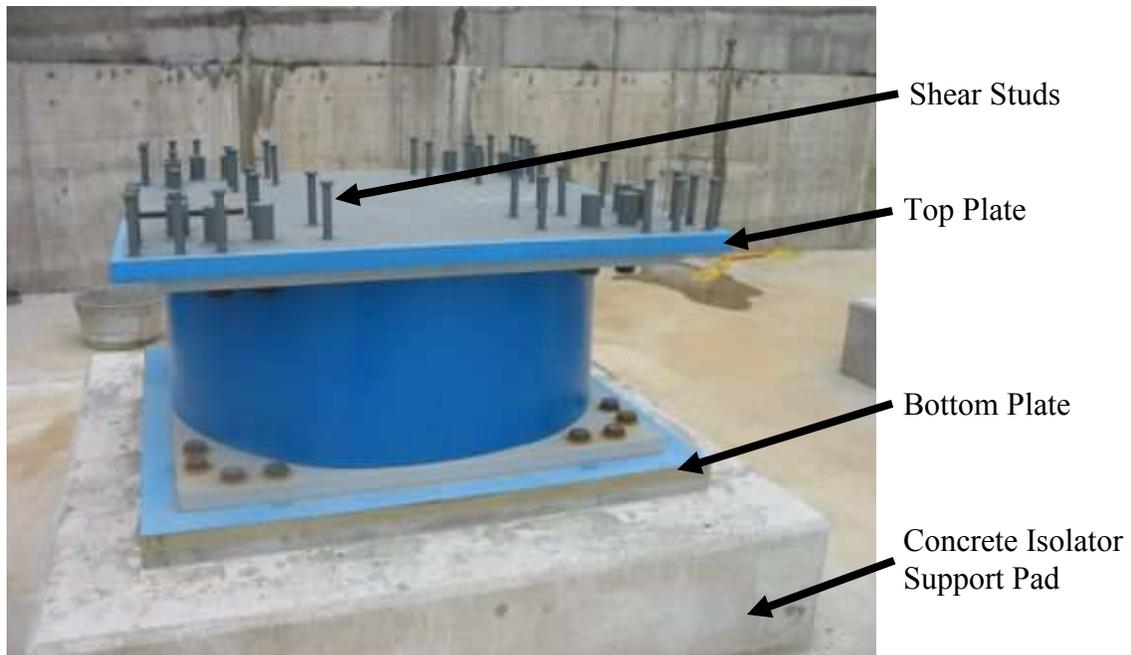
In addition, isolators must be able to resist vertical (gravity) loads while carrying occupancy loads before, during and after a seismic event. Simultaneous occurrence of large vertical loads and extreme horizontal seismic loads requires that isolators be flexible, to allow for horizontal movement while maintaining vertically stiff properties. The elastomeric and sliding isolators discussed in this chapter have been designed to resolve these requirements.

## **Types of Base Isolation Systems**

### ***Elastomeric Isolators***

Modern elastomeric isolators consist of layers of rubber and steel plates alternately orientated and bonded together (Taylor and Igusa 2004). The steel plates, or “shims”, provide vertical stiffness and resist the tendency of rubber to bulge near the edges (Taylor and Igusa 2004). Rubber layers made of natural or synthetic material provide the horizontal flexibility required to allow the ground below the isolators to move independent of the superstructure. These steel and rubber elements are combined into a single system through chemical bonding and heat or pressure curing (Taylor and Igusa 2004). Cyclic testing of isolation devices required as indicated by ASCE 7 verifies that these components will not deform excessively or separate during the activation and dissipation of earthquake energies. The properties of seismically

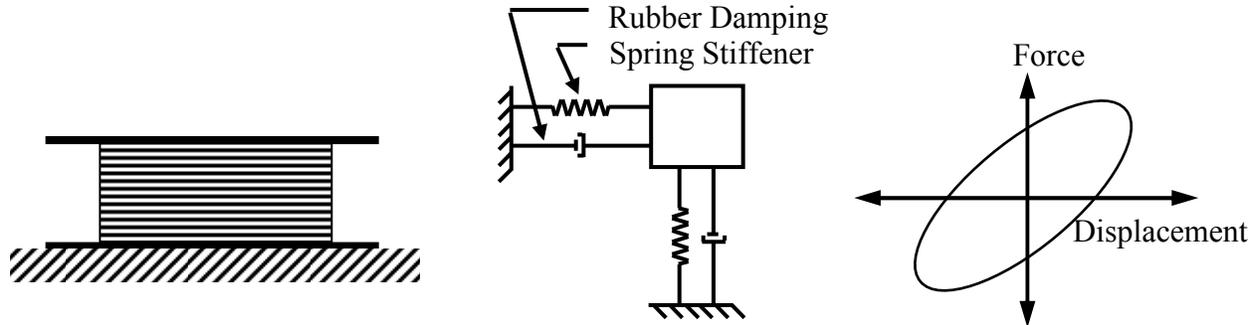
induced energies are adjusted through the isolation device, relieving the amount of energy transferred into the structure, reducing structural story drifts and decreasing the natural frequency (lengthening the fundamental period) of the isolated superstructure. Refer to figure 5.3 for a photograph of an elastomeric isolator. An interior cross-section of an elastomeric isolator is similar to that shown in Figure 5.11, except without the energy dissipation lead core.



**Figure 5.3 Elastomeric Isolator While Under Construction. (Photo courtesy of Andrew Taylor, KPF Consulting Engineers)**

The rubber properties of the isolator allow it to naturally realign to its original position after the earthquake. This is beneficial because no additional components are required to be designed for the system for realignment. A schematic diagram showing the dynamic elements and the ideal force-displacement curve for an elastomeric base isolation unit is shown in Figure 5.4. The schematic diagram represents the various components that work together to provide stiffness and flexibility in the system. The rubber damping represents the rubber layers in the elastomeric isolator and the small amount of damping that can be provided in the horizontal direction through the rubber material. Additionally, the spring stiffener represents the horizontal flexibility that is created by the rubber layers. Similarly, the vertical stiffener and damping

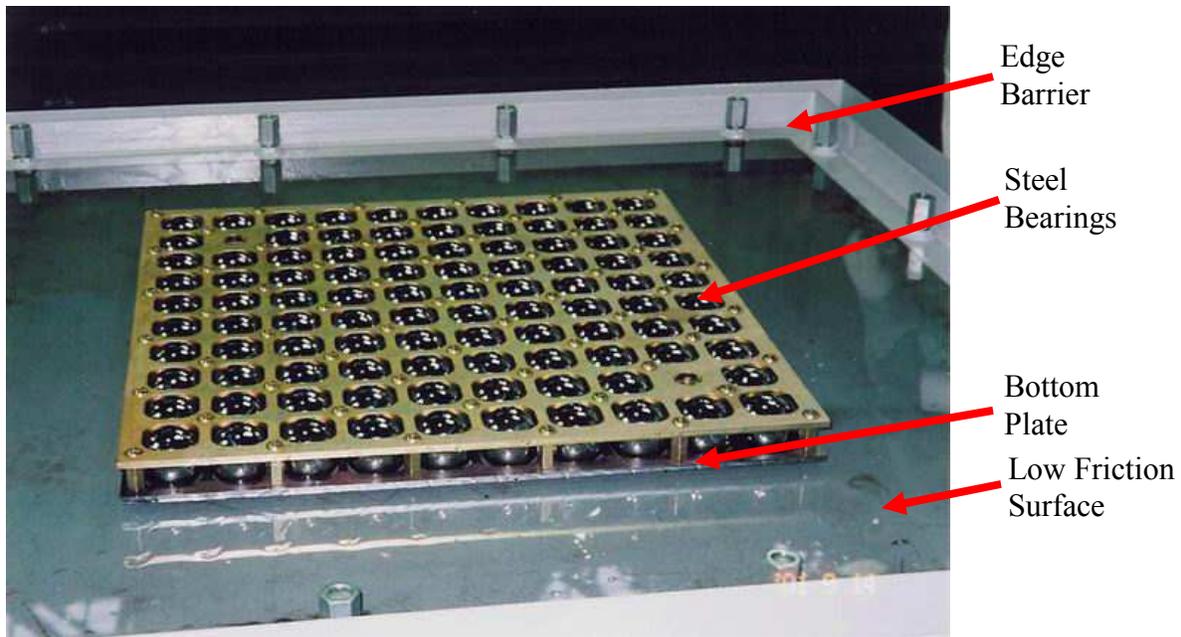
corresponds to the vertical stiffness in the steel plates and the damping provided by the combination of steel and rubber layers, respectively.



**Figure 5.4 Isolator Section, Schematic Diagram and an Ideal Force-Displacement Curve for an Elastomeric Unit. (Image reproduced from Matsagar)**

### *Flat Sliding Isolators*

Flat sliding isolators incorporate two elements that move and rotate within surface boundaries, allowing the building to move differently than the foundation it bears upon. Flat sliding isolators are most commonly found beneath lightweight structures since the devices are not activated through the weight of the superstructure. Sliding isolators typically incorporate a stainless steel plate which bears against an opposing low friction surface commonly made of polytetrafluoroethylene (PTFE) (Taylor and Igusa 2004). Friction imposed between the PTFE coating and the stainless steel surface results in energy damping. The coefficient of friction is dependant on the pressure on the surface and velocity of the sliding device undergoing movement. Additionally, the use of PTFE allows for a slower slip rate during activation of the isolation device. PTFE is utilized because of its corrosion resistance and to provide an extremely low coefficient of friction. An example of a flat sliding isolator is shown in Figure 5.5.

