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MSc. IN CIVIL ENGINEERING

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NONLINEAR STRUCTURAL ANALYSIS AND DESIGN OF REINFORCED CONCRETE BUILDING WITH BASE ISOLATION SYSTEM

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SULAIMAN E. SULAIMAN

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Supervisors:
Assoc. Prof. Dr Esra METE GÜNEYİSİ

By
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Sulaiman E. SULAIMAN

ABSTRACT

NONLINEAR STRUCTURAL ANALYSIS AND DESIGN OF REINFORCED CONCRETE BUILDING WITH BASE ISOLATION SYSTEM

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In the recent years, the base-isolation systems are gaining great attention as a solution to protect the structures against the seismic risk. The main role of such system is to separate the structure from its foundation and then eliminate the adverse effect of the earthquake motions. In this study, an existing 9-storey reinforced concrete (RC) building having three bays on the x-direction and five bays on the y-direction direction were considered as a case study model. As a base isolation, friction pendulum type system was used. The seismic performance of the base isolated structure was investigated and compared with the existing one. In this regard, the effectiveness of using base isolation in protecting the building against the seismic actions was examined under different earthquake accelerations. The structures were modeled by means of a finite element method and evaluated by nonlinear time history analysis. The performance characteristics of the fix-based and base isolated RC structures were determined in terms of the storey displacement, interstorey drift ratio, base shear, and columns bending moment. Based on the results of the nonlinear analysis, a substantial improvement in the seismic response of the based isolated structure in comparison to the existing one was observed.

Keywords: Base isolation; Earthquake; Nonlinear analysis; Performance; Reinforced concrete structure.

ÖZET

TABAN İZOLASYONLU BETONARME BİNANIN DOĞRUSAL OLMAYAN ANALİZİ VE TASARIMI

SULAIMAN, Sulaiman E.
Yüksek lisans tezi, İnşaat Mühendisliği
Danışman: Doç. Dr. Esra METE GÜNEYİSİ
Ocak 2015, 70 Sayfa

Son yıllarda, yapıları sismik riske karşı korumanın bir çözümü olarak taban izolasyonu büyük bir ilgi kazanmaktadır. Bu tarz sistemlerin asıl amacı yapıyı temelinden ayırıp, deprem hareketlerinin olumsuz etkilerini ortadan kaldırmaktır. Bu çalışmada, x-yönünde 3 açıklıklı ve y-yönünde 5 açıklıklı mevcut 9 katlı betonarme bir bina örnek olarak seçilmiş ve araştırmada kullanılmıştır. Taban izolasyonu olarak, sürtünme sarkaç tipi sistem kullanılmıştır. Taban izolasyonlu yapının deprem performansı ankastre mesnetli yapı ile karşılaştırmalı olarak incelenmiştir. Araştırmada, farklı deprem ivme kayıtları kullanılmıştır. Yapılar sonlu elemanlar yöntemi kullanılarak modellenmiş ve doğrusal olmayan zaman tanım analizleri gerçekleştirilmiştir. Taban izolasyonlu ve ankastre mesnetli betonarme yapıların performansları kat yerdeğiştirmesi, göreli kat ötelenmesi, taban kesme kuvveti, kolon eğilme momenti gibi tepki parametreleri dikkate alınarak değerlendirilmiştir. Analiz sonuçlarına göre, taban izolasyonlu yapının sismik davranışında mevcut yapıya kıyasla önemli iyileşmeler gözlemlenmiştir.

Anahtar Kelimeler: Taban izolasyonu; Deprem; Doğrusal olmayan analiz; Performans; Betonarme yapı.

To My Parents and Brothers...

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A dissertation such as this is often the work of one person, but always with the help of many others. A great many people have both, directly and indirectly, made this work possible for me. I owe a great debt to my supervisor Assoc. Prof. Dr. Esra Mete Güneyisi who has tirelessly given me superbly professional advice, answered emails at all hours and her great insight are clear within my work. Furthermore, her confidence, her commitment to research, the dedication to her students, the insight and the enthusiasm to risk management helped me to develop my interest in this thesis.

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LIST OF ABBREVIATIONS

BIS	Base Isolated Structures
SI	Seismic isolation
UBC	The Uniform Building Code [Earthquake Regulations for Seismic-Isolated Structures]
SREI	Steel Reinforced Elastomeric Isolator
FREIs	Fiber Reinforced Elastomeric Isolators
FBS	Friction Pendulum Structure
FPD	Friction Pendulum Damper
FPB	Friction Pendulum Bearings
PED	Passive energy dissipation
HDRB	High Damping Rubber Bearings
LCRI	Lead Core Rubber Isolators

CHAPTER 1

INTRODUCTION

1.1 Background

In August 1909 J. A. Calantarients, a medical doctor from the northern English city of Scarborough, wrote a letter to the Director of the Seismological Service of Chile in Santiago calling his attention to a method of building construction that he had developed whereby “In earthquake countries we can put large buildings with full safety, but with the basic principle where the degree of cruelty of an earthquake loses importance by provide lubricated free joint”. Calantarients made a patent application to the British patent office for his construction method, he said that we could built our building on “free joint” and many layer of mica, soft sand, or talc to allow the building to slide on the earthquake, in this case all forces that transmitted to superstructure would directly reduce (Naeim and Kelly, 1999).

What the doctor was prescribing was an early specimen of an earthquake-resistant design strategy known as base isolation or seismic isolation. Many mechanisms have been invented over the last century to try to achieve the goal of uncoupling the building from the damaging action of an earthquake, for example, rollers, balls, cables, rocking columns, as well as sand. Buildings have been built on balls, inclusive a building in Sevastopol, Ukraine, and a five-story school in Mexico City. At least one building, a four-story dormitory for the State Seismological Observatory in Beijing, has been built on a sand layer between the building and the foundation specially designed to slip in the event of an earthquake (Naeim and Kelly, 1999).

Dr. Calantarients mentioned in his letter that, “I made the experiment with balls many years before it was done in Japan or at any event before any amount of if appeared in the papers about 25 years ago”. The reference was almost certainly to the Englishman John Milne, who was a professor of mining engineering in Tokyo in the years 1876 through 1895. During this period Milne became very interested in earthquake phenomena, and he devised and improved a number of seismscopes and seismographs. Milne carried out pioneering researches on the seismology, so much so that he is often referred to as the “Father of Modern Seismology.” He also gave much thought to the design of buildings in seismically active areas and published rules for the earthquake-resistant construction that are still valid today (Naeim and Kelly, 1999).

To convert the basic natural period of a structure to the huge period limits, for example, 2 to 4 seconds the scientists invented seismic isolation technology by developing horizontally flexible devices and by putting it at the base of the structure to separate it from the ground. For earthquake excitability, this shifted period might lead to reduce floor acceleration and drift of inside storey observed on the superstructure compared with the corresponding fixed base structure. This reducing would allow the superstructure to stay elastic, or approximately elastic, following a design level process. Moreover, the probability of damage to displacement sensitive and acceleration sensitive devices would be minimized by reducing demands on non-structural components, and content (Warn and Ryan, 2012).

The concept of seismic isolation has become a practical reality during the last 20 years with the development of multilayer elastomeric bearings, which are made by vulcanization linking of sheets of rubber to thin steel reinforcing plates. These bearings are very stiff in the vertical direction and can carry the vertical load of the building. On the other hand, these bearings are very flexible horizontally, that can made building to move laterally under strong ground motion. Their development was an extension of the use of elastomeric bridge bearings and bearings for the vibration isolation of buildings (Naeim and Kelly, 1999).

The concept of base isolation has also provided a rich source of theoretical work, both in the dynamics of the isolated structural system and in the mechanics of the isolators themselves. This theoretical work, widely published in structural engineering and earthquake engineering journals, has led to design guidelines for isolated structures and design rules for isolators. Several countries are now formulating design codes for isolated structures. In the United States, design codes have been in use since 1986. The code writing process has undergone a steady evolution through code series that began with a simple regulation titled "Tentative Seismic Isolation Design Requirements" based mainly on equivalent static design methods, which was considerably modified and became the 1991 version of UBC code, "Earthquake Regulations for Seismic Isolated Structures". The 1994 and finally the 1997 version of the UBC are even more elaborate (Naeim and Kelly, 1999).

1.2 Research Objectives

The aim of this thesis is to investigate the design of the reinforced concrete (RC) frame building with and without base isolation through nonlinear structural analysis. Structural performance of conventional RC frame building and that protected with a base isolation system under the action of lateral loading were evaluated comparatively. Figure 1.1 also summarizes the main steps of the research considered.

Moreover, the research is expected to find answers to the following questions at the end of the work:

1. What are the base isolator technology and friction pendulum system about?
2. Is the base isolator system really helping in solving the problems in construction resistance to ground motion?
3. The amount of the difference between the displacements for the base isolated buildings and fixed base buildings?
4. The amount of the difference between the drift curves for buildings with and without based isolation?
5. The difference between the curves of bending for the fixed base building and base isolated buildings?

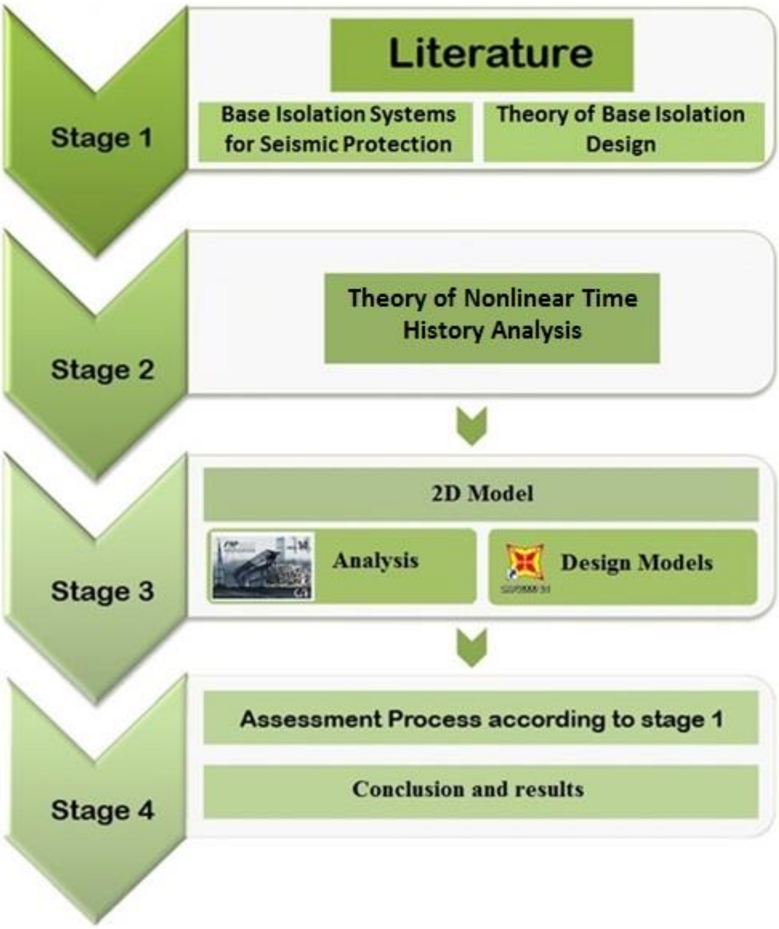


Figure 1.1 Steps of the research

1.3 Organization of the thesis

This master thesis contains the following 5 chapters including (1) Introduction, (2) Literature review, (3) Theories of design and building information modeling with software nonlinear time history analysis, (4) Results and discussion of the non-linear analysis studied in Chapter 3, (5) Conclusions based on the results of nonlinear time history analysis. The chapters are accompanied by a series of appendices after the references section.

Chapter 1: In the introduction chapter, a general background about the research problem is stated and the aim of the research as well as the methodology to be held is presented.

Chapter 2: A considerable amount of related literature based basically on number of articles and books is presented to approach to the statement of problem of the study.

Chapter 3: Review theories of design by explaining all the figures and tables that required in analysis and design are given.

Chapter 4: The responses of the frames with nonlinear properties are analyzed through time history analysis. The results of the analysis are assessed and discussed comparatively.

Chapter 5: The conclusions are drawn up relying on the assessment process of practical work.

CHAPTER 2

LITERATURE REVIEW

2.1 Structural protective systems

Many innovative ways exist to strengthen the structure considering functioning and safety against natural and synthetic dangers. In the recent years, there have been various different researches and developments in this domain. They can be categorized into three general areas as described in Table 2.1 (Soong and Spencer, 2002).

- Base isolation,
- Passive energy dissipation, and
- Active control.

Among the group, the base isolation technology now are considered the most widespread and sophisticated as compared with the other types. To promote damping, stiffness and strength, active and passive energy dissipation systems can be used because they include a group of materials and devices to achieve that for rehabilitation of aging or deficient structures. Furthermore, seismic hazard alleviation can satisfied by using base isolation or by using base isolation with a passive or active control system (Soong and Spencer, 2002).

Table 2.1 Structural protective systems (Soong and Spencer, 2002)

Seismic isolation	Passive energy dissipation	Semi-active and active control
Elastomeric bearings	Metallic dampers	Active bracing systems
Lead rubber bearings	Friction dampers	Active mass dampers
Sliding friction pendulum	VE dampers	Variable stiffness or damping systems
	Viscous fluid dampers	Smart materials
	Tuned mass dampers	
	Tuned liquid dampers	

2.1.1 Passive energy dissipation (PED)

Nearly a 25 year history for development and research of passive energy dissipation PED devices for the structural applications can be considered. The basic tasks of PED devices when integrated into the superstructure of a building is to consume or absorb some of the input energy, by reducing energy dissipation demand on the first structural parts and reducing the possibility of structural damage. Contrary to seismic isolation, moreover, these devices can be effective against wind forces as well as earthquake forces. And there is no need for an external supply of power unlike to active control systems (Constantinou et al., 1998).