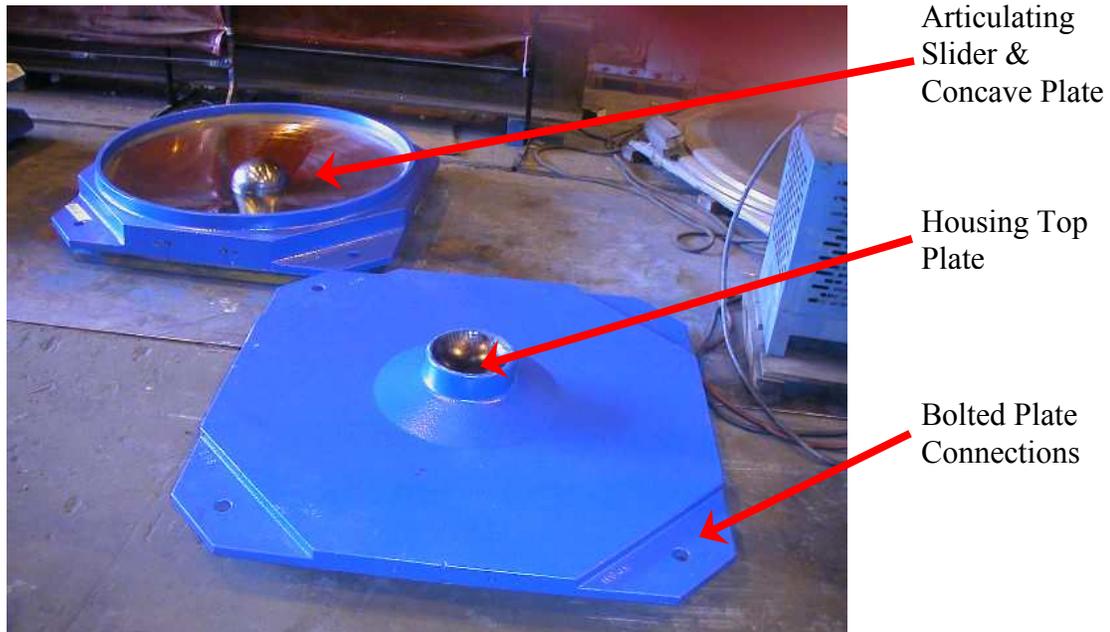


**Figure 5.5 An Example of a Type of Flat Sliding Isolator, Sliding Ball Bearings. (Photo courtesy of Andrew Taylor, KPFF Consulting Engineers)**

As the ground below the sliding isolator accelerates, the low friction property at the interface lowers the amount of energy transferred into the superstructure. Similar to that of elastomeric isolators, flat sliding isolators decouple the building from the foundation. However, flat sliding isolators do not have natural self-centering capabilities to realign the device to its original position after a seismic event. To create a system that is capable of re-centering itself, a few concepts have been incorporated in current isolator design. One solution is to place an edge barrier around the sliding isolator to limit its degree of maximum displacement. Along the isolator edge springs are distributed to minimize the impact forces between colliding elements. The springs then create a counterforce that helps to re-center the isolator device to its original position. Additionally, sliding isolators are designed to only function when a force is applied equal to or greater than the dynamic coefficient of friction. Sliding isolators have “stick and slip” phases, which means that the slider is locked by friction until a force is applied that overcomes the break-away friction force when sliding will occur (Sattary, Walters, and Elsesser 1993). This means that under wind loads and fairly minimal lateral seismic loads, the buildings isolators will not slide or move, instead the structure will act as a fixed base structure.

### *Friction Pendulum Sliding Isolators*

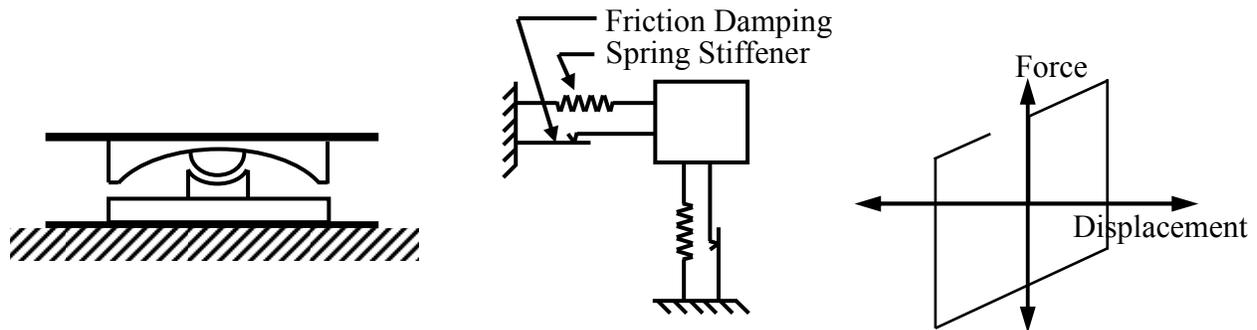
Friction pendulum sliding systems take the form of a spherical concave surface with an articulated slider (Taylor and Igusa 2004). The elements of a friction pendulum sliding isolator can be seen in Figure 5.6.



**Figure 5.6 Elements of a Pendulum Friction Slider Used in the Pioneer Courthouse in Portland, Oregon. (Photo courtesy of Andrew Taylor, KPFF Consulting Engineers)**

Pendulum friction sliders are very similar to flat sliding isolators. As a seismic event occurs, the articulated slider moves along a concave surface, moving the superstructure in the motion of a pendulum sway. This type of system incorporates a self-centering feature, placing the building back in its original position. The articulated slider can be a concave or convex surface, both acting in a similar fashion. For design purposes, the period of oscillation becomes a function of the radius of the concave surface and is independent of the mass of the superstructure because of the low friction interface (Taylor and Igusa 2004). The properties and functionality of the pendulum friction slider are relatively similar to flat sliding isolators which were previously been discussed. Figure 5.7 is a schematic image of the dynamic elements making up a pendulum friction slider along with the ideal force-displacement curve. The schematic diagram represents the horizontal function of the friction damper along the interface

surface and the stiffness of the system based on the concave or convex surface. Similarly, the vertical damping and stiffener represent the same features of the friction pendulum slider but in the vertical direction rather than horizontal.



**Figure 5.7 Isolator Section, Schematic Diagram and an Ideal Force-Displacement Curve for a Pendulum Friction Sliding Unit. (Image reproduced from Matsagar)**

### Structural Damping

The purpose of damping is to convert seismic energy into work that is then controlled by energy dampers, similar to shock absorbers on automobiles. This device works similar to a piston converting heat into work. Additionally, dampers aid to reduce, or dissipate, large amounts of energy that would typically be induced into the superstructure. Damping devices are not commonly used by themselves but usually in conjunction with base isolation devices. In earlier designs, damping systems have been used in tall buildings to aid in control of lateral wind forces, but today they are being utilized for seismic control also.

Damping systems are normally considered a secondary or supplemental control system. Common dampers are categorized as fluid viscous (hydraulic) or oil dampers, friction dampers, viscoelastic dampers, or yielding metal dampers. These dampers will be further discussed in the following paragraphs.

## *Fundamentals of Damping*

Inherent damping allows a structure induced with seismic forces to naturally dissipate a small portion of the absorbed energies. Energy is dissipation through structural and non-structural elements. This results in deformation of primary structural members, which in turn, causes damage to secondary structural and non-structural elements. The inherent damping for a fixed-base structure is assumed to be equal to 5%. For an isolated superstructure, the inherent damping decreases to roughly 2%. These values differ because of the high performance level obtained from base isolation. One primary goal of base isolation is to significantly reduce structural and non-structural damage during large earthquake motions. In doing so, less energy is dissipated into the buildings structural and non-structural components as compared to a fixed base structure. Instead, the energy is dissipated into the isolation elements, resulting in less inherent damping of the superstructure.

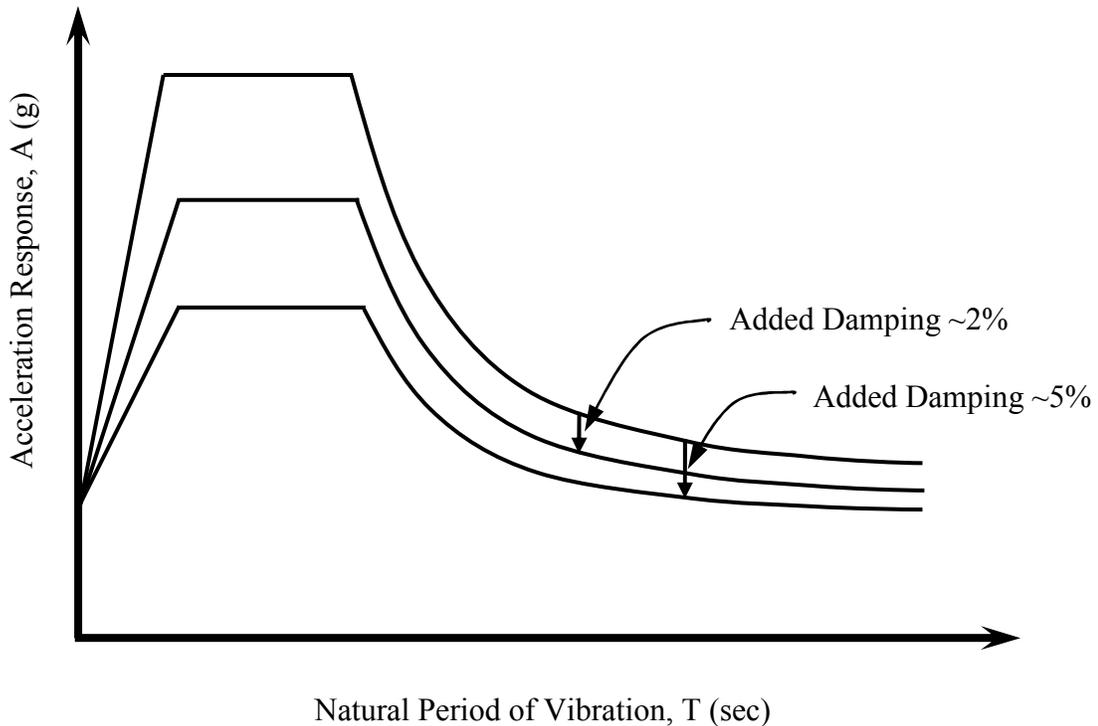
Elastomeric isolators also have an inherent damping characteristic since they are made of natural or synthetic rubbers. Base isolators have a damping capacity much greater than that of the superstructure. It is possible to develop additional damping from the elastomeric material by using special compounding agents to combine the elastomeric layers (Buckle and Liu 1993). Damping can also be increased in the isolation system through the use of high damping rubber materials. Another way to increase the amount of damping at the base of the building is to add a lead core to the center of the elastomeric isolation devices. The lead core will yield and dissipate a significant amount of energy induced into the elastomeric isolators.

Structural damping provides a method of energy dissipation through devices which in turn produce work or heat. These elements response similarly to that of a mass attached to a spring. As the body of mass moves, the damping element absorbs the energy by inducing a force on the spring and the spring correspondingly deforms. This deformation of the spring results in energy dissipation. The damping elements discussed in this chapter based on their means of energy dissipation. Some damping devices dissipate energy through the heating of liquid, such as the fluid viscous damper.

The properties of damping behave differently depending on the dynamic modes under observation. Since the superstructure acts as a rigid body during the first mode period it contributes very little to the modal damping (Chopra 2007). Damping induced in the first modal response is equivalent to the amount of damping provided by the isolation system or by separate,

independent devices. The damping in the first structural mode, also known as the second mode, is increased based on the level of damping provided within the superstructure. However, at higher modes, damping of the superstructure has less overall effect.

Figure 5.8 shows the effect that damping has on the design spectral response curve. As the amount damping is added to a structure, the design spectrum curve is shifted downward. The shift in the spectral response results in decreased design accelerations and lower forcing demands placed on the structure.



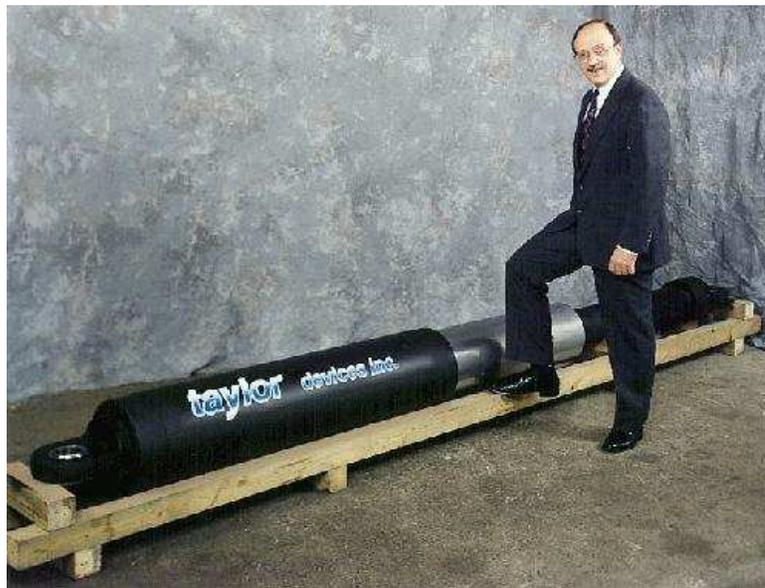
**Figure 5.8 Effects of Damping on the Design Spectral Response. (Image reproduced from Chopra)**

## **Types of Damping Systems**

### ***Fluid Viscous Dampers***

Fluid viscous, also known as hydraulic or oil dampers are designed to dissipate energy by applying a resisting force over a finite displacement (Constantinou 2008). To best understand

the complicated behavior of a fluid viscous damper, a thorough knowledge of fluid dynamics is required, therefore only the basic principles will be described in this report. As previously mentioned, energy dissipation is accomplished through heat transfer. When a seismic event occurs, energy is absorbed by the fluid causing it and other components to heat (Constantinou 2008). To completely dissipate energy from damping elements, convection and/or conduction must occur to release this buildup of heat in the system. Fluid viscous dampers can be best used to reduce the deflections and column stresses in a structure in addition to dissipating seismic energies (Constantinou 2008). Fluid viscous dampers can be small and compact, requiring no special equipment for them to function. Some of the first building applications of fluid viscous dampers were in the San Bernardino County Medical Center Replacement Project. However, their very first applications were not in buildings but were in shock isolation for military hardware (Constantinou 2008). Early applications transpired around the late 1800's and were utilized as a way to ease the recoil force produced from large cannons (Constantinou 2008). A photo of a large fluid viscous damper is shown in Figure 5.9, designed by Taylor Devices Inc.



**Figure 5.9 Taylor Devices Inc. fluid viscous (hydraulic) damper. (Photo courtesy of Taylor Devices Inc.)**

Fluid viscous (hydraulic) dampers are commonly constructed to act as part of a braced frame. Other practical locations for the placement of this type of dampers are in series with

isolation systems along the perimeter of the superstructure so that the isolation device and damping device move in conjunction with one another. This is similar to how an elastomeric isolator with a lead core functions together; this is described in detail later in this report.

### ***Friction Dampers***

Friction dampers are another type of damper and can be related back to the flat sliding and pendulum friction sliding isolators discussed earlier. This type of damping system is used to reduce or even eliminate energy transferred into the superstructure from the seismic ground motions. Damping is achieved simply through the friction of the sliding elements, similar to disc brakes on an automobile. Friction dampers are designed so they will not slip during smaller lateral forces, such as low wind loads or minor earthquakes. At increased lateral forces, friction dampers are design to slip just before yielding of the structural elements. Energy is dissipated as heat is built-up in the damper and removed through convection and/or conduction similar to fluid viscous (hydraulic) dampers. Unlike hydraulic dampers, friction dampers can be used alone rather than in conjunction with base isolation devices.

Friction dampers can be made out of any combination of steel elements coupled together so that the elements slide, one over the other. The photo shown in Figure 5.10 illustrates one type of application for friction dampers. This system is composed of a series of plates clamped together using high-strength bolts. The friction between the plated connections absorbs energy by friction and dissipates it as heat.



Friction Damping Element

**Figure 5.10 Pall Dynamics Friction Damper Used In Conjunction as a Cross Brace. (Photo Courtesy of Kinh H. Ha, Concordia University Department of Building, Civil & Environmental Engineering Professor).**

### *Yielding Metal Dampers*

The last type of damper that will be discussed is yielding metal dampers. This type of energy damper is more simplistic when compared to other damping systems. This type of damping system is designed to dissipate energy through inelastic hysteretic behavior of steel or lead elements, whether they are part of the superstructure, or part of a connecting element designed specifically to dissipate energy. Different metal types can be utilized as a way to obtain the desired level of damping. Yielding metal dampers can be used in conjunction with various base isolators as a way to absorb additional energy as a means of increasing an isolator's effectiveness to reduce seismic forces.

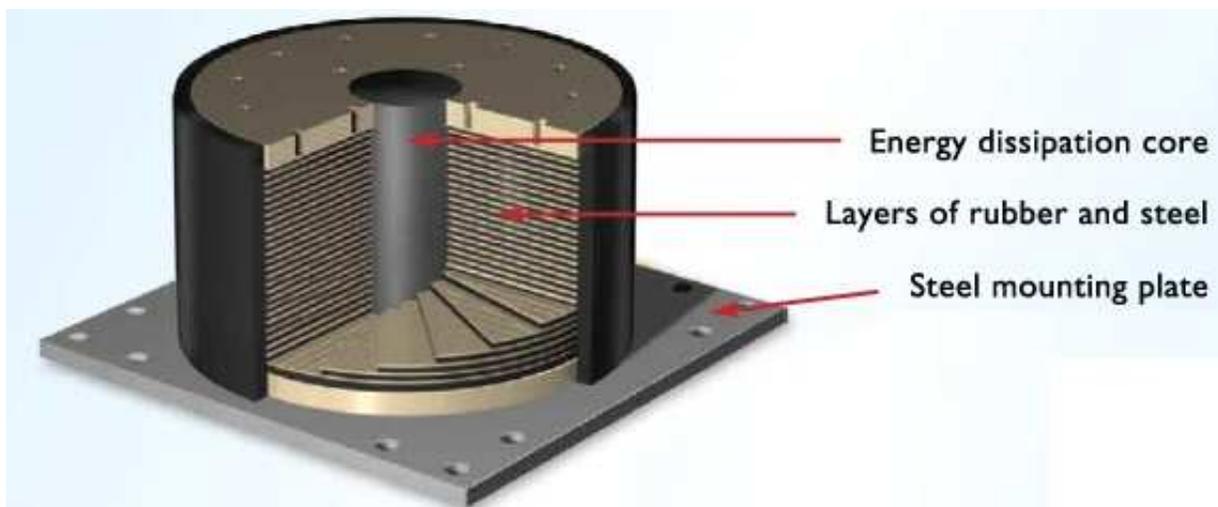
### **Hybrid Systems**

Hybrid systems presented in this section will focus on the combination of isolation and damping systems. Independently, isolation systems and damping systems work quite well. Engineers have decided to use these different characteristics to their advantage by using them

together in one building to dissipate energy, reduce seismic forces and therefore resulting in reduced damages.

### ***Lead Core Elastomeric Isolators***

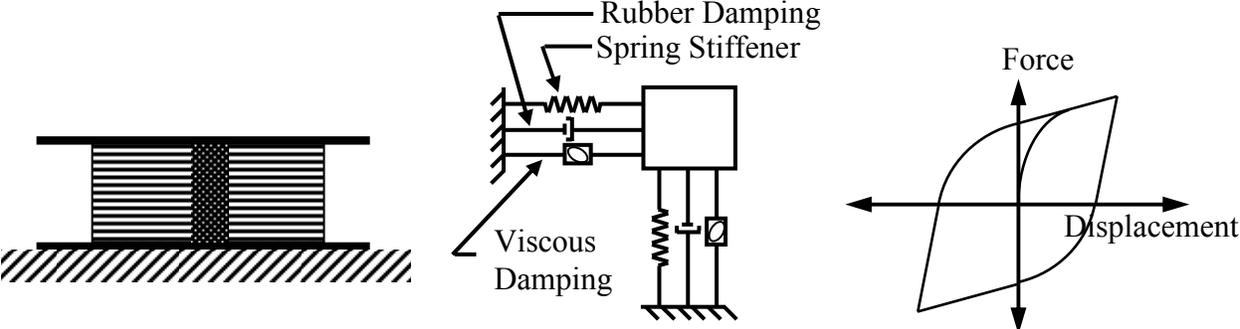
One type of hybrid system incorporates a damping device within an elastomeric isolator to help dissipate energy absorbed into the isolator. This type of system is termed viscoelastic dampers. The actual damping component is composed of a core made of lead encompassed by an elastomeric isolator. Elastomeric devices, as described previously, are composed of multiple alternating layers of steel plates and high strength rubber that are bonded together. A cross-section of a common lead core elastomeric isolator is shown in Figure 5.11.



**Figure 5.11 Cross-section of an Elastomeric Isolator with a Lead Core Center. (Image courtesy of Dynamic Isolation Systems)**

This system has two vital aspects; an increase in axial stiffness and immense horizontal flexibility. The lead central core or viscoelastic damper is used to absorb a significant amount of the energy induced into the isolators. Lead is utilized because its elastic-plastic post-yielding capacity helps maintain strength during plastic deformation cycles caused during a seismic event (Taylor and Igusa 2004). A schematic diagram showing the dynamic elements and the ideal

force-displacement curve for an elastomeric base isolation unit with a lead core is shown in Figure 5.12.