Serious efforts have been undertaken in recent years to implement the supplemental damping into a workable technology or to develop the concept of energy dissipation, and for this a number of the devices have been employed in many structures in the worldwide to provide the earthquake protective systems in the structure. These systems may take the form of supplemental energy dissipation devices. In that case, some of the input energy can be absorbed that would improve the structural performance through some type of supplemental "device" not by the structure itself. It is explained that by looking at the conservation of the following energy equation (Constantinou et al., 1998):

$$E = E_k + E_s + E_h + E_d \tag{2.1}$$

Where E is the absolute energy caused by the earthquake motion, E_k is the absolute kinetic energy, E_s is the refundable elastic strain energy, E_h is the area nonrefundable energy dissipated by the structural system through inelastic or other forms of action, and E_d is the energy dissipated by complementary damping devices (Constantinou et al., 1998).

The absolute energy input E, formed through the work done by the total base shear force with the ground displacement at the foundation. It, so contains the impact of the inertia forces of the structure (Constantinou et al., 1998).

Energy dissipation mechanisms are one of the modern seismic protection systems, which dissipate energy and also increase the stiffness and strength. For example, Figure 2.1 shows the force-deformation curves of a simple one-story structure with and without some of these energy dissipation systems (EDS), such as viscoelastic, friction and metallic yielding. EDSs. As seen from the figures, the curves are exhibited to extend well into the inelastic range, as expected to be the case in applications of seismic hazard mitigation. The addition of energy dissipation systems increases the strength and/or stiffness of the structure. In general, the addition of an energy dissipation system would result in a reduction in drift and, therefore, reduction of damage (due to energy dissipation) and an increase in the total lateral force exerted on the structure (due to increased strength and/or stiffness). Mitigation of both drift and total lateral force might be achieved only when deformations are reduced to levels below the elastic limit. In general passive energy dissipation systems are divided into (Constantinou et al., 1998):

- Metallic dampers,
- Friction damper,
- Viscoelastic dampers,
- Viscous fluid dampers,
- Tuned mass dampers, and
- Tuned liquid dampers.

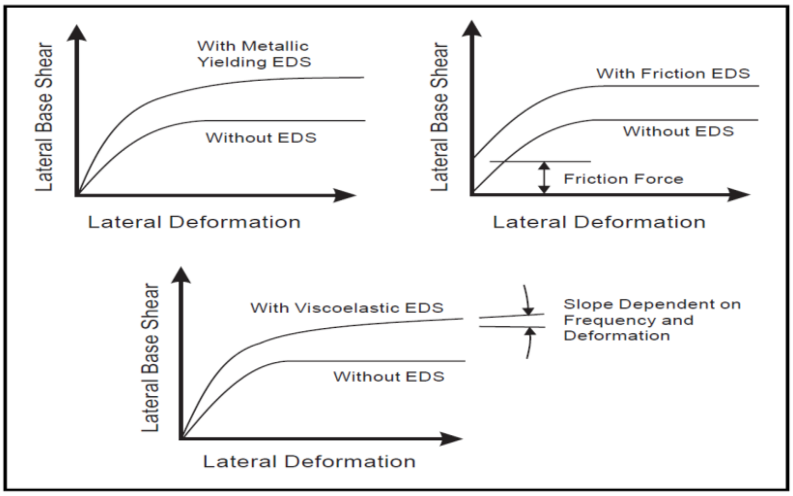


Figure 2.1 Impact of energy dissipation systems on force-deformation curves of a structure (Constantinou et al., 1998)

Passive energy dissipation systems include a range of devices and materials to promote strength, stiffness, and damping, and can be used all of them for rehabilitation of aging or deficient structures and natural hazard mitigation as shown in Figure 2.2. To supply the idea of energy dissipation or supplemental damping, generally, the systems are characterized by a capability to improve the energy dissipation in the structural systems in which they are installed. This can be achieved either by transferring of energy among vibrating modes or by conversion of kinetic energy to heat. The first method includes devices that work on principles like yielding of metals, phase transformation in metals, fluid orificing, deformation of viscoelastic solids or fluids, and frictional sliding. The latter method includes additional oscillators that work as dynamic absorbers. (Soong and Dargush, 1999; Soong and Spencer, 2002)

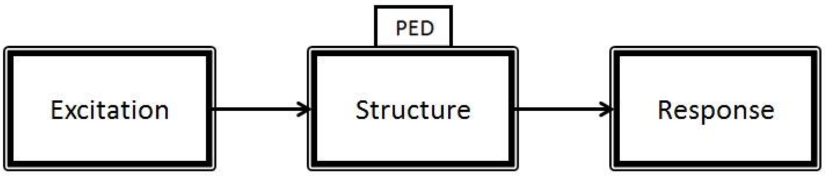


Figure 2.2 Diagram of passive energy dissipation (PED) for structures (Soong and Spencer, 2002)

Among the current passive energy dissipation systems, those depending on fluid orificing represent technologies and on deformation of viscoelastic polymers in which the U.S. industry has a worldwide lead. These techniques have been found recent applications in natural hazard mitigation in the shape of an element of passive energy dissipation systems (Soong and Dargush, 1999).

2.1.2 Semi-active and active control

Active control strategies have been developed as a means of reducing the effects of severe loads. These systems are worked by using external energy submitted by means of actuators to transfer the forces to the structure. The measurements of the structural responses are so necessary to determine the appropriate control action. Nearly for two decades, researchers have achieved the possibility of using active control methods to diminish the structural responses by improving the passive approaches (Dyke et al., 1996).

A variety of active control mechanisms have been suggested in the technical literature. Their mechanisms include the active tendon system (Roorda, 1975;

Yang and Giannopoulos, 1978; Abdel-Rohman and Leipholz, 1978a), the active bracing system (Reinhorn et al., 1989b), the active tuned mass damper/driver (Abdel-Rohman and Leipholz, 1983; Chang and Soong, 1980), and the active aerodynamic appendage mechanism (Soong and Skinner, 1981; Abdel-Rohman, 1984). To evaluate the effectiveness of active structural control systems for earthquake hazard mitigation, the National Center for Earthquake Engineering Research (NCEER) has carried out extensive experiments on scale models of buildings. Chung et al. (1989), and Reinhorn et al. (1989b) also applied active control algorithms to a six story model structure (Dyke et al., 1996).

The first implementation of an active control system to a full-scale building was the Kyobashi Seiwa building in 1989 (Kobori, 1994; Sakamoto et al., 1994) as shown in Figure 2.3 Two active mass drivers (AMDs) were installed on the top floor to reduce structural vibrations due to moderate earthquakes and strong winds, and to increase the comfort level of the building's occupants. A primary AMD (4 tons) was employed to control the lateral motion and the secondary AMD (1 ton) controls the torsional motion. Active and hybrid structural control systems have subsequently been installed in over twenty buildings and utilized during the construction of more than ten bridges (Dyke et al., 1996).

Figure 2.3 Kyobashi Seiwa building with active mass driver system (Gómez et al., 2007)

The devices of semi-active control have received a lot of attention in recent years since it is not required large power sources. Moreover, it allows the adaptability with active control devices. Many of them can operate on battery power, which is critical during seismic events where the probability of failure of energy sources is too big. According to presently accepted definitions, one of the controlled system that cannot increase the mechanical energy is a semi-active control device (i.e., including both the device and the structure), but it has properties which can be dynamically diverse to limit the responses on a structural system. Therefore, semi active control devices do not have the potential to destabilize the structural system (in the bounded input/bounded output sense) in contrast to active control devices. Initial studies indicated that appropriately implemented semi-active systems had the potential to achieve, or even surpass, the performance of fully active systems, that is why semi-active systems performed significantly better than passive devices. It's allowed for the possibility of effective response reduction during a wide array of dynamic loading conditions (Dyke et al., 1996).

The devices of semi-active intrinsically are nonlinear and their control strategies can optimally reduce the structural responses considered as one of the main challenges. To take advantage of the particular characteristics of the semi-active devices different strategies have been developed, including clipped-optimal control, bang-bang control, fuzzy control methods, bi-state control, and adaptive nonlinear control. Active and semi-active control system can be classified into (Dyke et al., 1996):

- Active bracing systems,
- Active mass dampers,
- Variable stiffness or damping systems, and
- Smart materials.

2.1.2.1 Active bracing systems

Active control uses tendons and structural braces. Their mechanisms has been very complicated. Generally, the systems of this type consist of a set of braces or pre-stressed tendons connected to a structure and electrohydraulic servomechanisms are controlled on the tensions in structures. This control mechanism has to do with the fact that braces and tendons already exist members of many structures. Thus, active bracing control can make the use of existing structural members and thus modifications of an as-built structure or minimize extensive additions. This is attractive, for example, in the case of strengthening or retrofitting an existing structure (Soong et al., 1991).

Analytically in connection with the control of slender structures, active tendon control has been studied for offshore structures, tall buildings, and bridges. Early experiments involving the use of tendons were conducted on a series of small-scale structural models, which included a king post truss, a simple cantilever beam and a free standing column, while control devices differs from the tendon control with manual operation from servo valve controlled actuators (Soong et al., 1991).

Recently, a comprehensive experimental study was carried out in order to study the feasibility of active bracing control using a series of structural models that accurately calculated. Figure 2.4 shows the model structures growing in complexity and weight as the experiments progressed from stage 1 to stage 3. The experiments had more control features. Figure 2.5 reveals a schematic diagram of the model structure studied during the stages 1 and 2. By using the method of mass simulation, a three-story steel frame was modeled as a shear building. At stage 1, the top two floors were rigidly braced to simulate a single degree of freedom system. The model was set on a shaking table which provided the external load. As indicated in Figure 2.5 the control force was forwarded to the structure through two sets of diagonal pre-stressed tendons mounted on the side frames (Soong et al., 1991).

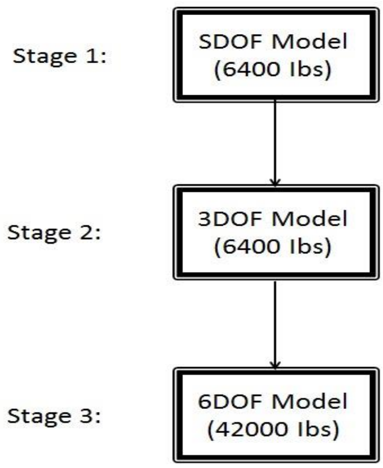


Figure 2.4 Laboratory tests of the active control system (Soong et al., 1991)

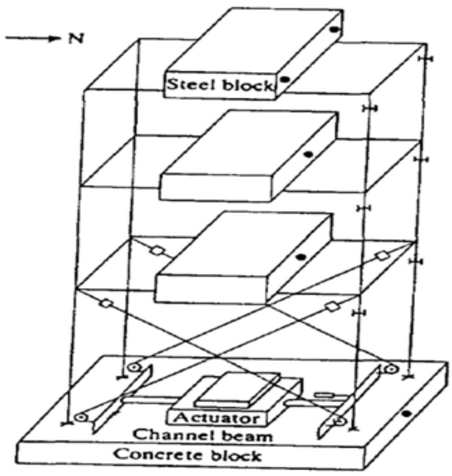


Figure 2.5 Schematic diagram of structure with bracing systems at stage 1 and 2 (Soong et al., 1991)

2.1.2.2 Active mass dampers

This control mechanism was studied as a part motivated by the fact that passive tuned mass dampers for motion control of tall buildings are already in existence. Generally, they are tuned to the first fundamental frequency of the structure, and when the first mode is the dominant vibrational mode it would be very effective for building control. When the vibrational energy spread over a wider frequency band during seismic force attacks, this system woild not be effective. Therefore, when they work according to active control principles it is normal to ask what additional benefits can be derived by that. As expected, when a series of feasibility studies of semi active and active mass dampers (AMD) have been made along these lines and they proved, its promotes effectiveness for tall buildings under either severe wind loads or strong earthquakes. In the recent years, by using scaled-down building models in the laboratory many experimental studies of active mass damper systems have been conducted. In the work of Kuroiwa and Aizawa, (1987), an AMD was placed on top of a four story model frame. The model structure, 1 m (width) x 1 m (depth) x 2 m (height) and weighing 970 kg, was placed on a shaking table which provided in simulated earthquake type base motion. The moving mass was a variable, ranging from approximately 1% to 2% of the structural weight. Following a closed loop control algorithm and using three representative earthquake inputs, experimental results showed that the maximum relative displacement reduction at the top floor could be as high as 50% (Soong et al., 1991).

2.1.2.3 Variable stiffness or damping systems

The main objective of a variable stiffness system is to utilize active systems in order to adjust the structural stiffness so that the resonant modes of the structure can be steered away from dominant modes associated with the seismic input at each time instant. Small-scale experiments using this principle have been carried out. The experimental setup consists of a three story steel frame where active control actions are provided by bracing members with cylinder locking devices as schematically shown in Figure 2.6. Each joint between a brace and the structural frame is engaged or disengaged by opening or closing a control valve in an active mode, thus altering structural stiffness (Soong et al., 1991).

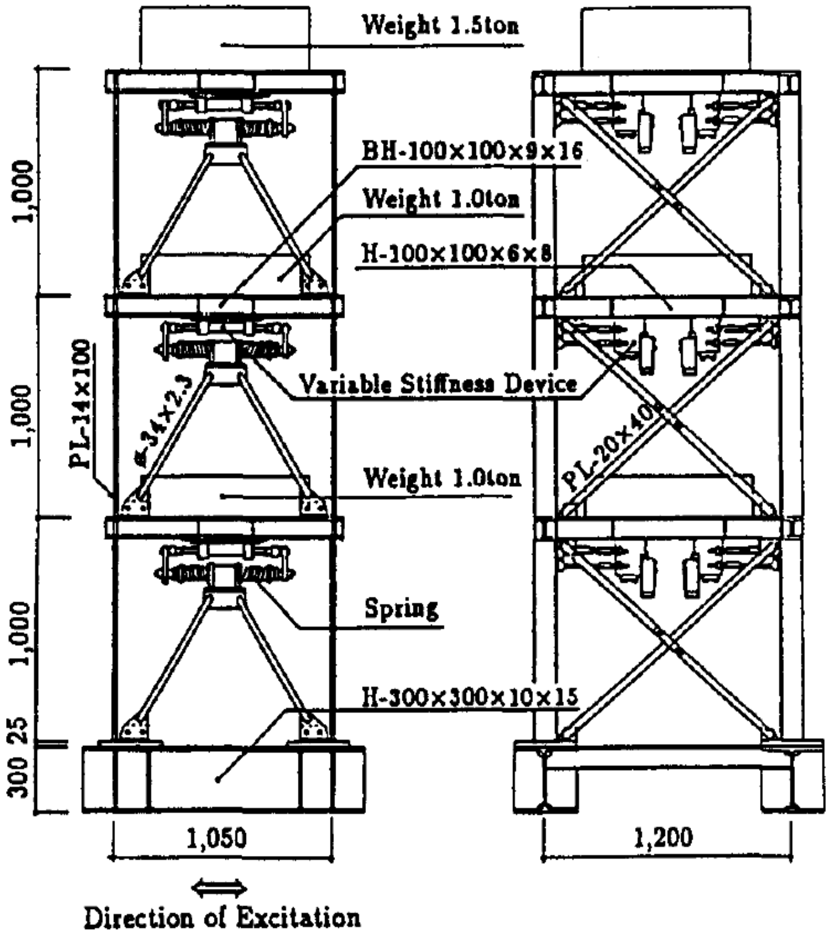


Figure 2.6 Variable stiffness system (Kobori et al., 1993)

An attractive feature of this control scheme is that it does not require a large amount of external energy. In the experiment mentioned above, the only energy requirement was the 12 volts of electricity needed to operate the switch (Soong et al., 1991).