**Figure 5.12 Isolator Section, Schematic Diagram and an Ideal Force-Displacement Curve for an Elastomeric Unit with a Lead Core. (Image reproduced from Matsagar)**

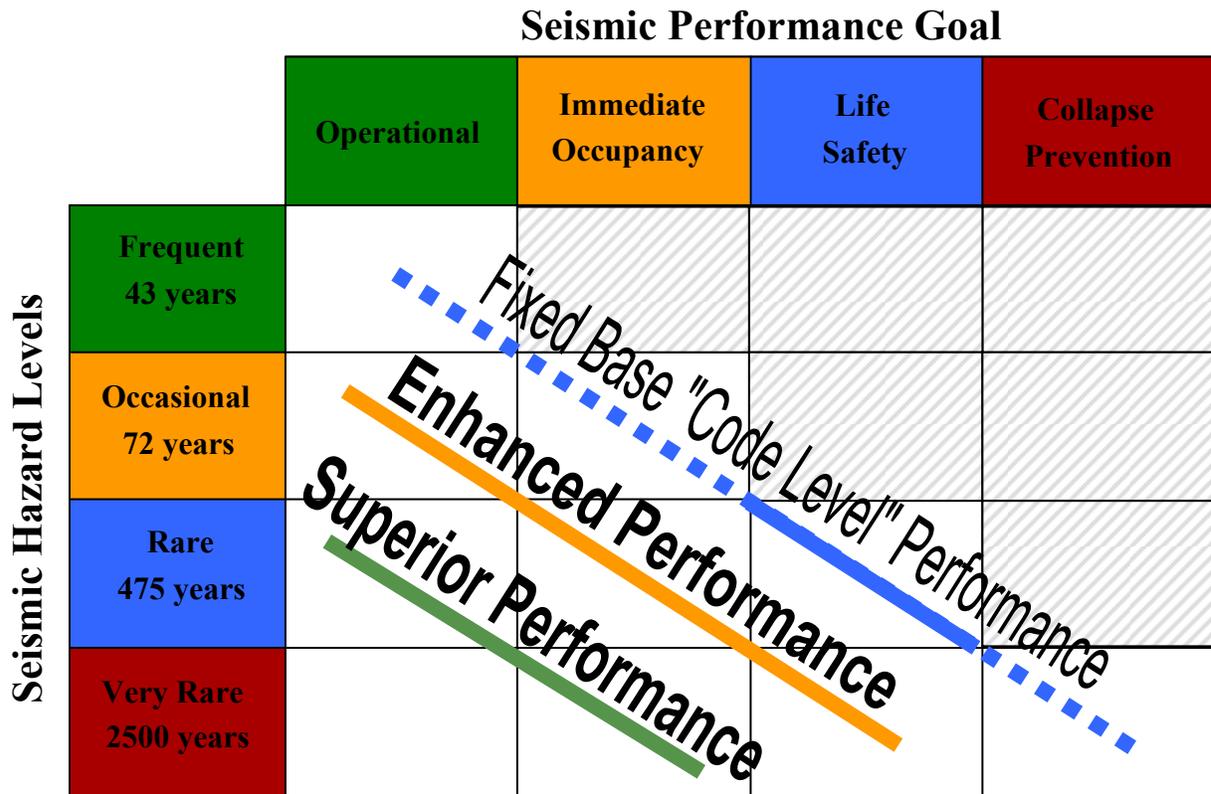
## **CHAPTER 6 - Base Isolation Design Requirements**

The 2006 International Building Code (ICC 2006) indicates that the current edition of the American Society of Civil Engineers/Structural Engineering Institute, 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 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.

### **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, as discussed in Chapter 5, 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 6.1 illustrates the different seismic levels of performance that a structure can be designed for and the type of performance

goal that can be achieved. Such extreme performance measures allow a structure to be immediately occupied and also operational after a design based and even a maximum considered seismic event. These performance standards insure life safety and collapse prevention.



**Figure 6.1 Comparison of Seismic Performance Levels for Conventional Fixed-Base Structures and Base Isolated Structures. (Figure Reproduced from Andrew Taylor)**

### 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:

- $S_I$ , the mapped maximum considered earthquake spectral response acceleration parameter, is less than or equal to 0.60g
- Site classification investigation of the soil indicates it is Class A, B, C, or D
- Height of the superstructure above the isolation interface is less than or equal to four stories or 65ft (20 m)
- Effective period of the isolated structure is less than or equal to 3.0 sec at maximum displacement,  $T_M$
- 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
- The superstructure is of regular configuration with a symmetrical building plan having no vertical out-of-plane offsets causing a complex structural design
- 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 6.2 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.

### ***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 6.2. Additionally, the limitations placed on the ELFP by ASCE 7, are listed in the previous section. Figure 6.2 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 be designed. Figure 6.3 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.

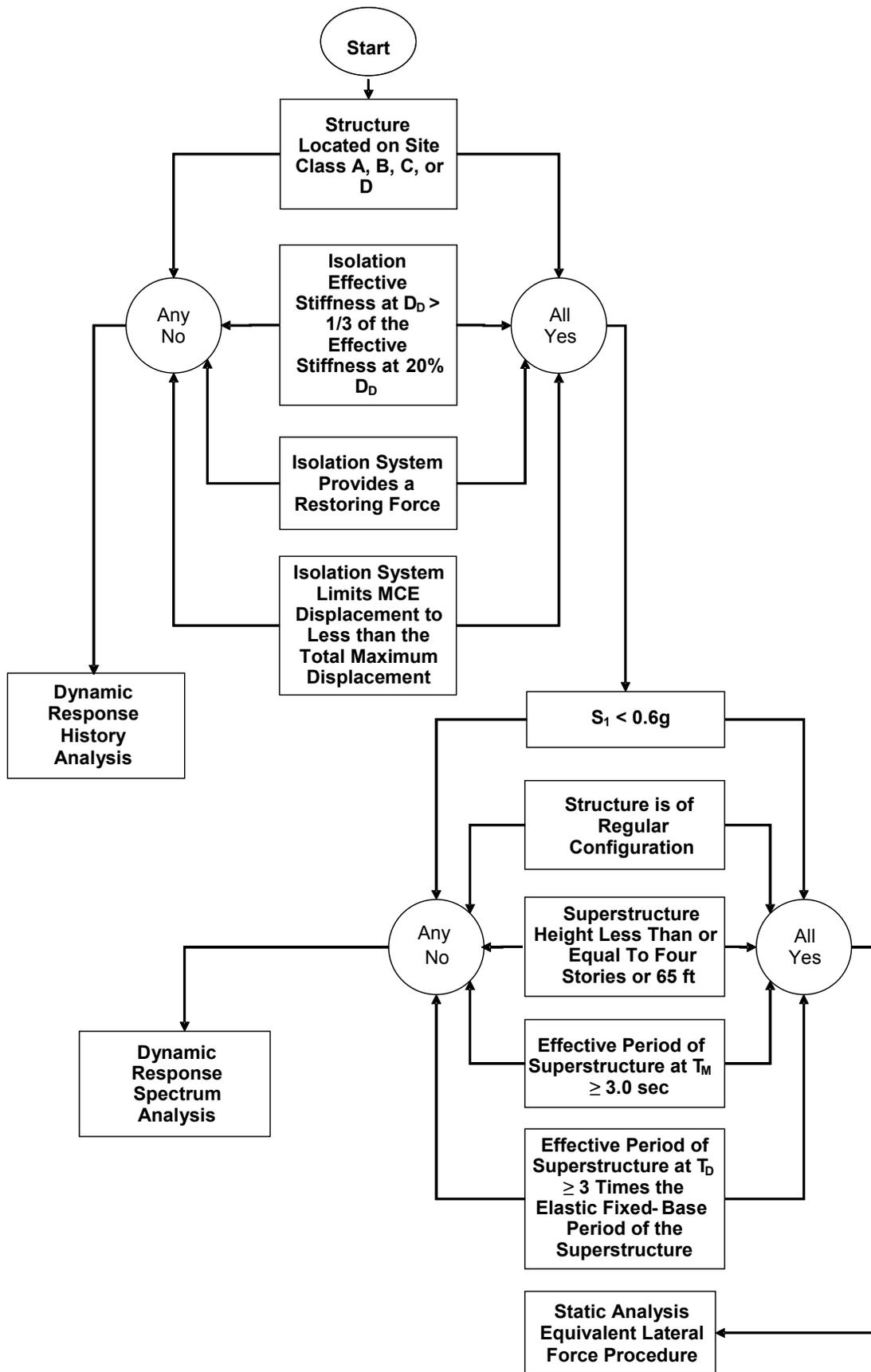


Figure 6.2 ASCE 7-05 Determination of Permitted Analysis Procedure.

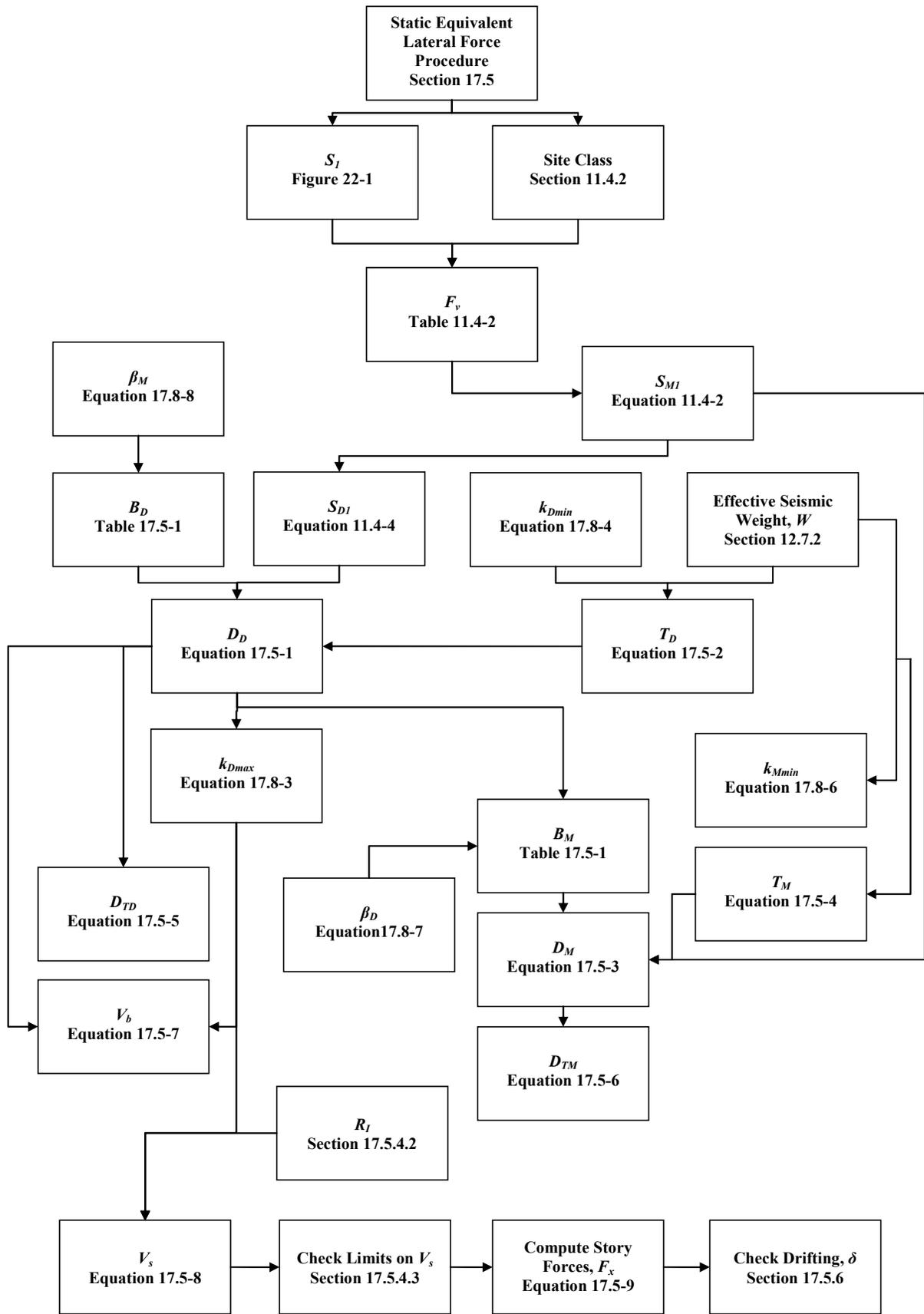


Figure 6.3 ASCE 7-05 Equivalent Lateral Force Methodology.