2.1.2.4 Smart materials

The first use of smart material converting was made in 1932 on cadmium-gold. And in 1938 the phase converting was observed in brass (copper zinc). In the Naval Ordinance Laboratory, in 1962, coworkers and Beehler found the transformation and attendant shape memory effect in Nickel-Titanium. This family of alloy Nitinol named after their study. A number of other alloy systems with the shape memory effect were found after a few years the discovery of Nitinol. After the discovery of Nitinol product, product development of use smart materials began to increase, and many of the smart materials containing exotic and expensive elements were discovered. As commercial and attractive systems, the copper based alloys were only come close to challenging the Nitinol family. A number of companies began to provide Ni-Ti materials and components, in the 1980s and early 1990s, and an increasing number of products, especially medical products, were developed in the market (Cai et al., 2003).

Since Titanium and Nickel smart material was first studied in the Naval Ordinance Laboratory, it rapidly found applications in many fields such as biomedical engineering, mechanical, and aerospace etc. In structural engineering, many new cost efficient and high quality materials have come to use with the development of material science. Nowadays, more and more researchers are focusing on smart materials for their performance and properties in civil engineering applications (Cai et al., 2003).

At different temperatures, smart material can exist in two phases: Martensite, which exists in low temperature and Austenite, which exists in high temperature. Under the change in stress or external temperature, these two phases would transform to the other phase depending on the type of the change appears. Smart material exhibits many special properties during the transformations between these two phases, like super elasticity effect, two way memory effect, shape memory effect, etc. (Cai et al., 2003).

The field of structures and smart materials constitutes an emerging market with sensors, image processing, actuators, and technological innovations in engineering materials. Smartness describes self-adaptability, memory, selfsensing, and multiple functionalities of the structures or materials. These characteristics provide numerous possible applications for these structures and materials to be used in manufacturing, civil infrastructure systems, aerospace, and biomechanics. We observed a great benefit from the self-adaptive features of smart structures since it utilizes the embedded adaptation of smart materials like shape memory alloys. Smart materials can detect cracks and faults by changing their properties that is why being useful as a diagnostic tool. To compensate for the fault, it can utilize this characteristic to activate the smart material embedded in the host material in a proper way, this phenomenon is called self-repairing effect (Cai et al., 2003).

2.1.3 Seismic isolation

2.1.3.1 Elastomeric bearings

Base isolation, or the method of decoupling a structure from its base, is an effective way to diminish the horizontal motion produced by an earthquake. In a base isolation system, the horizontal stiffness is low enough to prevent the ground motion from being transmitted to the structure. Elastomeric bearings and sliders are widely used in the current practice (Sadid, 2004).

These system is set to achieve the objective of reducing the force transmission to the structures. But due to their cost, these systems can only be used in expensive and large buildings. Using this system in smaller buildings is based on the ability of reduced the cost and at the same time maintaining the characteristics under other loads. Many low cost and lightweight base isolation devices appropriate for residential buildings are manufactured and experienced for their dynamic working. These devices are manufactured by binding alternating layers of fiberglass mesh and neoprene rubber. The fiberglass is used to get a higher percentage of vertical stiffness to the bearings with retention of low lateral stiffness. The impacts of axial load, ground acceleration and rubber hardness on the lateral stiffness of these devices are tested. And by doing some dynamic testing, it is discovered that might be capable of producing low cost bearings with a low horizontal stiffness for smaller buildings (Sadid, 2004).

Steel reinforced elastomeric isolator (SREI) bearings are currently the most commonly used types of isolator. However, their weight and high price have generally limited their application to large and expensive structures. Alternatively, fiber reinforced elastomeric isolators (FREIs) employ fibers rather than steel plates, as a reinforcement sheet. Fiber reinforced elastomeric isolators (FREI) bearings can provide adequate levels of vertical and lateral stiffnesses as required in a base isolation device. Furthermore, unique aspects such as potentially low manufacturing cost, adequate energy dissipation capability, the possibility of being produced, light-weight in large rectangular sheets and subsequently cut to the required size, provide promising advantages for this type of bearing. Also, unbonded application of these bearings results in additional advantages through beneficial changes in the bearing's lateral load-displacement characteristics (Toopchi-Nezhad et al., 2008).

High damping rubber bearing (HDRB) is a unit of steel-elastomeric isolation consists of steel shim bonded to thin rubber in different layers as described by the designer. Continue with this process results in low horizontal stiffness and large vertical stiffness as shown in Figure 2.7 (Nelson, 1999).

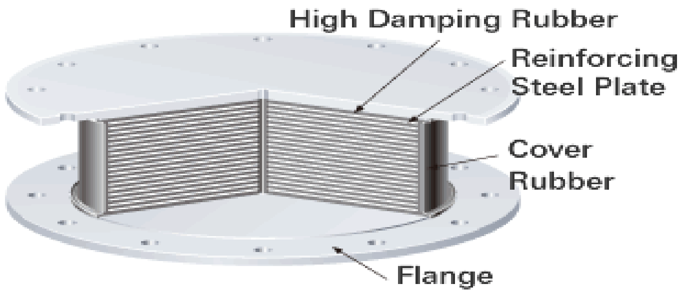


Figure 2.7 High damping rubber bearing (Bridgestone, 2014)

2.1.3.2 Lead rubber bearings (LRB)

Since 1970 several articles have been published to present how to protect the structures from damage resulting from earthquake attack by using base isolation (Skinner et al., 1975; Skinner and McVerry, 1975). These bases isolated structures usually sit on rubber elastomeric bearings or sliding bearings. The main goals of the isolation system is to separate the structure from the ground and avoid the risks of earthquake by increasing bearings and the ringing period of the structure to a higher proportion more than the range of the earthquake energies (Robinson, 2011).

All base isolation studies presented that more reduction in a moment and forces movable to the structure happens if a damping normally 5% of the weight is put with the same level of base isolation (Lee and Medland, 1981; Meggett, 1978). Another feature of using the damping leads to a reduction of displacement by an estimated 2% to a more susceptible to treatment size of nearly ±100 mm (Robinson, 2011).

The plastic deformation of lead for dampers started with the use of a steel piece with the lead extrusion damper which behaves as a rigid plastic solid. A more recent innovation was the lead rubber bearings (LRB) (Robinson, 1975; Robinson and Tucker, 1983; 1977). It's composed of an elastomeric bearing with a lead plug down its center as in Figure 2.8. The cheapest solution for the base isolation of the structure probably is the use of LRB. Its supports are provided by an elastic restoring force and the amount of damping by the selection of the appropriate size of lead plug (Robinson, 2011).

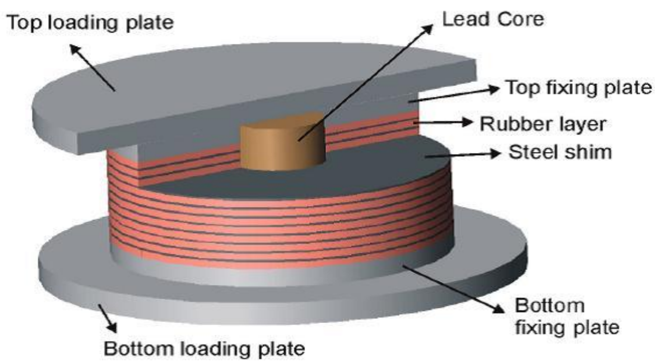


Figure 2.8 Lead rubber bearing (Andrade and Tuxworth, 2009)

LRB in one unit provides the combined features of energy absorbing capacity, horizontal flexibility and vertical load support required for the base isolation structure (BIS) from the earthquake attack. It has been tested several of small displacements with nearly 1100 cycles at ±3 mm. In New Zealand, more than 92 of LRB have been used to isolate three bridges and one building (Robinson, 2011).

2.1.3.3 Sliding friction pendulum

A significant number of studies on the base isolation systems has concentrated on the use of frictional elements to achieve the required flexibility of the structural system by adding damping to the isolated structure. A pure friction system considers the simplest represent of this sliding system without any restoring force. It's very effective for a wide range of frequency due to this system supporting a relatively rigid superstructure. Moreover, it allows for a limited amount of earthquake force to input and transmitted this amount equal to the maximum limiting frictional force (Scheaua, 2011).

For the earthquake resistant design of the structures, an effective tool is the application of the seismic isolation that can be provided in both retrofit and new construction. Friction pendulum is a seismic isolation bearing with a mechanism depends on surface friction properties and its concave geometry as shown in Figure 2.9. Dissipating hysteretic energy via friction by administered into a pendulum motion as a plate glides simultaneously on the concave dish is considered for supported structure. Seismic isolation bearings can define as a structural joint that are installed between foundation support columns and a structure. The aim of this device is to minimize damage that occurs by large lateral displacements that accompanies the earthquake (Scheaua, 2011).

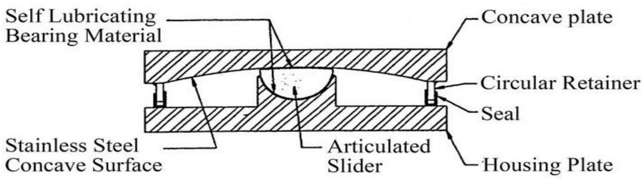


Figure 2.9 Cross-section of sliding friction pendulum (Petti et al., 2013)

Friction pendulum systems as shown in Figure 2.9 are made up of steel concave surface in contact with an articulated friction slider set on the dense chrome free to move during lateral displacements. These tools are designed for each facility depend on the earthquake displacement capacity, the size of the structure being supported, soil conditions, and the load capacity requirements. Bearings can be designed to absorb different sizes of displacement by modifying diameter of the bearing seismic event due bearing seismic event due to the gravity and the concave surface of the bearings (Scheaua, 2011).

2.2 Development of seismic isolation worldwide

Buildings began to use the base isolation systems in the USA from approximately 10 years ago, many examples are listed in Table 2.2. These buildings were built to remain effective without damage during and after a major earthquake (like emergency operations and medical centers). Buildings that used isolation system is considered so expensive. However, this technique considered the best way to provide a protection from the earthquake attack, especially for hospitals. In northern California a few structures have been built with this system (The San Francisco Main Library which has just been completed), historic structures have recently modified by the same technique (Kelly, 1998).

Table 2.2 Building with base isolation systems in the USA (Kelly, 1998)

Name of building	Location	Status
Foothill Communities Law and Justice Center	Rancho Cucamonga	New building, completed
USC University Hospital	Los Angeles	New building, completed
Rockwell international office bldg	Seal Beach	Retrofit of existing building, completed
County of Los Angeles Fire Command and Control Center	East Los Angeles	New building. completed
Martin Luther King, Jr/Charles Drew Medical Center	Willowbrook	New building, completed
Kaiser Regional Data Center	Corona	New building, completed
Emergency operations Center	Los Angeles	New building, completed
Veterans Administration Medical Center	Los Angeles	Retrofit of existing building, completed
Los Angeles City Hall	Los Angeles	Retrofit of existing building, in design development, construction scheduled for near future
County of San Bernardino Medical Center	Colton	New building, near completion
Caltrans Traffic Management Center	Kearny Mesa	New building, near completion
Hoag Memorial Hospital Nursing Tower	Newport Beach	Retrofit of existing building, beginning construction
City and County of San Francisco 911 Center	San Francisco	New building, near completion
County of San Diego Emergency Communications Center	San Diego	New building, near completion

The first isolated building was built in 1914, Oakland City Hall Figure 2.10. At the time of its construction was the tallest building in the west coast of the US, and then Los Angeles City Hall, which was built in 1926 superseded as a tallest building at that time (Kelly, 1998).

Figure 2.10 Oakland City Hall, California (Wikipedia, 2014)

The Oakland City Hall is 97.5 m tall and 19 stories high. It has a 3-story podium, a 10-story office tower, a full basement, and a 2-story base of the clock tower that is itself 26.5 m high. The structure of the building is a riveted steel frame with infill masonry walls of granite, terracotta, and brick. The building was suffering from a major damage in the 1989 and it's closed because Loma Prieta, California, earthquake. A combination of 75 ordinary rubber bearings and 36 lead rubber bearings were used as isolator system in this building. The bearings range was 445 mm tall and are between (737-940 mm) in diameter, to provide a seismic gap a moat was constructed around the building of 508 mm (Kelly, 1998).

Another building is the San Francisco City Hall Figure 2.11 which designed in 1912 superseded the previous structure which destroyed in the 1906 San Francisco earthquake. It's considered classic building and listed in the national register of historic places. This building has been retrofitted after the damage done by the Loma Prieta earthquake in 1989. The retrofit process includes strengthening the frame by using concrete shear walls with base isolation systems. The main aim of the retrofit process was to maintain the historic fabric of the building. The isolation equipment was placed above the existing foundation, with 530 lead-plug rubber bearings. As in the Oakland City Hall project the isolation system placed after a complicated process of shoring and cutting. Almost all columns are supported by four isolators under the steel structure. The construction started in 1994 and was planned to be completed in 1998 (Kelly, 1998).