### ***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,  $S_I$ . 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](http://www.usgs.gov)) or Figure 22-2 in ASCE 7. 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  $S_I$  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,  $S_{DI}$ , 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.

### ***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 6.3 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,  $\beta_D$  and  $\beta_M$ , 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 6.1 and 6.2 (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.

$$\beta_D = \frac{\Sigma E_D}{2\pi k_{Dmax} D_D^2} \quad \text{Equation 6.1}$$

$$\beta_M = \frac{\Sigma E_M}{2\pi k_{Mmax} D_M^2} \quad \text{Equation 6.2}$$

Where:

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

$\Sigma E_M$  = 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.)

### ***Effective Period***

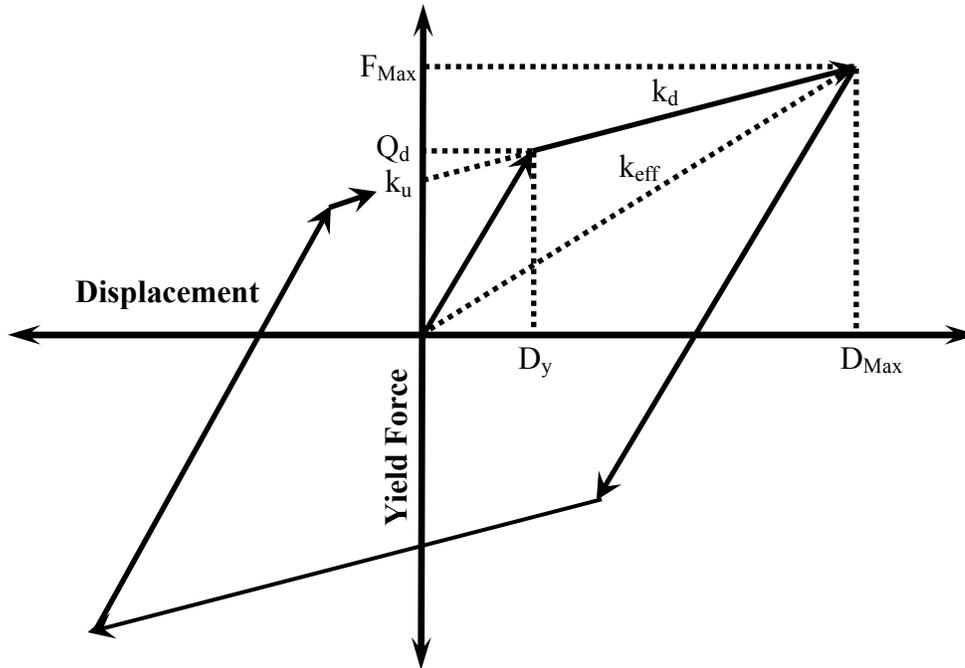
For the purpose of understanding the ELFP design variables and respective equations listed in Figure 6.3 and discussed throughout this section, the isolator's stiffness and effective damping will be approximated. Initially assumed effective stiffness values of the isolation

system are used in Equations 6.3 and 6.4 (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.

$$T_D = 2\pi \sqrt{\frac{W}{k_{D\min}g}} \quad \text{Equations 6.3}$$

$$T_M = 2\pi \sqrt{\frac{W}{k_{M\min}g}} \quad \text{Equation 6.4}$$

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 6.4 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 6.4 Theoretical Hysteretic Loop for Elastomeric Isolator. (Reproduced from Naeim)**

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.

***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,  $S_{DI}$  or  $S_{MI}$ , 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 6.3. Displacement values are computed using the following Equations 6.5 and 6.6 (ASCE 7 equations 17.5-1 and 17.5-3). However, these equations do not include any inherent or accidental torsion that may be induced into the system.

$$D_D = \frac{gS_{D1}T_D}{4\pi^2 B_D} \quad \text{Equation 6.5}$$

$$D_M = \frac{gS_{M1}T_M}{4\pi^2 B_M} \quad \text{Equation 6.6}$$

### ***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 6.7 and 6.8 (ASCE 7 equations 17.5-5 and 17.5-6).

$$D_{TD} = D_D \left[ 1 + y \frac{12e}{b^2 + d^2} \right] \quad \text{Equation 6.7}$$

$$D_{TM} = D_M \left[ 1 + y \frac{12e}{b^2 + d^2} \right] \quad \text{Equation 6.8}$$

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. Torsional 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 6.7 and 6.8 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 variable  $y$ . 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.

### ***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.

### ***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.

### ***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 6.9 (ASCE 7 equation 17.5-7) 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.

$$V_b = k_{Dmax} D_D \quad \text{Equation 6.9}$$

### ***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 6.10 (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.

$$V_s = \frac{k_{D\max} D_D}{R_I} \quad \text{Equation 6.10}$$

Where:

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

### ***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.

### ***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 6.11 (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.

$$F_x = \frac{V_s w_x h_x}{\sum_{i=1}^n w_i h_i} \quad \text{Equation 6.11}$$

### ***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.

## **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.

### ***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 values.

### ***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 6.11, 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

structure and the weight at the story level under consideration. This relationship can be seen in Equation 6.11.

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.

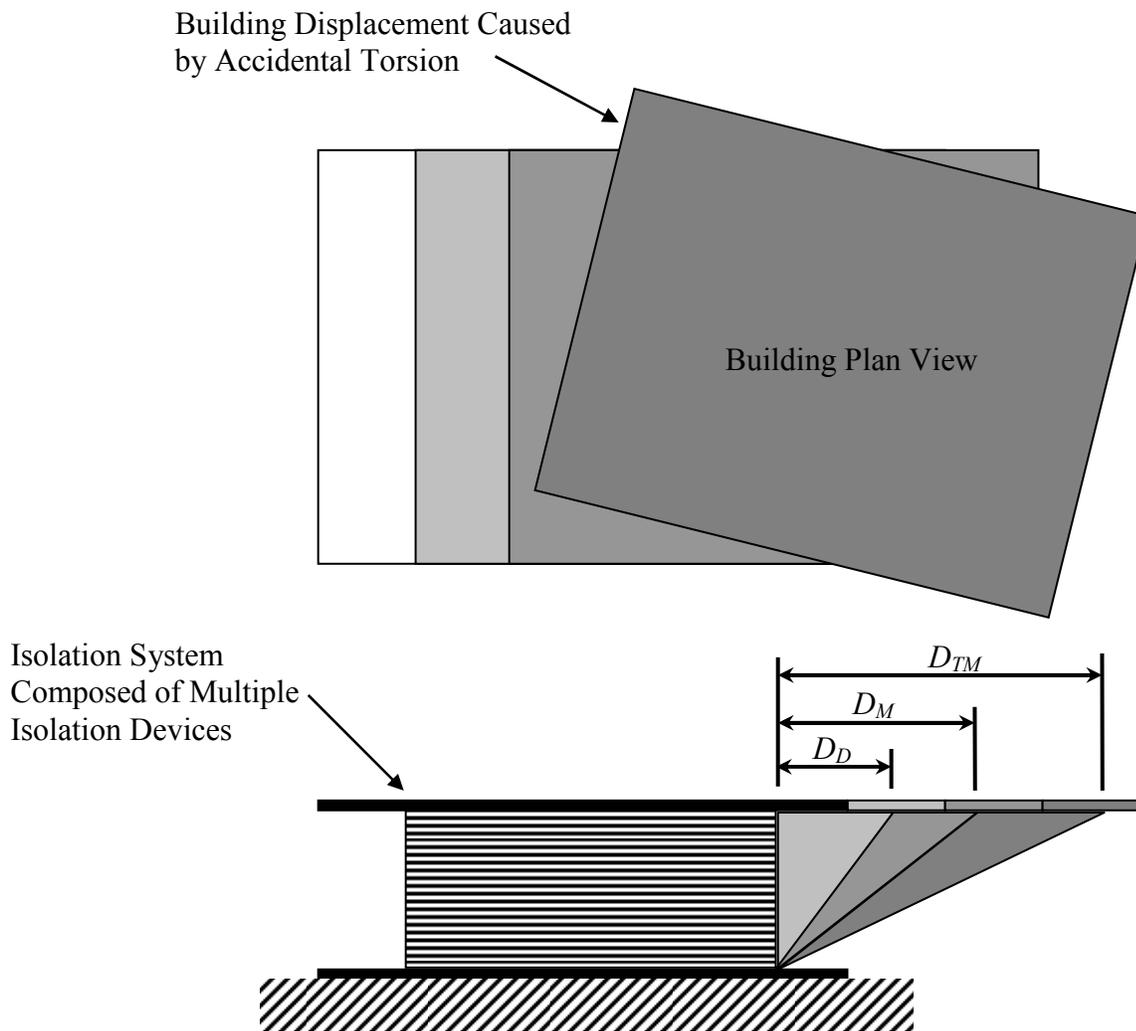
### ***Effective Fundamental Period***

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 6.3 and 6.4 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 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

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.

### ***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 6.5.



**Figure 6.5 Maximum and Minimum Design Displacements for an Isolation System. (Figure Reproduced from NEHRP)**

### ***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, interstory drifting, the total elastic displacement of the structure under factored, strength-level design earthquake forces, should be computed or obtained from the analysis. Interstory 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 interstory drift values obtained by Equation 6.12 (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.

$$\delta_x = \frac{C_d \delta_{xe}}{I} \quad \text{Equation 6.12}$$

Where:

$C_d$  = deflection amplification factor

$\delta_{xe}$  = 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.

### ***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.

### ***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  $3/8^{\text{th}}$  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.

### ***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.

### ***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 , Walters and Elsesser 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, Whittaker, Aiken, and Egan 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.

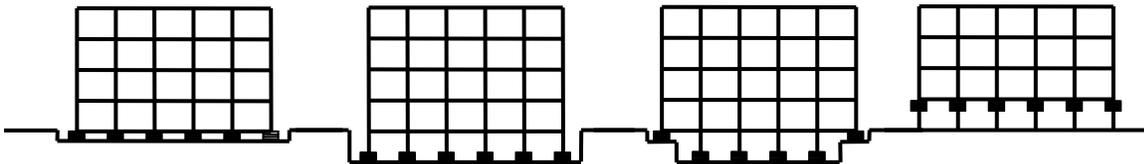
Structural ductility is less significant in structures that are base isolated because less seismic forces are induced into the structure above the isolation interface. Isolation devices are designed so that the initial response to ground movement is significantly small. As the forces at the ground level tend to increase with magnitude, the isolators' stiffness decreases proportionally (Sattary, Walters and Elsesser 1993).

### **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.

## 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 6.6 illustrates various methods of vertical isolator placement.



**Figure 6.6 Vertical Location and Positioning of Isolation Devices.**

## Environmental and Material Effects

The isolation system is required to be designed for any environmental effects on the structure over time that may hinder the structural performance of the system. These environmental and material related issues are in addition to those hazards caused by earthquake and wind loads. An example of the types of issues that designers need to be concerned with are:

creep, fatigue, extreme hot or cold temperatures and moisture, to name a few. The effects of aging on the isolation system should also be considered during design.

## **CHAPTER 7 - Other Design Process Considerations and Performance Issues for Base Isolated Structures**

A number of design considerations when using base isolation systems, although not directly expressed in ASCE 7 or the 2006 IBC, must be addressed during design. This section discusses some of the common issues associated with base isolation and the ways they have traditionally been resolved. Some design concerns are more significant than others and a few can also have considerable impact on building costs. There may be other issues not covered in this section that are more case specific and should be thoroughly evaluated by the design engineer.

### **Movement and Displacement**

One of the most obvious issues that should be considered during the overall design of the system is the movement of the entire structure at the perimeter of the building. The isolated structure will deform and displace a significant distance when fully engaged during a seismic event. During the first dynamic mode of an isolated building, a large deformation will take place in the isolation system while the superstructure above acts as a rigid element (Kelly 1997). Displacement depends on the properties of the isolators and ASCE 7 places an upper limit on the amount of displacement that is permitted. Once the maximum horizontal displacement of the isolation system and the drift associated with the superstructure has been determined, no structural or non-structural elements can interfere with the system movement within that computed distance. Another issue dealing with allowing movement of the structure involves utility service connections which in traditional design come straight through the perimeter of the structure and into the building usually as rigid or semi-rigid elements. If such rigid elements are used in an isolated structure these utility lines could bust causing damage and potential dangers as the superstructure of the building moves. Service connections into the building should be constructed of flexible elements, as shown in Figure 7.1, which can move up to the maximum allowed displacement of the isolators. Additionally, when stairs or elevators are connected to the superstructure they must be detached from any non-isolated portion of the

structure. This issue is resolved by suspending such elements so that movement is allowed between two building levels in which one floor level is essentially fixed to the ground and the other isolated above the ground.



**Figure 7.1 Flexible Utility Connections. (Photo courtesy of Andrew Taylor, KPF Consulting Engineers)**

### *Seismic Gap*

Many factors affect the performance of an isolated building. First, as previously stated, horizontal displacements caused by movement of the structure are limited but must be allowed to take place. This criterion is important and therefore a seismic gap or clear space is required around the perimeter. A properly sized and detailed gap will eliminate damage to the structure by not impeding movement. This seismic gap, also coined as a building moat, around the buildings perimeter allows for movement induced by a seismic event when isolators are engaged. When the total maximum displacement of the isolation system becomes greater than 12” the

building costs related to the seismic gap begin to increase dramatically. This is because displacements increase the required size of the seismic gap at the perimeter of the structure. Over the seismic gap a cover plate is required to eliminate falling hazards. The moat cover is then connected to the building so that it moves with the structure but does not impede the movement by sliding over the exterior paving or sidewalks. As the building moat increases in width, the cost of the cover plate increases, directly affecting the cost of the building. Figure 7.2 shows an example of a seismic gap and cover at the perimeter of the building.



**Figure 7.2 Perimeter Seismic Gap (Moat) at the Conexant Wafer Fabrication Facility in Newport Beach, California. (Photo courtesy of Andrew Taylor, KPFF Consulting Engineers)**

### *Seismic Portals*

Large buildings are sometimes composed of several adjacent structures attached by expansion joints or gaps that allow these multiple structures to act separately. For isolated buildings this becomes more critical than for fixed base structures because of the magnitude of the displacement. Since two adjacent structures will nearly always react differently under seismic induced forces, the structures must be separated so as to not cause damage by pounding into one another. ASCE 7 recognizes this critical feature and notes that an adequate separation

between adjacent buildings and surrounding structures shall be provided that is no less than the maximum total displacement. The means and methods of doing so are left up to designers. Engineers have developed some design approaches to resolve this situation. One approach is to provide seismic joints between the two structures, allowing them to move independent of each other. Similar to an exterior moat, a gap at connecting floor levels is made to maintain normal access between the structures. Another option is to provide a seismic portal between adjacent structures. Seismic portals are in essence, a simple span walkway between structures that span a distance greater than the total maximum structural displacement from one building to another allowing the necessary structural movement.

### **Vertical Load Transfer**

Vertical gravity loads from the superstructure must be supported and stabilized by an isolation system durable enough to withstand such forces. Sufficient loads placed on top of base isolators eliminate possible overturning effects. Distribution of loads just above the isolation device may significantly affect the devices stability and performance during a seismic event. These loads from the superstructure can be transferred in many different ways into the isolator devices. The most ideal way to transfer loads is by locating isolators directly beneath the building columns. If for some reason this is not possible, transfer girders must be used to direct loads out of columns and into isolators that are not placed directly under column locations. Isolators can also be placed underneath shearwalls that are used as the lateral force resisting system. The primary concern, discussed in the next sections, is that enough gravity loads be placed on the isolators to eliminate overturning issues. It is also important to transfer gravity loads uniformly among isolators to avoid undesired torsional affects and isolator irregularities.

### **Overturning and Near Fault Ruptures**

Overturning of the structure as a whole and the individual isolators is a critical issue discussed within ASCE 7. Many opinions on ways to reduce overturning of the superstructure and isolation devices exist. Concerns deal with near fault ruptures where large ground deformations can take place if a structure is built near or along a fault line. Of course this can be

avoided by not constructing a structure near a fault, however the question still of concern is; how far from a fault is enough so that the issue of fault rupture is not a concern? Within near fault regions, structures can experience large horizontal seismic forces as well as significant vertical uplift. If these forces act together there is a possibility isolators may overturn from unbalanced horizontal and vertical loads. ASCE 7 addresses overturning by specifying that the factor of safety for overturning at the isolation interface shall be no less than 1.0 when computing the governing load combinations. Vertical seismic forces shall be determined using the MCE along with the weight of the structure,  $W$ , above the isolation interface as defined for traditional fixed base structures. Each isolation device shall be designed and determined to be adequately stable under the governing design vertical loading in conjunction with a horizontal displacement equivalent to the total maximum displacement of the isolation system. Figure 7.3 illustrates the resulting isolation deformation and the locations of the vertical and horizontal loads placed on the isolation units which may cause overturning. Large values of eccentricity, measured from the isolators' original center of mass to the location of the applied loading, can cause potential overturning. Eccentricity results in induced moments at the isolation interface. The structural building code, ASCE 7 indicates that the predetermined governing equation for the maximum vertical loading is to be equal to Equation 7.1 (ASCE 7, Section 2.3.2, load combination 5). Minimum vertical loading is to be determined from Equation 7.2 (ASCE 7, Section 12.4.2.3, load combination 7). The seismic values to be used in these equations should be no less than the forces determined from the peak response due to the MCE.

$$W_{\max} = 1.2D + E + L + 0.2S \quad \text{Equation 7.1}$$

$$W_{\min} = (0.9 - 0.25S_{MS})D + \rho Q_E + 1.6H \quad \text{Equation 7.2}$$

Where:

$D$  = dead load

$L$  = live load

$S$  = snow load

$E$  = seismic load, using  $S_{MS}$

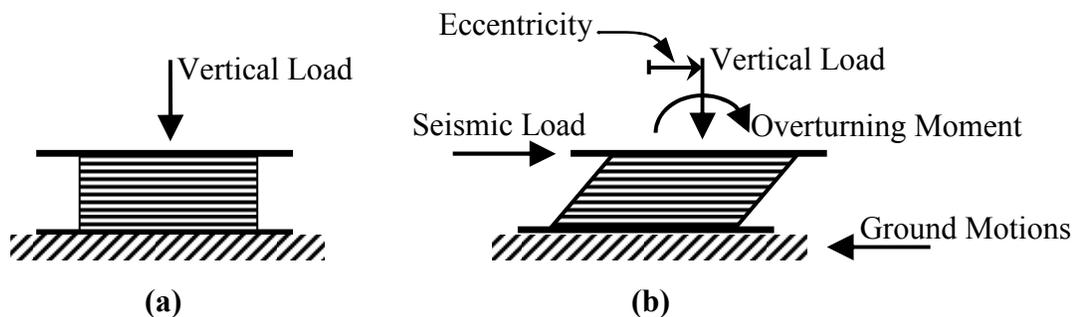
$\rho$  = redundancy factor

$Q_E$  = horizontal seismic force effects

$H$  = lateral earth pressure load

$W$  = total maximum or minimum vertical load

When calculations indicate that an isolator will overturn it does not necessarily signify overturning of the entire isolation system. Adjacent isolators that are not overturning, help to counter the effects of overturning on an individual unit. Isolators should be analyzed individually and as a system to determine the overall global structural stability. ASCE 7 specifies that local uplift effects on any element should not be allowed unless the deflections that result from such uplifting do not cause instability or excessive stresses in the isolation devices or other corresponding structural elements.



**Figure 7.3 (a) Isolator Unit Prior to Seismic Loading; (b) Isolator Unit Induced With Seismic Loading and Displacements Causing Eccentricity and an Overturning Moment. (Image reproduced from Taylor and Igusa)**

### **Tall and Slender Structures**

Structures that are considered to be slender may not be best fit for use with base isolation. Slender structures are buildings which have a very large height-to-width ratio. Tall and narrow structures have issues involving overturning and uplift when vertical and lateral earthquake forces are applied simultaneously or separately to them. A structure at a given height and with a larger base size may not be as susceptible to such overturning and uplift forces as a structure with a small base size at the same height. In some cases, slenderness may only control what type

of isolation system is chosen and not whether it is justifiable. Sliding isolators which rely on the structures weight to engage the system should not be used in situations where uplift is possible because they do not have the capabilities to resist tension as do elastomeric isolator. Sliding isolation units do not have tension resisting elements; there is no element attaching the upper sliding element to the lower sliding element, this contact is obtained purely from the vertical loads of the structure. Additionally, lightweight structures with sliding isolators will have difficulties transferring loads through shear friction since a significant amount of weight is necessary to engage the isolation units. Elastomeric isolators are also not considered to be adequate in tension resistance, but small amounts of tension are tolerable. This is because elastomeric isolators under tension loads are not well understood and have been only minimally researched to date.