Figure 2.11 City Hall, San Francisco, California (Hofmann, 2011)

Another example of the isolation system application is Los Angeles City Hall building given in Figure 2.12. It is a steel frame building with 32-stories in hight, and a total floor area close to 92 900 m^2. Several different elements were used to provide the lateral resistance such as RC walls, interior clay hollow core tile walls, and steel cross-bracing. In addition to these, the masonry infill perimeter walls were used to ensure the superstructure stiffness. Some damage in the building happened in 1994 during the Northridge California earthquake, especially on the 25th and 26th soft storeys. 560 high-damping natural rubber isolators were used as retrofit equipment, which were supplied by Bridgestone Engineered Products Company, Inc. In addition to these, 52 mechanical viscous dampers were used at the isolate level. Furthermore, 12 viscous dampers were installed between the 24th and 26th floors to control the story drifts (Kelly, 1998).

Figure 2.12 Los Angeles City Hall (Wikimedia, 2008)

Figure 2.13 The Foothill Communities Law & Justice Center, California (Kelly, 2013)

Another building retrofitted is the Foothill Communities Law and Justice Center as shown in Figure 2.13. It is a four story building located about 12 miles from the San Andreas fault. It was subjected to a devastating earthquake with a magnitude of 7.9 in the southern of California in 1857 and an infamous earthquake in San Francisco in 1906. The Foothill Communities Law and Justice Center were retrofitted by using pads of rubber which sits atop its foundations (Kelly, 2013).

Many essential facilities were built with isolated systems. Emergency control centers are one of those facilities that must remain functional during and after earthquakes. These buildings are not designed according to the Uniform Building Code (UBC), but in accordance with performance based requirements. An isolated emergency center considered from the recent example on the isolated buildings is a two story Traffic Management Center for Caltrans, California. Figure 2.14 shows concentrically braced panels at the perimeter of the superstructure which is a steel frame. High damping natural rubber isolators have been used as an isolated system. This structure is very stiff and the design is quite conservative. This building was designed with no moat around the structure, thus no wall, so there is no probability of an effect on the superstructure in dangerous events and this will eliminate the possibility of high floor accelerations which may influence on the sensitive equipment (Kelly, 1998).

Figure 2.14 Los Angeles, California, fire Command Control Facility (Collins, 2011)

The use of isolation systems in Japan began slowly compared to the USA, and then the isolation studies and development of seismic protective systems increased quickly. In 1986, the first large base isolated building was completed in Japan. In June 30 of the 1998, 550 of base-isolated buildings have been approved. Although, these buildings require special approval from the Ministry of Construction, several reasons have contributed in the rapid development of isolation systems in Japan. The spending on scientific research and development in engineering were on a high level, a large portion of them have been allocated to the base isolation systems. When making seismic design decisions, the high seismicity of Japan encourages the scientists to favor the long-term benefits of life-cycle costs of buildings and life safety (Kelly, 1991).

The West Japan Postal Computer Center given in Figure 2.15, now considered the largest base-isolated building in the world located in Sanda, Kobe Prefecture. It has six stories with 47,000 m^2 (500,000 ft^2) structure in this building, 120 elastomeric isolators have been used to isolate the bases of this building with a number of lead and steel dampers. The isolated period of this building was 3.9 sec, and it is located about 19 miles (30 km) from the epicenter of the Hyogo ken Nanbu (Kobe) earthquake which occurred in 1995. During its life, this building had experienced many ground motion. The isolation system was reduced the peak ground acceleration to 127 cm/s^2 (0.13 g) at the sixth floor, which it was 400 cm/s^2 (0.41 g) in the fixed base case. The displacement was about 12 cm (4.8 in.) with isolators (Kelly, 1991).

Especially after the Kobe earthquake, the use of isolation technology in Japan continued to increase. A rapid increase in the number of permits for base-isolated buildings caused by the superior performance of the West Japan Postal Computer Center. Thus, several condominiums and apartments were constructed with this system (Kelly, 1991).

Figure 2.15 The West Japan Postal Computer Center (Kelly, 1991)

Europe is considered similar to other states in the area of use and development of base isolation systems like, USA, Japan and China. The reduction of the seismic motion transmitted by the ground that requires a study of the concept of seismic isolation (SI) for earthquake protection of structures dates back many hundreds of years (Martell and Forni, 1998).

The temple of Diana which stands in Ephesus considered an example of Greek magnificence that worthy of true wonder, took 120 years to build it. This monument is on unstable and slippery soil. A layer of wool fleeces, and a layer of coal chippings were laid underneath. Although, this building was erected in a marshy area, there was no fear of cracks in the soil or earthquake. In the recent years, several types of isolation systems have been proposed and applied. Europe similar to other continents in the development of the seismic isolation. Western Europe (especially, France) has been started in the 1960s to give considerable momentum to the improvement of seismic isolation systems that consisting of rubber bearings, which consider important in projects that require special design such as nuclear structures. In several European countries, research and development has begun on the innovative anti seismic techniques. In 1989, in Italy a team working on Seismic Isolation named GLIS was formed and now has more than 180 members. Several conferences to study the seismic resistance have been held by the GLIS, especially in 1993 and 1997. In Europe, another group to work on the base isolation of structures named WG 8 related with the European Association for Earthquake Engineering was established. Several industrial plants and civil buildings provided with anti-seismic devices in Italy, and reproduced by permission of GLIS are listed in Table 2.3 (Martell and Forni, 1998).

Table 2.3 Industrial plants and civil buildings with innovative devices in Italy (Martell and Forni, 1998)

Building and plants				Isolators-dissipators-restrain			
No	Type	Location	Year	Type	No	Diameter (mm)	Height (mm)
1	New fire station headquarters	Napoli	1981	Mechanical dissipators and isolators	NA	—	—
2	2nd fire station	Napoli	1985	Neoprene bearings and oleodynamic restrain	24 72	700 —	150 —
3	Hospital	Siena	1989	Friction dissipation	NA	—	—
4	Civic Centre	Monte d'Ago, Ancona	1989	Neoprene bearings	6	900	180
5-9	Telecom Italia Centre of Marche Province	Ancona	1989-92	HDRBs	182 115	600 500	204 204
10	S Giovanni Battista Carife (retrofitted)	Avellino	1990	Oleodynamic restraints	18	—	—
11	CNR Laboratory	Frascati	1990	Mechanical dissipators	NA	—	—
12	Apartment houses (twin-isolated and non-isolated houses)	Squillace Marina (Catanzaro)	1990-92	Rubber bearings HDRBs	14 13 16	400 500 300	205 205 208
13	Navy building	Ancona	1991-92	HDRBs	24 20	600 500	303 303
14	New ENEL headquartes	Napoli	1989-93	Mechanical dissipators and Oleodynamic restraints	142 NA	— —	— —
15	Navy Medical Centre	Augusta (Sicily)	1992-93	HDRBs	16 8	400 500	354 328
16-19	Apartment houses of the Italian Navy	Campo Palma (Augusta, Sicily)	1992-93	HDRBs	48 36 36 16 24 16 16	300 350 400 400 450 450 450	254 294 304 294 306 282 258
20-21	FIAT industrial buildings	Melfi (PZ) and Prato La Serra (CB)	1993	Oleodynamic restraints	173 NA	— —	— —
22	Sports Hall	Rimini	Completed	Oleodynamic restraints	NA	—	—
23	Faculty of Engineering	Brescia	Completed	Oleodynamic restraints	NA	—	—
24	Department of Mathematics, University of Basilicata	Potenza	1995	HDRBs	89	500-800*	262-278
25-28	Blocks 1-4, Faculty of Agriculture, University of Basilicata	Potenza	1995	HDRBs	132	500-800*	262-278
29,30	Airport hangars	Bologna and Torino	Completed	Oleodynamic restraints	NA NA	— —	— —
31	Turbine and Thermal Power Cycle Buildings	Montalto Power Station	Completed	Oleodynamic restraints	NA	—	—
32	Large water supply pipe	S Giacomo (Abruzzo)	Completed	Oleodynamic restraints	NA	—	—
33	Standard telephone switchboard houses	In seismic areas	To be constructed	HDRBs	NA NA	—	—
34	Bell tower	Trignano-RE	In progress	SMA devices	4	—	—
35	Gas-insulated electric substations	ENEL Plants	Designed	HDRB/RBR/Wire rope	4-8	—	—

NA = not available

* These HDRBs have a square, rather than a round, section so the measurements given are for the dimensions of the sides of the squares

Many other isolated projects that were constructed in Russia and former Soviet Union countries for many years and reproduced by permission of GLlS, are listed in Table 2.4 (Martell and Forni, 1998).

Table 2.4 Industrial plants and civil buildings with innovative devices in Russia and Soviet Union (Martell and Forni, 1998)

Type of (SI) system	Location	Construction	Buildings
Pendulum suspension and steel springs system	Ashkabad-City (Turkmenistan)	1959	1
Flexible ground-floor columns, in conjunction with energy dissipation switching-off elements and rigid inelastic displacement limiters	North-Baikal. Siberia and Kamchatka (Russia)	1972-1996	125
Pile-in-tube system with switching-off inelastic elements	Tynda, Siberia (Russia)	1989	2
Low-friction teflon/steel supports sliding between the foundations and walls, in conjunction with rigid displacement limiters	Bishkek (Kirgizia) and Kamchatka (Russia)	1984-1990	40
Kinematic (rocking) RC supports consisting of columns with aspheric ends, in conjunction with switching-off and dry friction elements	Sebastopol (Ukraine)	1972-1974	3
Rocking upside-down mushroom-type RC supports	Alma-Ata (Kazakhstan) and Kamchatka (Russia)	1979- I989	120
Rocking RC supports consisting of columns with plane ends, in conjunction with displacement limiters and friction surfaces	Buryatia, Caucasus, Siberia (Russia)	1987- 1990	8
Steel-laminated rubber bearings	Minsk (Byelorussia) and Tashkent (Uzbekistan) and Vanadzor (Armenia)	1985-1996	4
Total number of applications			303

Similar to USA, Japan, Italy, in New Zealand seismic isolation systems have developed and advanced to protect the new and existing bridges, buildings, and industrial establishments. Several buildings were constructed in New Zealand by isolating system. The William Clayton building in Wellington-New Zealand given in Figure 2.16, was the first building in the world to be isolated with lead-rubber bearings which is completed in 1981. To isolate this building, 80 lead-rubber bearings were used. Many other buildings in New Zealand have been isolated in the same way, but retrofit technique with seismic isolation for the existing buildings in New Zealand has just getting started (Robinson, 1995).

Figure 2.16 William Clayton building in New Zealand (Robinson, 2011)

Another clearest example of isolated bases in New Zealand was Bannockburn Bridge, which was completed in 1988 and isolated by using lead-rubber bearings located between the bridge deck and the piles, while at the abutments by using several pieces of the lead- rubber bearings and the lead-extrusion dampers to provide the main damping force. The Parliament Library and the old Parliament Building in central Wellington are other examples of retrofitted buildings as shown in Figure 2.17. They are now under way where 514 bearings for the lead-rubber isolation have been tested in the laboratory, other stages of engineering work are nearing completion (Robinson, 1995).

Figure 2.17 Parliament Library and old Parliament Building in New Zealand (Holmes, 2014)

On the waterfront in central Wellington was constructing a major new building in June 1993, called the Museum of New Zealand. This building has a triangular plan (190 x 104 m) with five floor levels of a total floor area of 35,000 m^2 and a height of 23 m, planned to be isolated by 142 lead rubber bearings. The photograph of lead rubber bearing is given in Figure 2.18. In this base isolation systems, teflon sliding bearing were used under shear (Robinson, 1995).

Figure 2.18 Lead rubber bearings used in the Museum of New Zealand (Stewart, 2012)

The widespread application of this technology is still impeded by over conservative attitudes. For example, number of bureaucratic mandates are available in the United States, (i.e., peer reviews, feasibility studies and plant, site inspections) that an engineer must satisfy to isolate a structure. If bearings doesn't become a catalog commodity with allied and certified characteristics, with the analysis procedures and simple design that promote the benefits of base isolation; this technology will remain restricted to a few projects a year and it will be difficult to implement, importantly. Base isolation provisions are now in the UBC. Although the requirements of base isolation are so conservative, the isolation reduce design requirements in the superstructure (Naeim and Kelly, 1999).

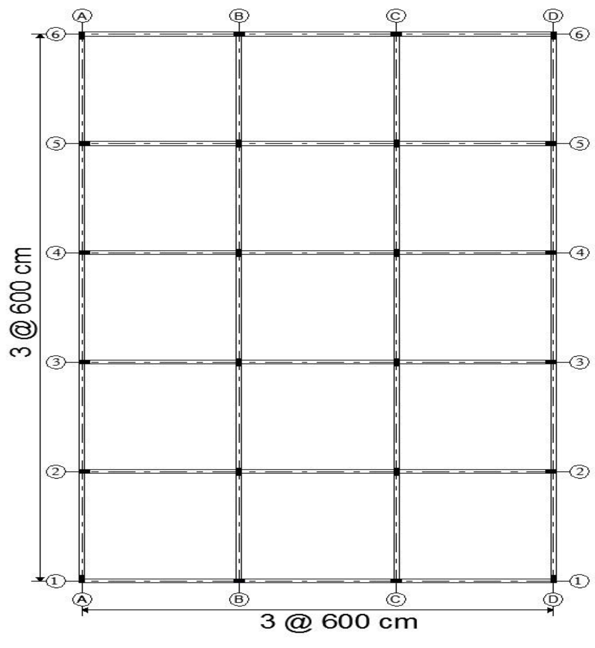
The performance of many bases isolated buildings, were experienced by earthquakes. Except the 1994 Northridge earthquake, these earthquakes were either small and nearby or have been distant and moderate, so the acceleration of isolated structures experienced has not been sufficient. In earthquake-prone regions of the world when more isolated buildings are built, engineers can expect learning more about the behavior of such structures, and the degree of conservatism that is currently present in the design of these structures will be possible to reducing of it. Then a sufficient information about the response of base isolated buildings to major earthquakes will be available, the next step is harmonization between the codes for isolated structures and fixed base with a common code base on a certain level of structural performance and seismic hazard, which will allow the cost-effective application of this new technology for those appropriate building types. The long term stability of the mechanical characteristics of the isolator and its constituent materials, considered the most important area to all systems for future research. Inspection and retesting of examples that have been in service for many years constitutes the best way to control the long term performance of insulators. For 30 years, elastomeric systems in the form of non-seismic bridge bearings have been used and have established a record of satisfactory performance (Naeim and Kelly, 1999).