### **Wind Loads**

Wind loads on isolated structures can become a design issue in regions where wind speeds are very high or for taller structures that can be greatly affected by such loads. Isolators are designed so they will not engage during a wind event therefore the structure must act as a fixed base structure to resist these forces. Isolators may become unjustified when wind loads become in excess of 10 percent of the structures weight (Booth and Key 2006).

### **Soil Reports**

For the design of isolated structures in high to moderate seismic areas, geotechnical data is highly important because such reports provide information used by engineers to determine the properties required of base isolators and how well a structure will respond during a MCE or DBE. There is a lack of consensus among geotechnical engineers regarding the preparation of design information intended for use in base isolation design (Asher and Hussain 1993). To obtain a relatively accurate cost and benefit value, engineers should be able to obtain the design spectra and spectrum compatible time-history records from a geotechnical engineer for the area under consideration (Asher and Hussain 1993). Discrepancies in a soil report can result in problems during preliminary analysis and design development stages. It is important that

geotechnical and structural engineers develop adequate lines of communication to clearly understand what information is required for design (Asher and Hussain 1993).

### **Multiple Isolator Types**

Vertical deformations in the isolator should be considered in addition to the more severe horizontal deformations. All isolators produce a relatively insignificant amount of vertical displacement when engaged during a seismic event or simply under just gravity loads. Nevertheless, it is important to develop a uniform displacement over an entire building otherwise undesirable strains may be induced into the structure from differential movements. Designers contemplating the use of more than one type of isolation device should pay particular attention to vertical displacements. A sliding isolator will have a different amount of vertical displacement than an elastomeric isolator and similarly for different sizes and shapes of the same type of isolator. Building codes do not explicitly address displacements based on vertical isolator displacement. Instead it is noted that the isolation system shall be stable under vertical loads from seismic forces based on the peak response due to the MCE (ASCE 2006).

### **Isolator Connections**

Another critical element of isolation design is how forces are transferred at the interface between the superstructure, isolators, and isolator foundations. Connections can be potential weaknesses in the overall structure if they are not adequately designed to withstand the transfer of high forces. The type of lateral force resisting system (LFRS) may also potentially affect isolator performance. For instance, if the LFRS was composed of concrete moment frames the base of these frames may be designed to transfer forces from the superstructure by inducing large moments into the isolators. Generally, this would also be the case for fixed base structures, but they play a more important role in isolated structures. Isolator forces are transferred from the superstructure and into the isolation system primarily through shear.

Just like many other high performance structural systems designed to resist seismic forces, advanced connections are required. This means additional construction details and connection design must be made beyond that of typical fixed base structures. Special inspectors

and experienced construction workers are essential. In turn design and construction costs increase for more advanced construction practices not common in today's industry. Figure 7.4 shows an example of the type of connections that are required to be detailed and designed. The friction sliding isolator transfers loads from the superstructure into the isolator through the shear connections located at the top of the isolator cap plate.



**Figure 7.4 Friction Sliding Isolator Interface Connection under Construction at the Pioneer Courthouse Retrofit in Portland, Oregon. (Photo courtesy of Andrew Taylor, KPFF Consulting Engineers)**

Loads are transferred from the isolator into the foundation in similar a fashion. The bottom plate of the isolation unit can be anchored into a basement slab to transfer forces out of the structure which then are dissipated into the foundation below the slab. If the isolators are placed between floor levels at column locations, the shear connections at the top isolation plate will be utilized at the bottom plate of the isolation device as well. The portion of the column that is below the isolator device will then transfer the loads into the foundations below the structure.

### **Technology Verification**

Slow advancement of base isolation in the United States may be partially due to the lack of technological verification. Many base isolated structures were in the vicinity of the Loma

Prieta and Northridge earthquakes, however, ground motions were not strong enough to engage the isolation devices during the event and resulting performance could only be observed rather than measured and studied. Some engineers and designers have concerns that ground motions are relatively higher than some earlier base isolation systems were designed to. Even though many base isolated structures have been designed since those earthquakes, still none have been through an event of great enough magnitude to verify that base isolation acts the way engineers have designed the structure to react. In countries such as Japan, where there is significant growth and application of base isolation, many isolated structures have demonstrated that isolated structures can perform adequately and furthermore behave as intended.

### **Isolator Testing**

Base isolators are required by ASCE 7 to undergo extensive testing prior to being sent to the construction site. This produces an enormous impact on project delivery and construction scheduling. Short construction schedules are not very compatible with base isolation because of the time required for manufacturing and testing. Each structure has different properties and therefore requires unique isolator properties. Additionally, no single isolator is exactly identical to another; this is not an ideal feature. The question arises as to why these are not stock elements that can be simply ordered from a product catalog. These devices are not simple wide-flanged beams with established design properties and capacities controlled by design codes. Design properties may not be well established because of the small number of base isolation designers. Isolation manufacturers do not share ideas and isolator properties since the market is relatively small. Each manufacturer has developed and enhanced the properties of their isolation devices to gain reputation within the engineering profession. The current base isolation market is very competitive and specific isolation materials and properties are kept as confidential. The limited number of isolator manufacturers in the United States makes the industry very restricted in terms of further development.

Current building codes require that design properties of isolators be established by the structural engineer and prototypes are then to be tested to verify and validate their performance values. ASCE 7 specifies that the damping and deformation properties of the isolation system be obtained through testing prior to construction. ASCE 7 also indicates that the results of these

tests cannot be used by manufacturing companies for quality control testing. This results in isolators being tested twice, once for the project and once for quality control performance. Prototype tests required by ASCE 7 are to be performed on two full-sized isolators for each common type and size of isolator being utilized. However, these devices shall not be used in construction unless otherwise approved by the registered design professional for the structure at hand. Requirements for the sequence and cycles of testing are indicated in ASCE 7, Section 17.8 and are beyond the scope of this report.

Testing performance must also be deemed adequate once results are obtained. ASCE 7, Section 17.8.4 describes the conditions in which testing results can be considered adequate and sufficient. Within this list of conditions are a variety of ranges in which specimen results may fall under specified loading tests in order to be considered satisfactory testing. If these measures are not met, testing of additional specimens may be required.

After a seismic event has occurred, it is often required to have isolators which have undergone significant engagement tested to reestablish the design properties of the isolators. Sometimes it is desired to have a selection of isolators tested prior to construction and then again at a designated time in the future. This type of testing verifies that the seismic components are still adequately representing the initial design standards. Additionally, all of these different test results may be used in order to further advance base isolation.

Owners must consider the cost and time related to the testing of isolator elements. The cost and time related to obtaining test data for use with the design of the superstructure and to verify the isolator properties can significantly impact the buildings construction and design schedules. Projects with tight schedules and deadlines may not be able to meet these requirements if not clearly indicated early in the preliminary design stages.

### **Accessibility of Isolators**

At some point in time isolator devices may be required to be removed and replaced. One reason for this may be that the isolator was damaged during an earthquake. Another reason may be so testing can be performed to verify design properties of the isolation system over time. Regardless of the reasoning, isolators must be able to be accessed and removed. There may be significant time and costs associated with a structure whose isolation system is not readily and

easily accessible. Additionally, periodic inspections of the isolation system may be required based on the design service agreements. Isolators may also need to be visually inspected after a seismic event to establish that the elements are structurally sound and stable.

## **Design Reviews**

Prior to construction a seismically isolated structure, as indicated by ASCE 7, shall undergo a design peer review to identify any design deficiencies and validate that the appropriate analysis has been implemented as specified by ASCE 7. The design review should be conducted by independent professional engineers who are registered in the appropriate jurisdiction and have experience in seismic analysis and understanding behind the theory and applications of isolation. It is important that those individuals performing the design review be knowledgeable in seismic design and base isolation. To help establish what elements should be reviewed, ASCE 7 has developed a list of requirements, listed below, which should not be limited to only those points indicated. Design reviews should include the following key items:

- Site-specific seismic criteria, development of site-specific spectra ground motion time histories and any other design criteria's specified for the project under consideration
- Preliminary design of the total design displacement of the isolation system and lateral force design level
- Prototype testing
- Final design of the entire structural system and supporting analyses
- Isolation system quality control testing program

Design reviews should not necessarily be limited to only these elements. However, design reviews or peer reviews can cause substantial delays in a project if not completed in a timely and professional manner. One issue that has come up in past design reviews indicates that reviewers are not always sufficiently knowledgeable in seismic base isolation. Additionally disagreements can arise from the developmental procedures used to create site-specific spectra ground motion time histories. The earlier these issues can be recognized and discussed under

well established lines of communication between the design reviewer and the design professional the fewer delays that will result in the project schedule.

### **Cost of Base Isolation**

Large initial costs are associated with the design and construction of an isolated structure. Unfortunately, sometimes it is the high upfront price tag that eliminates consideration for base isolation. The uncertainties in the cost of the structure may also pose reasons of concern for owners and builders. Cost approximations early in the design stages are difficult to summarize and adequately determine. The true cost of the building is ultimately best analyzed after it has been constructed to obtain adequate values. The initial costs are sometimes used as comparison to a fixed base structure during preliminary design stages to determine the best option. However, such a comparison is not accurately reasonable unless the fixed base structure is analyzed assuming that the same structural performance will be obtained. Since a fixed base structure can be designed to approximately the same level of standards, though difficult in some aspects, it does not always produce an even comparison. A life cycle cost of an isolated structure will help to obtain the structures actual cost. This is because the costs of the structure over time have a greater significance on the total costs than does the initial costs associated with the base isolator devices. Engineers should advise their clients that initial cost comparisons are not adequate for such seismic designs.

## **CHAPTER 8 - Application of Base Isolation**

In the United States buildings that are design 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 in the United States are also provided.

### **Commonly Isolated Structures**

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.

Building heights may be another criteria to consider when deciding whether or not to propose a structure to be isolated. Tall buildings can have very long periods, and since the prime reason for seismic isolation is to change the structures natural period to be longer than the period of the ground motion, taller buildings may not be appropriate. In the United States, typical height limits are around 8 to 10 stories for structures with moment frames and 12 to 15 stories for structures with shear walls (Booth and Key 2006). However, it should be pointed out that other countries that are developing seismic isolation have been applying the technology to structures well above the typical application heights in the United States.