CHAPTER 3

METHODOLOGY

3.1 Analytical model of the structure

A nine storey reinforced concrete (RC) building as a case study model was utilized in order to compare the seismic response of the structures with and without friction pendulum base isolation systems. The original structure was first designed by El-Amoury and Ghobarah (2005) and it was slightly modified in the current study and used for different purposes. For this, the building was modeled considering a series of planar frames connected at each floor level by rigid diaphragms. The column foundations were considered as fixed base case for the first one and as base isolated case for the second one. These frames were designed as three bays on the x-axis and as five bays on the y-axis, the structure was considered as regular in shape in order to carry out the analysis on two dimensional model which makes easy to interpret the results of the analysis. Typical floor plan and elevation of the case study RC building as well as its 3-dimensional view are given in Figures 3.1.



a)

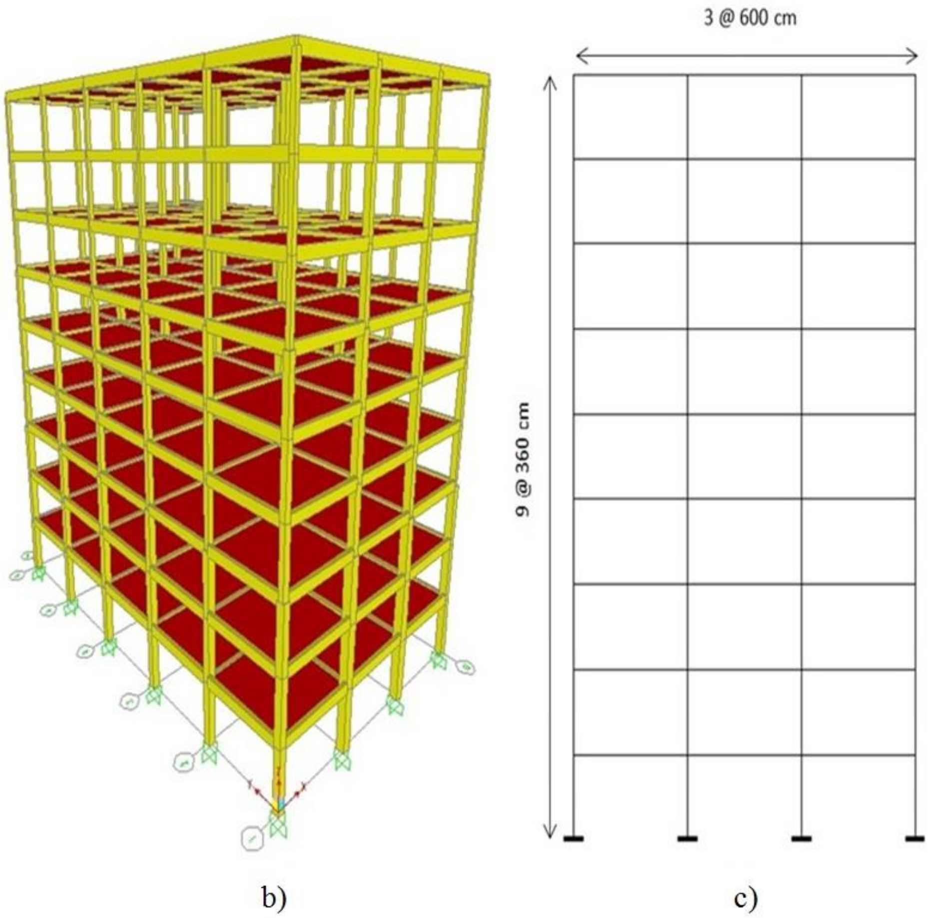


Figure 3.1 9 storey RC structure a) plan, b) 3-dimensional, and c) frame elevation views (El-Amoury and Ghobarah, 2005)

The selected structure was analyzed by using the finite element program SAP 2000 non-linear version 14 (CSI, 2011). The buildings measured (18 m × 30 m) in plan, with a bay width equal 6 m (3 bays by 5 bays) and floor height equal 3.6 m, all slab thicknesses were 15 cm. The dimensions of the beams were 60 cm in height and 25 cm in width. It was the same dimensions in the exterior and interior sides. The exterior and interior frames of the buildings comprised three bays. The largest dimension of the columns designed toward external axes, nevertheless, the short direction of the columns designed parallel to the x-direction. The dimensions of columns were changed for every three storeys and varied from external side to internal side for every three storeys as shown in Table 3.1. The weight due to the dead load of the frames was calculated to be 663.5 kN/*floor*, and the design live load of the building was taken as 2.00 kN/m^2. The frame represents typical office building that was constructed in accordance with minimum design loads for buildings and others structures

(ACI-318, 1963) and uniform building code (UBC-64, 1964). The building mass was assumed to be a uniform distribution over the height and the stiffness was taken as a non-uniform lateral distribution. The compressive strength of concrete used in the structure was 21 MPa while the yield strength of the steel was 300 MPa.

Table 3.1 The dimensions of columns in the building

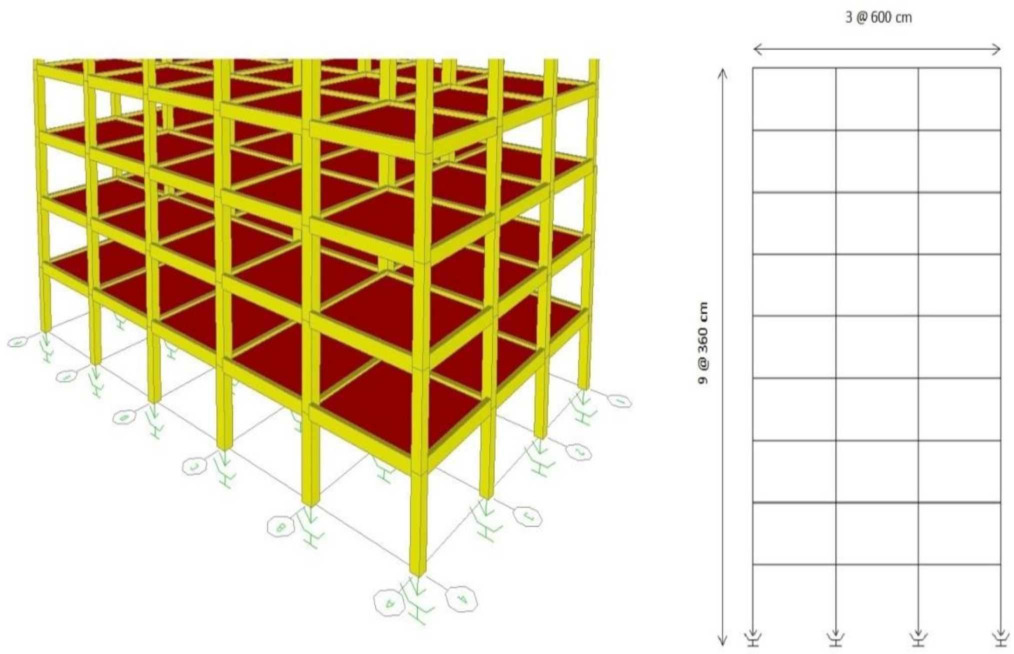
Storey level	1	2	3	4	5	6	7	8	9
Dimensions of the external columns (cm)	50x50	50x50	50x50	40x40	40x40	40x40	30x30	30x30	30x30
Dimensions of the internal columns (cm)	60x60	60x60	60x60	50x50	50x50	50x50	40x40	40x40	40x40

Friction pendulum base isolation system was used in improving the seismic performance of the RC buildings. For this, friction pendulum isolators were introduced as N1-link elements by considering 0.5 m in length between the columns and the fixed based, just like in the case of rubber isolators (Figure 3.2). The properties of friction pendulum isolators that have been used in the design are shown in Table 3.2.

Table 3.2 Design properties of friction pendulum isolators

Non-linear link type	Friction isolator		
Directional properties	U1	U2	U3
Linear effective stiffness (kN/cm)	350000	17.5	17.5
Non-linear effective stiffness (kN/cm)	350000	350	350
Friction coefficient slow	–	0.03	0.03
Friction coefficient fast	–	0.05	0.05
Rate parameter	–	0	0
Radius of sliding	–	1	1

All the values that remaining outside the above table were taken as zero. The effective damping value was entered as zero because in the nonlinear time history analysis it had no functionality.



a) **b)**

Figure 3.2 9 storey RC building a) 3-dimensional view with friction isolators and b) elevation of the frame with friction isolators under investigation

3.2 Nonlinear analysis method

Nonlinear analysis is an aid for the structural analysis used essential component of a computer. These aided tools are needed because of the expenditure of the testing of prototypes, therefore the increasingly being replaced by simulation with nonlinear finite element methods. The clear necessity of nonlinear analysis can be recognized in the study of Belytschko et al. (2013):

a) Designing high-performance and efficient components of certain industries (e.g. aerospace, defense and nuclear),
b) Assessing functionality (e.g., residual strength and stiffness of structural elements) of existing systems that exhibit some types of damage and failure,
c) Establishing causes of system failure,
d) Simulating the true material behavior of processes, and
e) Research to gain a realistic understanding of physical phenomena.

3.2.1 Classification of nonlinear analysis

The nonlinearity can be classified in two categories, first one is according to the problem and the other one is according to the tools of solution or analysis (Bonet and Wood, 1997):

3.2.1.1 Classification according to problem

Three sources of nonlinearity exist in the analysis of solid continua, namely, material and geometric nonlinearity and in special case due to changing initial or boundary conditions (Bonet and Wood, 1997).

3.2.1.1.1 Material nonlinearity

This is infinitesimal displacement and strains, occurs when the stress-strain behavior given by the constitutive relation is nonlinear (e.g. Nonlinear strain-displacement relations). The given figures (Figures 3.3-3.6) show the many cases of material nonlinearity. In the first figure, the frame displaced horizontally and vertically (for any reason), this case can be understood by observing the relationship between stress and strain, showed that the material tend to behave nonlinearly after increasing the load above the yielding stress σ_y (Bathe, 2006).

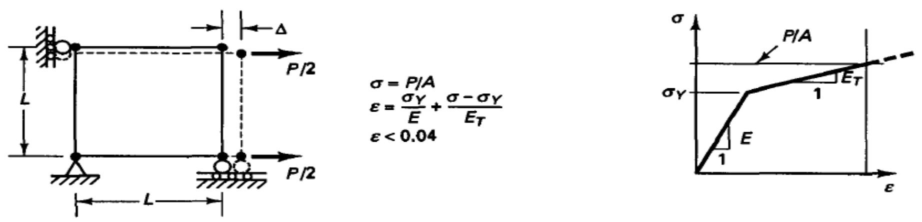


Figure 3.3 Materially-nonlinear-only (infinitesimal displacements, but nonlinear stress-strain relation) (Bathe, 2006)

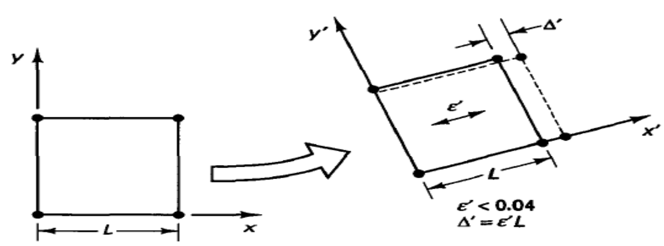


Figure 3.4 Large displacements and large rotation, but small strains. Linear or nonlinear material behaviour (Bathe, 2006)

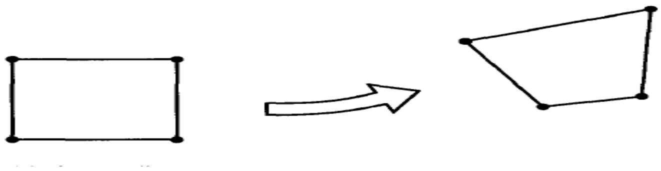


Figure 3.5 Large displacements, large rotation and large strains, Linear or nonlinear material behaviour (Bathe, 2006)

3.2.1.1.2 Geometry nonlinearity

In normal structural analysis, it would be enough to consider small deformations and strains, even many parts of the structure can only be subjected to small strains in order to maintain their ability for using. So, when the elastic deformations are present, then a linear constitutive equation could be introduced. But, even under this assumption there are many problems which create rotations or large displacements as in beams, shells or cables. These problems require a nonlinear theory which includes the geometry in an exact way, therefore the geometry nonlinearity is related to the structural instability. This occurs when the geometry of the bodies are changed, however; large or small, has a significant effect on the load deformation behaviour including deformation-dependent boundary conditions and loading. The geometric nonlinearity arises purely from geometric consideration (e.g. nonlinear strain-displacement relations), and the material nonlinearity is due to the nonlinear constitutive behaviour of the material of the system (Bathe, 2006).

Various sources of nonlinearities are linked with special boundary constraints. One of the main causes of such nonlinear behaviour is boundary constraints which lead to change with the deformation state of a system (e.g. during the increase in the assumed load). this happens when a certain body contacts with another one during a deformation process as shown in Figure 3.6 (Bathe, 2006).