### **Foothill Communities Law and Justice Center**

The Foothill Communities Law and Justice Center (FCLJC) located in San Bernardino, California is notably the first base isolated building in the United States. Construction began in 1982 and was completed in 1985. The 4-story structure was built on top of 98 high-damping rubber isolators located at the sub-basement level (MACTEC Engineering & Consulting 2006). The base isolators were designed to withstand an 8.3 magnitude earthquake (MCEER Information Service 2008). The FCLJC's lateral forces resisting system consists of steel frames with braced frames in some locations to provide stiffness to the structure (MCEER Information Service 2008). Additionally, it should be noted that this structure was constructed only 12 miles from the San Andreas Fault line. Total cost for the project was \$30 million dollars (MACTEC Engineering & Consulting 2006).

## **University of Southern California Hospital**

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.

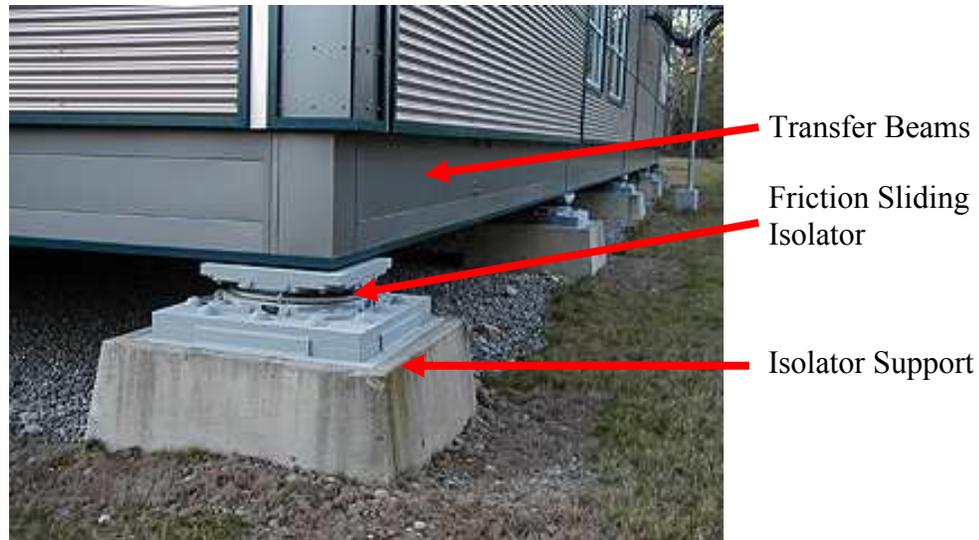
## **San Francisco International Airport**

The San Francisco International Airport, International Terminal in California is one of the largest buildings implementing base isolation in the world. A total of 267 friction pendulum isolators support the structure at the foundation level (MCEER Information Service 2008). Design goals for the International Terminal were to maintain operation during a seismic event with minimal structural damage. With a total of five stories and 2.5 million square feet, this San Francisco Airport expansion was completed in 2001 costing \$2.4 billion dollars (SEA 2005). The airport expansion project is located just 2 miles from the very active San Andreas Fault and 16 miles from the Hayward Fault (SEA 2005).

## **Washington State Emergency Operations Center**

One of the first base isolated structures built in the state of Washington is the Washington State Emergency Operations Center. The Emergency Operations Center 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 8.1, 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).



**Figure 8.1 Washington State Emergency Operation Center Friction Pendulum System (FPS) Isolators. (Photo Courtesy of KPF Consulting Engineers)**

### **Los Angeles City Hall Seismic Retrofit**

The Los Angeles City Hall was constructed in 1928. The structure is 28 stories in height and has approximately 890,000 square feet of floor area. Structurally, the building is composed of steel frames with reinforced concrete walls, steel cross bracing to provide additional stiffening in the structure, and hollow clay tile walls within the interior, all which act to help resist lateral seismic forces (Naeim and Kelly 1999). During the 1994 Northridge earthquake the Los Angeles City Hall was damaged significantly at the upper levels of the structure. Because of damage sustained, it was decided that the structure should be retrofitted and seismically strengthened with base isolators and damping devices. The building was retrofitted with 475 high-damping elastomeric isolators in combination with 60 sliding isolators and 52 mechanical viscous dampers (Naeim and Kelly 1999). This is a prime example of a structure which utilizes various types of isolation devices incorporated with damping. At the levels where severe damage occurred during the Northridge earthquake, 12 viscous dampers were installed between the floor levels to control interstory drift and to protect against future damage to the structural

and non-structural components. Naeim (2001) indicates the retrofit to be estimated at a total cost of close to \$150 million dollars. Of the total cost of the retrofit, the isolators accounted for only \$3.5 million dollars (Naeim 2001).

### **Future Advancement in Seismic Restraint Systems**

In today's advanced seismic design, engineers are thinking to the future, even when today's designs have yet to be perfected. Maybe this is typical of the common engineers mind, always thinking of ways to make a design that's better than current procedures and practice. This section describes a more recent type of damping and isolation system that has been invented and is undergoing researched for future use.

One of the more recent engineering designs is the smart building. The smart building is being designed by University of Notre Dame engineers, Billie Spencer Jr and Michael Sain and funded by the National Science Foundation (NSF). Smart buildings rely on the viscosity of a fluid along with similar damping effects as more common devices; however, their performance is controlled by a magnetic field. The fluid is similar to an oily water mixture and with the presence of a magnetic field the liquid begins to thicken (Science Daily 1998). The building must be equipped with sensors that are used to determine how the building is reaction to the loads being applied at that one instant (Science Daily 1998). Sensors then trigger the damping and isolation systems to reaction in a manor to resist the forces. Smart buildings would require the use of power to supply energy to the building sensors and magnetic field. The work of Spence and Sain, in combination with Lord Corporation, has been presented at international conferences (Science Daily 1998). Other similar devices use magnetics as well and some use computers as a means of control for a similar design to that of Spence and Sain.

### **Design Development of Base Isolation Outside of the United States**

Outside the United States, many types of structure are base isolated to even include non-essential buildings. Design and technology advancements of base isolation in other countries are growing at a significantly greater rate than in the United States (Ryan, Mahin and Mosqueda 2008). Commercial, residential and mixed use buildings that do not require special performance

outcomes during a seismic event are commonly base isolated in countries other than the United States. The majority of these buildings range from 4 to 10 stories in height (Ryan, Mahin and Mosqueda 2008). In countries such as Japan, the public recognizes the significance that base isolation has on structures that surround their lives on a regular basis. Additionally, outside of the United States, structures are advertised based on their type of seismic designs instead of only by their use and functionality. Base isolation is considered general public knowledge, something that is not only understood by engineers and technical experts. Whereas many people in the United State do not understand what base isolation is or know if the building they live or work in is isolated.

A number of reasons why engineers and owners in the United States only have interest in the isolation of highly critical facilities and historical structures exist. One major reason is earthquakes seem to be of little concern to the public on a daily basis. The public is aware that the costs associated with earthquakes can be substantial. However, the United States has yet to see such major earthquake destructions in recent years that other countries have experienced several times during a single year. Earthquakes in the United States are more isolated to certain regions, some very prone to large earthquakes and others only slightly prone based on occurrence predictions demonstrated by seismologists. Another factor affecting the types of buildings designed to be isolated is the construction and technology industry. Construction practices for base isolated structures are lacking and laborers are inexperienced with the seismic technology. It is difficult for contractors to provide accurate construction and labor costs for building owners and designers when they lack the understanding of the features of base isolation. This in turn causes contractors to be apprehensive of such jobs, especially depending on the type of project delivery method. Projects where contractors must bid on the price of constructing the building are unappealing because of the numerous unknowns about the isolation system. Contractors are apprehensive of the risks and liability associated with the construction and bidding of a base isolated building.

Overall, the design of an isolated building is timely, requiring a great amount of effort in a number of areas extending beyond just design. Construction scheduling, design issues, lack of code requirements, lack of adequate technological support from isolation distributors and the disagreement within the design profession make base isolation difficult (Asher and Hussain 1993). This simply indicates that more research and understanding within the design profession

need to be implemented. More educational and technical seminars may help spread common understanding and increase the interest in the design of base isolation within the United States.

## CHAPTER 9 - Summary

In the mid to late 20<sup>th</sup> century, structural base isolation was developed as an alternative method to resist seismic energies and to provide a level of safety beyond what had been traditionally designed for and that of which was established by building codes at the time. A traditional structure designed to such a level of performance would have very high costs associated with it. This technology was originally implemented by civil structures such as bridges before being utilized to base isolate buildings. Several types and variations of base isolation devices have been developed over the past few decades and many newer concepts are being deliberated about even though some are currently unfeasible.

Seismic building code provisions have progressed over the past century. Their inclusion of the design practice for base isolation of structures is a monumental step towards enhanced seismic resistant structures. As earthquakes become seen as a greater hazard over the next century in areas around the United States, the adaption of base isolation may become more prominent. Over recent years in the U.S., earthquakes occurrences have been sparse and often over looked by much of the public as a minor hazard. Though it is only a matter of time for an earthquake to be triggered in a highly populated region where resulting damages could be massive and widespread. Educating the public may also help to gain recognition for base isolation in the following years to come.

Overall, the design of regular, base isolated structures is quite simplistic with the help of some basis design assumptions and isolation properties established by structural designers well experienced with base isolation. However, many argue that the simplified design method may be too simplistic and produce overly conservative results. The dynamic theories of base isolation also tend to suggest this same over conservatism. With the use of more rigorous dynamic analysis procedures engineers are allowed to utilize the accuracy and capabilities of current structural analysis software to gather more accurate design results which are more representative of the true structures response to a seismic event.

Many costs associated with base isolation need to be considered during the design and throughout construction of an isolated structure. Large upfront costs often seem to outshine the extensive structural protection provided by isolation over the buildings life expectancy. To

encourage the use of base isolation, life cycle cost analysis should be conducted as often as possible for buildings of the type. Additionally, the development of universal isolation properties and sizes to be utilized for use with the simplified method of analysis would tend to lower isolation unit costs. As the use of base isolation increases, associated construction practices for the isolation system will become more commonly understood and accepted methods.

The modern idea and concept of base isolation has been utilized in many structures around the world including within the United States. The Foothill Community Law and Justice Center was the first building to be constructed with base isolators in the U.S. Since its completion in 1985, many other buildings have been constructed or retrofitted with base isolation units. Today there are a large number of highly notable base isolated structures and that list continues to expand as does the research and technology advancements. The advantages of base isolation are well defined by its dynamic theories and, if designed properly provide numerous advantages well above what can be offered from the design of fixed base structures.

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