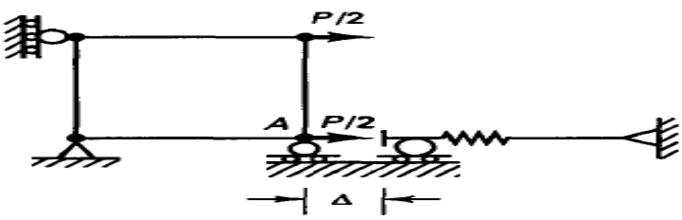


Figure 3.6 Change in boundary condition at displacement Δ (Bathe, 2006)

3.2.1.2 Classification according to solution

This is explained briefly below (Bathe, 2006):

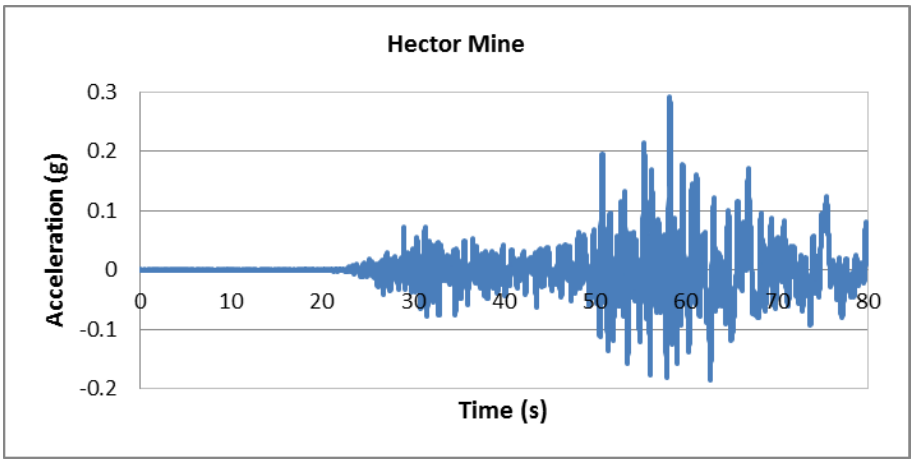
- Methods oriented on evaluation of all eigenvalues and eigenvectors (e.g. Jacobi method),
- Methods oriented on evaluation of a selected group of eigenvalues and eigenvectors, and
- Methods oriented on evaluation of a single eigenvalue and eigenvector (usually extremal, e.g. power method, reverse iteration method).

3.3 Details of analysis in this study

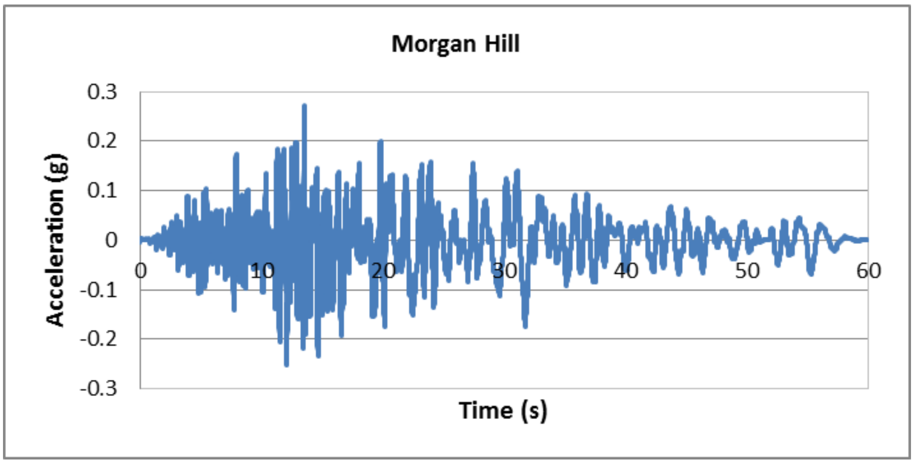
The total design base shear or design lateral force along any principal direction was given in terms of the design seismic weight and the horizontal seismic coefficient of the structure. The design horizontal seismic coefficient was evaluated based on the importance of the structure, response reduction factor of the lateral load resisting elements, fundamental period of the structure, and the mapped acceleration parameters of the site. The finite element program of SAP 2000 Advanced 14.0.0 (CSI, 2011) was used for the modelling, analysis and design of the strctuers.

Time history analysis is one of the most commonly used technique to assess the nonlinear behaviour of the buildings. For this, nonlinear time history analysis was conducted to compare the seismic performance of the existing RC frames with fixed base and those with friction isolators.

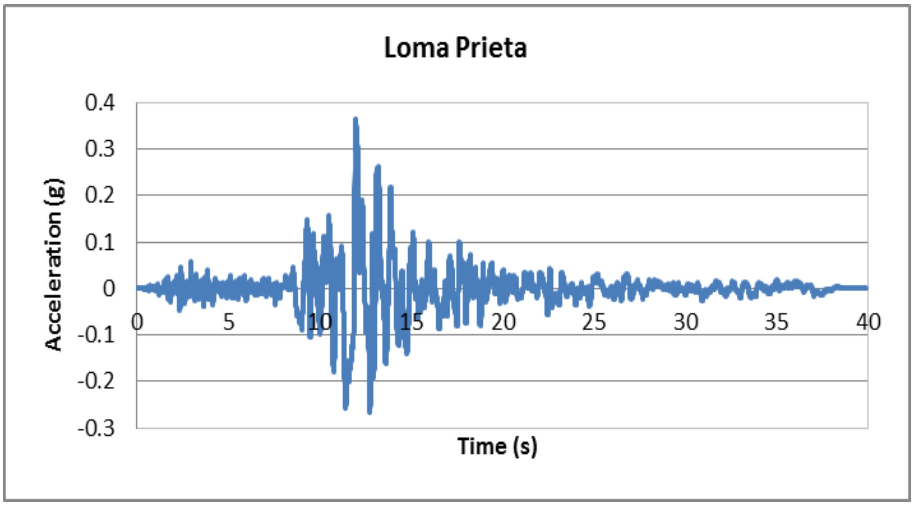
The dynamic analysis for the building when exposed to four different ground motions was carried out by solving the equation of motion using this software program. These earthquakes were Hector Mine, Morgan Hill, Landers, and Loma Prieta. The complete time history records of the earthquakes that have been utilized are shown in Figure 3.7.


Hector Mine
Acceleration (g)
0.3
0.2
0.1
0
-0.1
-0.2
0
10
20
30
40
50
60
70
80
Time (s)

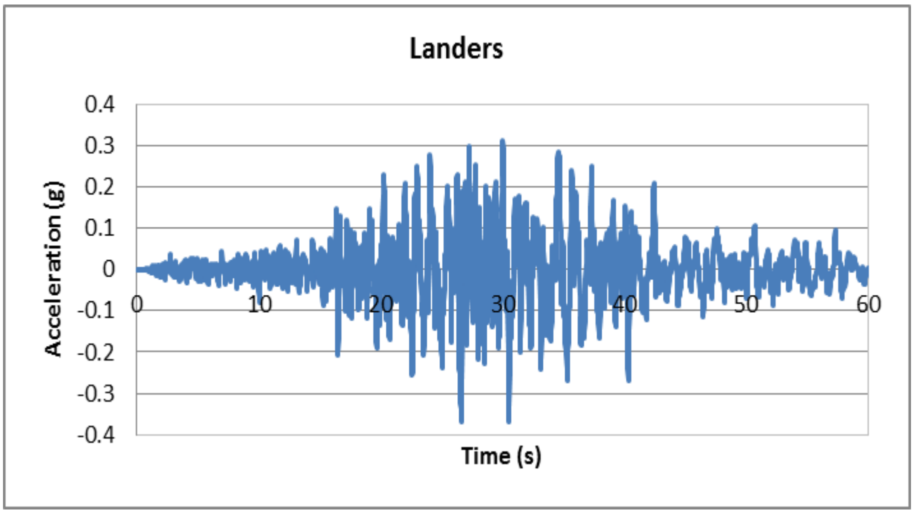
a)


Morgan Hill
Acceleration (g)
0.3
0.2
0.1
0
-0.1
-0.2
-0.3
0
10
20
30
40
50
60
Time (s)

b)


Loma Prieta
Acceleration (g)
0.4
0.3
0.2
0.1
0
-0.1
-0.2
-0.3
0
5
10
15
20
25
30
35
40
Time (s)

c)



d)

Figure 3.7 Ground acceleration for a) Hector Mine, b) Morgan Hill, c) Loma Prieta, and d) Landers earthquakes

The number of output time steps and other properties such as the peak ground acceleration (PGA), peak ground displacement (PGD), peak ground velocity (PGV), properties of the site and the magnitude (M) for all earthquakes are shown in Table 3.3 whereas the output time step size for Landers is 0.08 seconds and for the other are 0.02 seconds.

Table 3.3 Properties of the scaled earthquake ground motion

Earthquake record	Hector Min	Morgan Hill	Loma Prieta	Landers
Year	1999	1984	1989	1992
Magnitude (M_w)	7.13	6.19	6.93	7.28
Mechanism	Strike-Slip	Strike-Slip	Reverse-Oblique	Strike-Slip
V_{s30} (m/s)	301.0	239.7	594.5	308.6
PGA (g)	0.292	0.271	0.365	0.370
PGV (cm/s)	57.427	41.908	72.405	44.124
PGD (cm)	84.132	18.988	19.066	12.666
Number of output time steps	4050	2997	3995	3131
Scale factor	27.6	11.6	3.0	8.8

The analysis was carried out such that the structure has been reached to failure, thus became possible to identify the yielding point for the system. The hinge properties for the structural components were identified according to FEMA-356 (2000), taking into consideration the failure mechanism and the component type. After choosing the hinge properties for the model, the structures exposed to a gradual increase of lateral force to reaching into the suitable displacement. The elastic spectra and design code spectrum for the earthquakes are given in Figure 3.8. The elastic design spectrum was obtained according to the Turkish Earthquake Code, 2nd seismic zone, soil type Z4, for seismic hazard level of 10% probability of exceedance in 50 years.

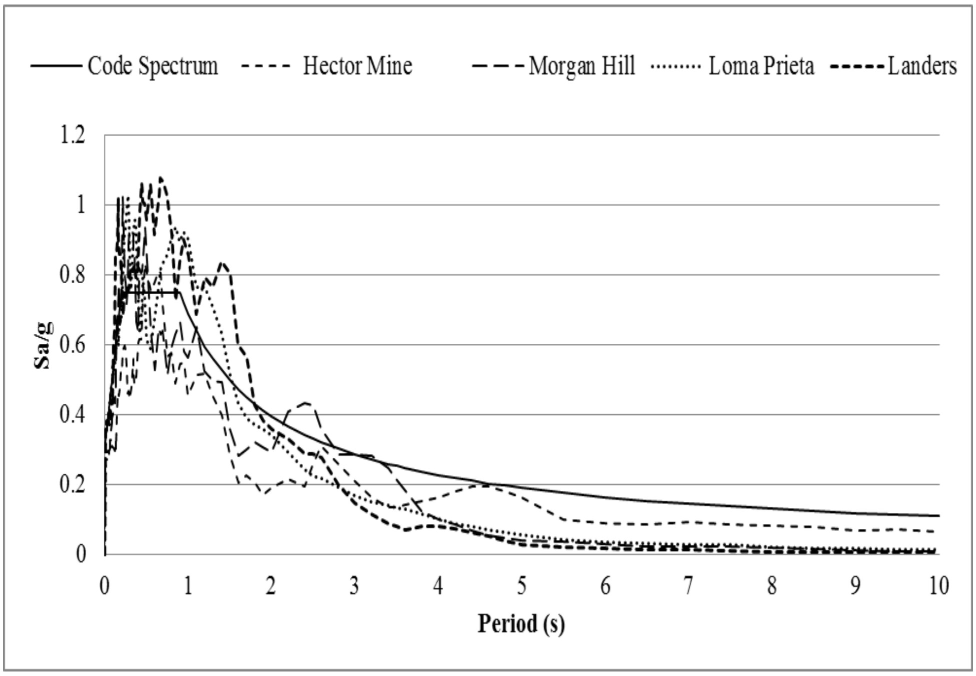


Figure 3.8 Elastic spectral accelerations of the ground motions

Elastic behaviour would occur on all of member length, and then deformation outside the elastic limit could happen internally in the hinges, which were considered along the lines of discrete places. While obtaining inelastic behaviour through the integration of the plastic curvature and plastic strain which occurred inside a selected hinge length, usually on the member depth (FEMA-356, 2000). By adopting plastic hinges with hysteretic relationships achieved the nonlinearity and that was according to FEMA-356 (2000) at each end of the column and beam members. The biaxial moment hinges and axial force (PMM) were taken for the column members whereas the flexural moment hinges (M3) were taken in order for the beams. The dynamic properties for the existing RC frame and that equiped with friction pendulum isolators are shown in Table 3.4.

Table 3.4 Dynamic properties for the existing and base isolated frames

Type of frame	T_1 (s)	T_2 (s)	T_3 (s)
Fixed based case	1.51	0.55	0.31
Base isolated case	2.51	0.75	0.39

CHAPTER 4

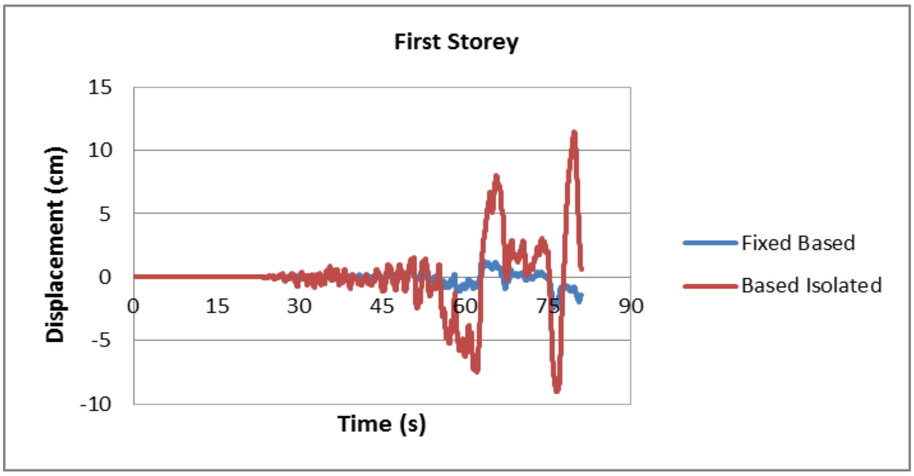
RESULTS AND DISCUSSION

4.1 General

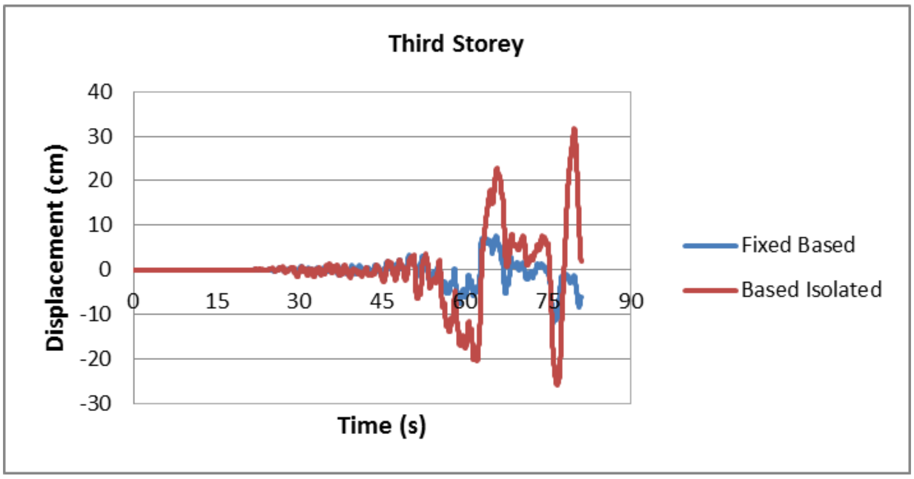
In this chapter, the results of the existing frame (fixed base) and base isolated frame (with friction pendulum isolator) obtained from nonlinear time history analyses were provided and discussed comparatively. Performance characteristics in terms of storey displacement, interstorey drift ratio, base shear, and the storey bending moment were given below.

4.2 The difference in storey displacement

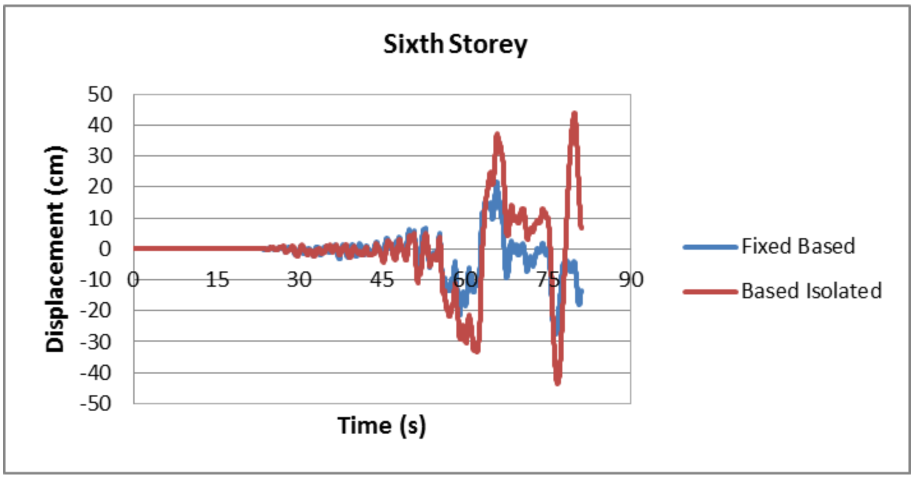
It was found that the displacement of existing and base isolated reinforced concrete (RC) frames increased through the height of the structures. The maximum displacement was obviously appeared in the top storey of the buildings. However, the amount of the displacement varied, depending mainly on the characteristics of the ground motion used. In Figures 4.1-4.4, the displacement time history of the first, third, sixth, and ninth storeys for each case of ground motions are illustrated.



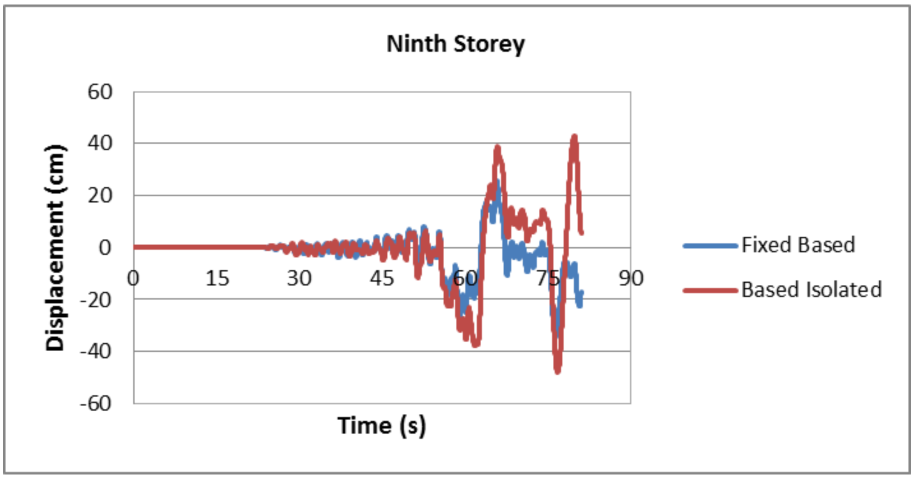
a)



b)

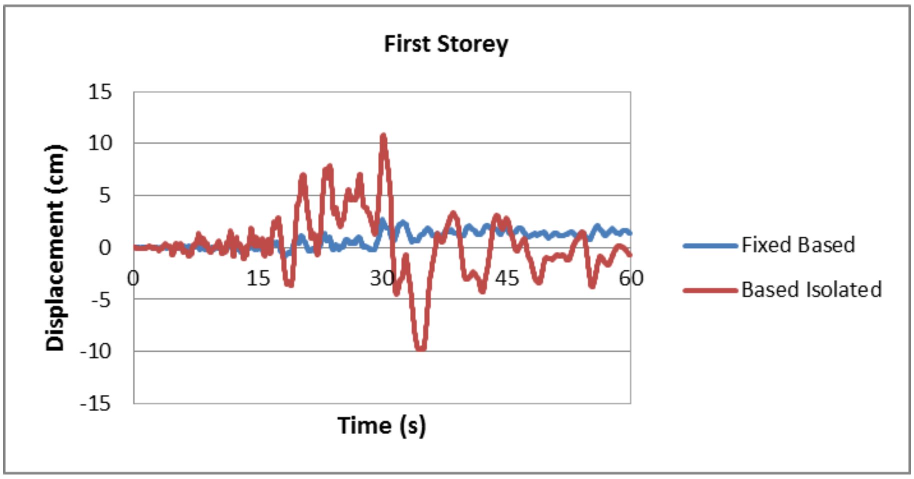


c)

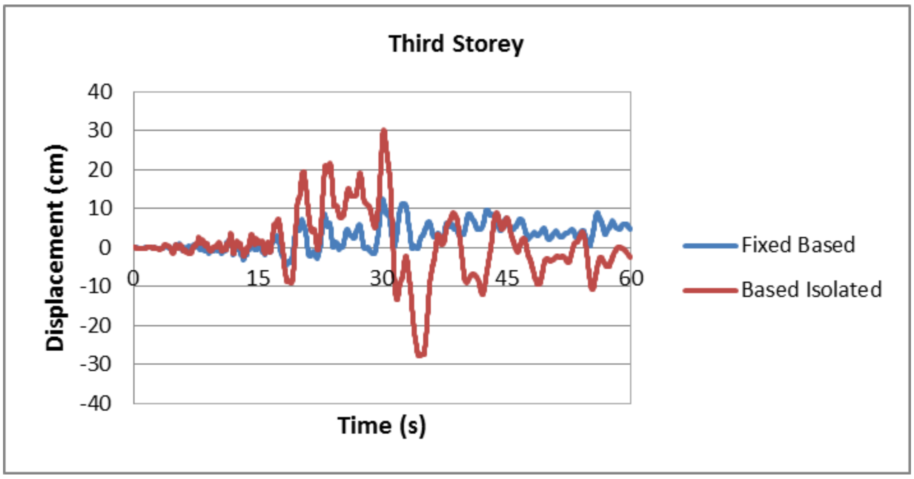


d)

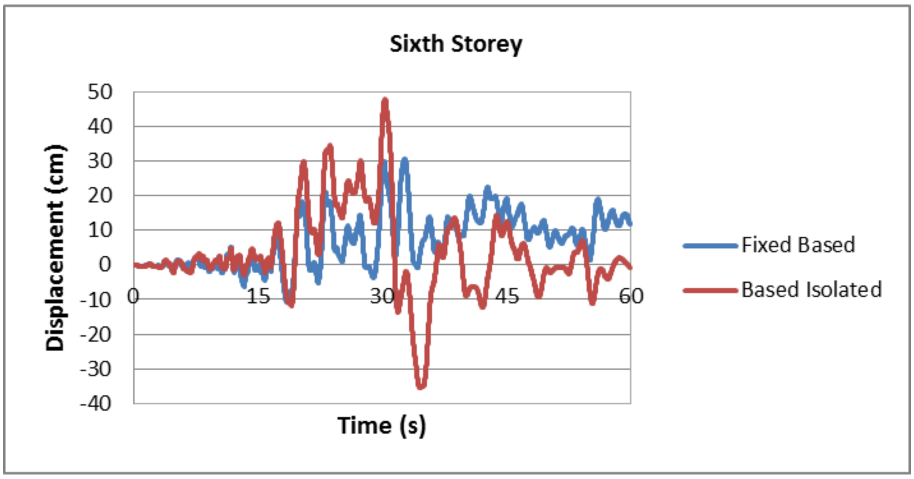
Figure 4.1 Variation of the storey displacement with time for a) first, b) third, c) sixth, and d) ninth storeys of RC frames under Hector Mine earthquake


First Storey
15
10
5
0
-5
-10
-15
Displacement (cm)
0
15
30
45
60
Time (s)
Fixed Based
Based Isolated

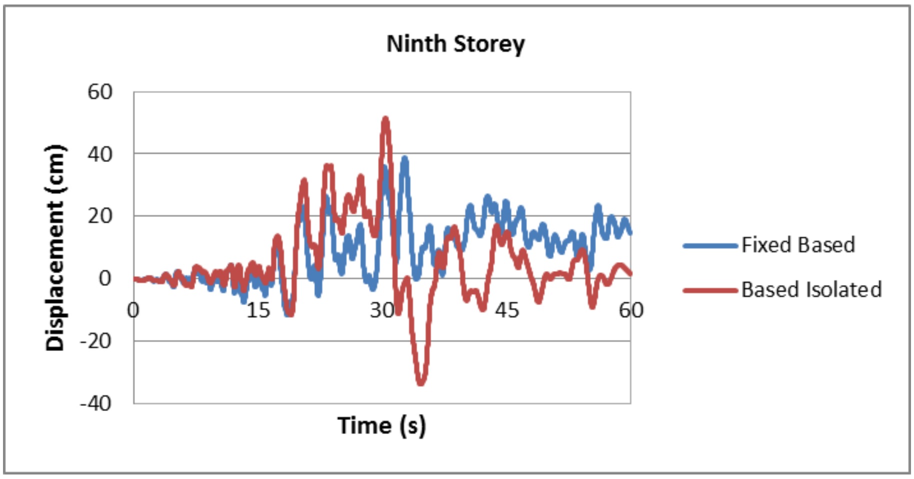
a)


Third Storey
40
30
20
10
0
-10
-20
-30
-40
Displacement (cm)
0
15
30
45
60
Time (s)
Fixed Based
Based Isolated

b)

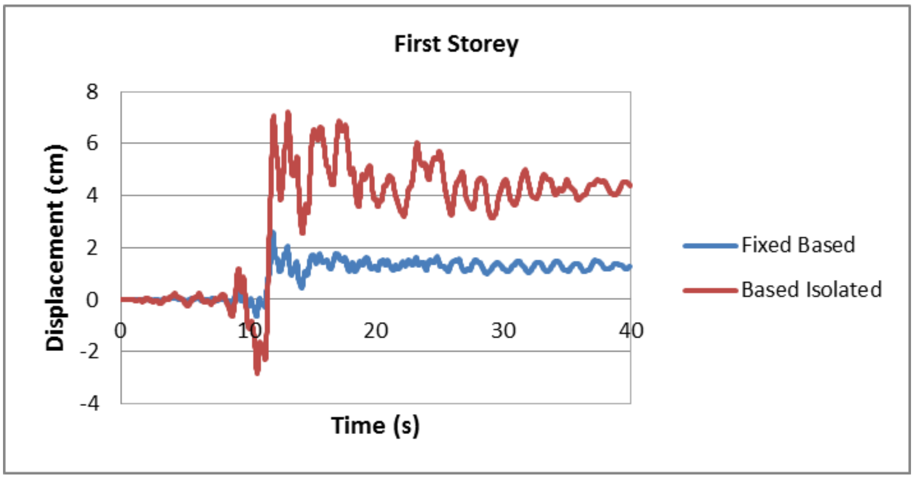

Sixth Storey
50
40
30
20
10
0
-10
-20
-30
-40
Displacement (cm)
0
15
30
45
60
Time (s)
Fixed Based
Based Isolated

c)

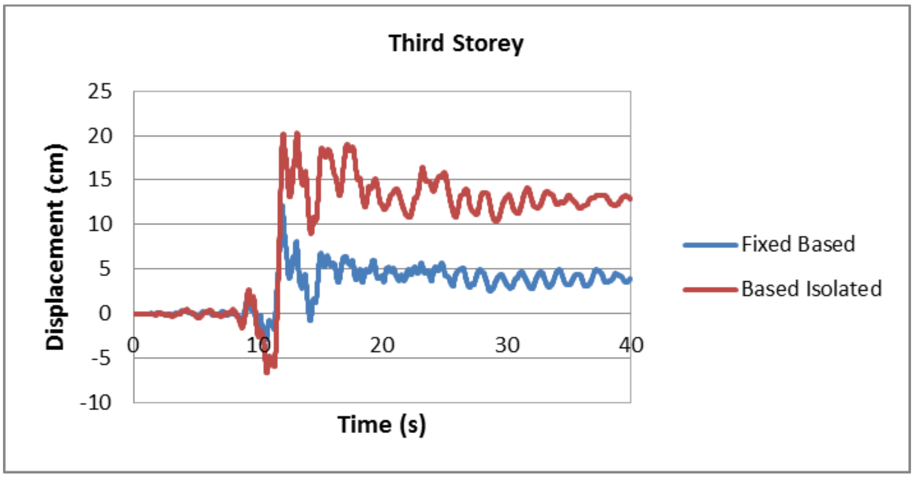


d)

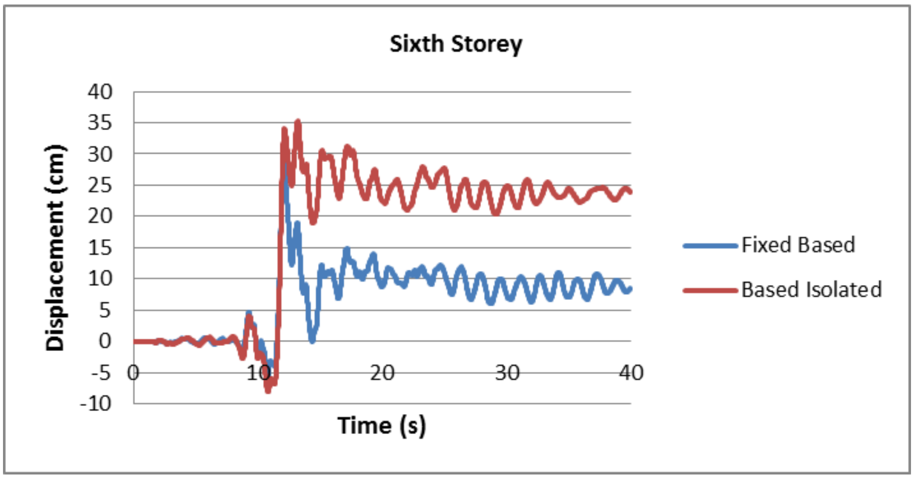
Figure 4.2 Variation of the storey displacement with time for a) first, b) third, c) sixth, and d) ninth storeys of RC frames under Morgan Hill earthquake



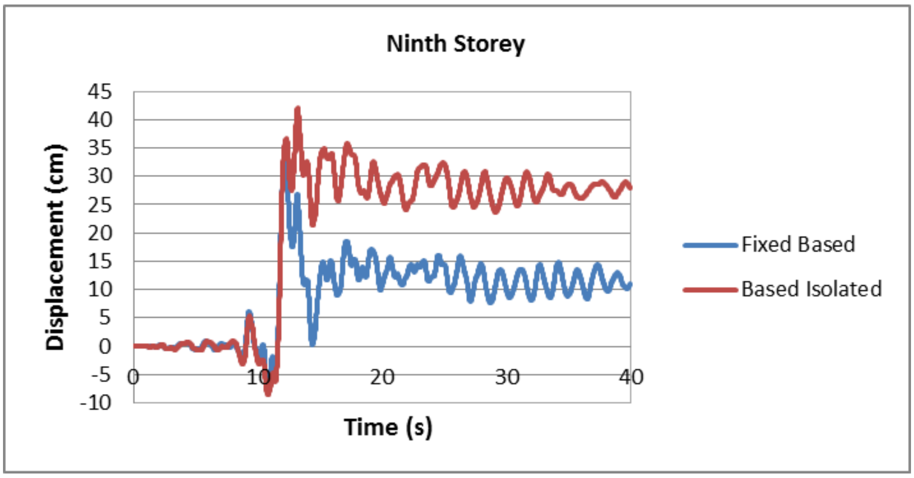
a)



b)

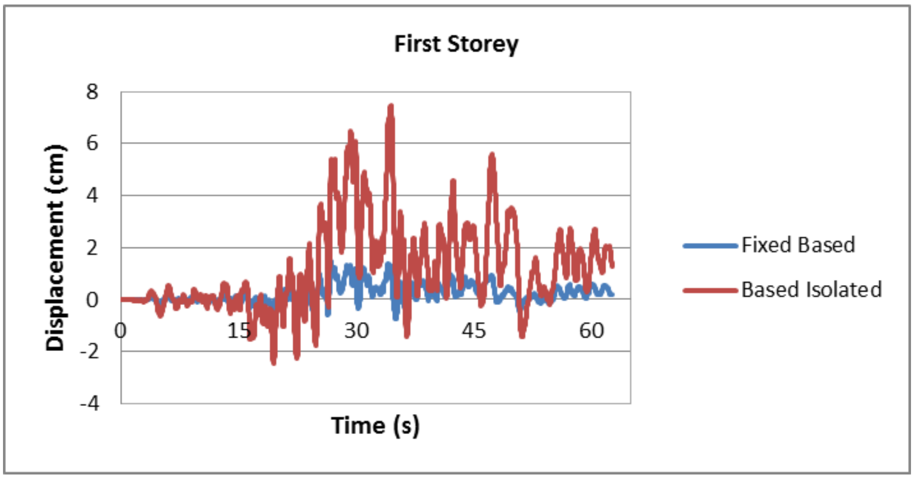


c)

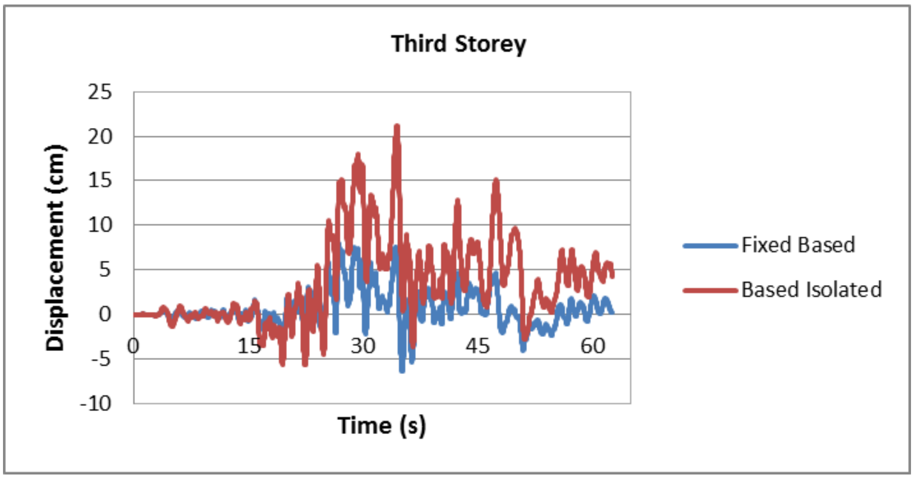


d)

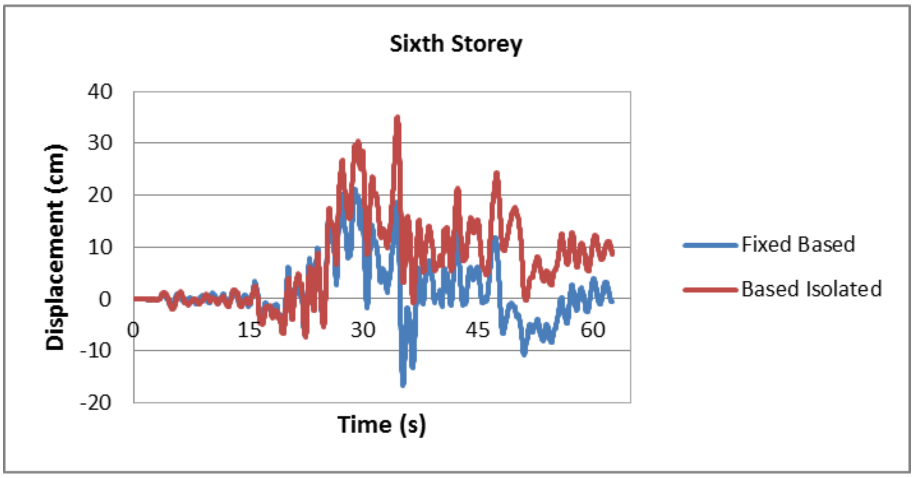
Figure 4.3 Variation of the storey displacement with time for a) first, b) third, c) sixth, and d) ninth storeys of RC frames under Loma Prieta earthquake



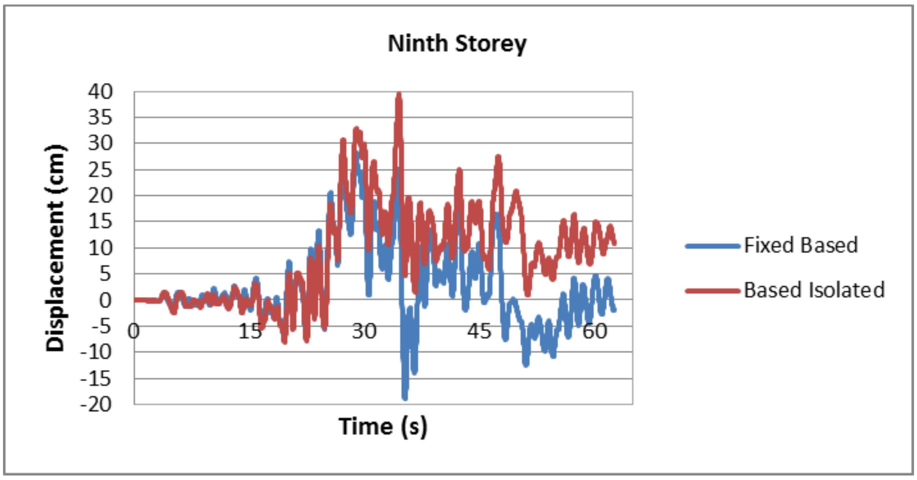
a)



b)



c)



d)

Figure 4.4 Variation of the storey displacement with time for a) first, b) third, c) sixth, and d) ninth storeys of RC frames under Landers earthquake

In previous figures, it was observed that all structures (after installed with base isolation for all earthquakes) would be dramatically affected. Moreover, the use of a friction pendulum base isolator in the structure increased significantly the value of the maximum displacements of each storey. The frame type and the number of storey also influenced the maximum storey displacement. It was obvious that in the ninth storey of the base isolated model, the value of the maximum displacement was greater than the other storeys when the amount of the displacements were compared in each storey.

From the nonlinear time history analysis, the results showed that the base isolated structure had remarkably higher storey displacements than the existing frame (fixed base case) for all earthquake ground motions, for example;

1. Figure 4.1 compared the maximum displacement of the fixed and isolated frames when subjected to Hector Mine earthquake, the maximum value in the ninth storey was 34.24 cm for the fixed frame and 47.94 cm for the isolated frame.

2. Figure 4.2 showed the results of the frames subjected to Morgan Hill earthquake, the maximum displacement value was 38.82 cm for the fixed frame and 51.37 cm for the isolated frame.

3. Figure 4.3 revealed the results of the frames subjected to Loma Prieta earthquake, the maximum displacement value was 33.94 cm for the fixed frame and 42.00 cm for the isolated frame.

4. Figure 4.4 illustrated that the result of the frames subjected to Landers earthquake, the maximum displacement value was 28.40 cm for the fixed frame and 39.50 cm for the isolated frame.

As a result, it was pointed out that the use of the friction pendulum isolation in the frames under the given earthquake record increased significantly the maximum displacement in comparison to the original frames. From this high level of displacement, there was no probability to occur damage during this type of a major earthquake. Furthermore, in Appendix, all the deformed shapes of the structures with and without base isolation were given for comparison purposes.

4.3 The interstorey drift ratio

Storey drift is an important indicator of structural behaviour in the performance based seismic analysis. To get the drift ratio, the value of the displacement results from the nonlinear time history analysis is divided by the storey height. All plots for the drift ratio for the fixed base and base isolated frames are given in Figures 4.5 to 4.8. The analysis of the results indicated that using isolation systems decreased significantly the values of drift ratio of the structure under the all earthquake ground motions. For instance, as seen in Figure 4.5, for the RC building under the Hector Mine earthquake, the maximum drift ratio of the fixed base frame was achieved as 0.26% while the maximum drift ratio of the base isolated frames was obtained as 0.16%. In addition, as seen in Figure 4.6 for the building under the Morgan Hill earthquake, the maximum drift ratio of the fixed base frame was found as 0.24% while the maximum drift ratio of the base isolated frames was obtained as 0.21%. Also in Figure 4.7, for the building under the Loma Prieta earthquake, the maximum drift ratio of the fixed base frame was achieved as 0.28% whereas the maximum drift ratio of the base isolated frames were obtained as 0.24%. And in Figure 4.8 for the building under the Landers earthquake, the maximum drift ratio of the fixed base frame was found as 0.51% while the maximum drift ratio of the base isolated frame was obtained as 0.21%. In the previous figures, it could be observed that the use of base isolation systems would reduce dramatically the drifts in the frames. On the other hand, in the case of frames with base isolated, the storey drift demands were significantly lower.

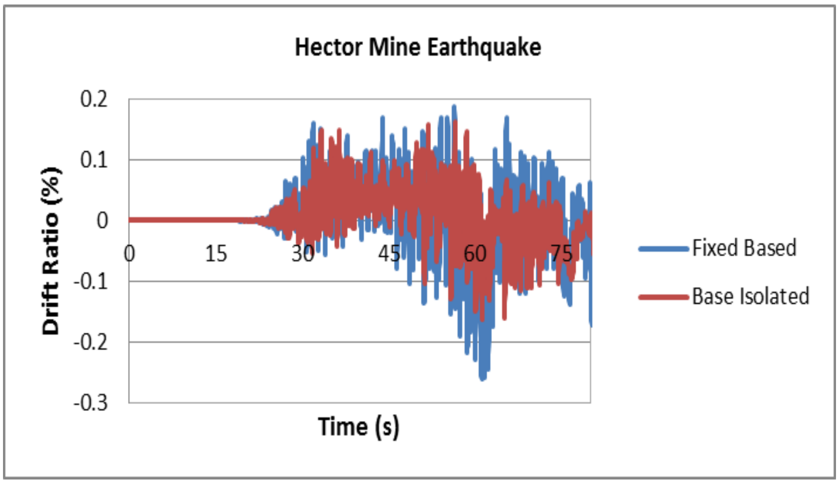


Figure 4.5 Drift ratio for top storey under Hector Mine earthquake

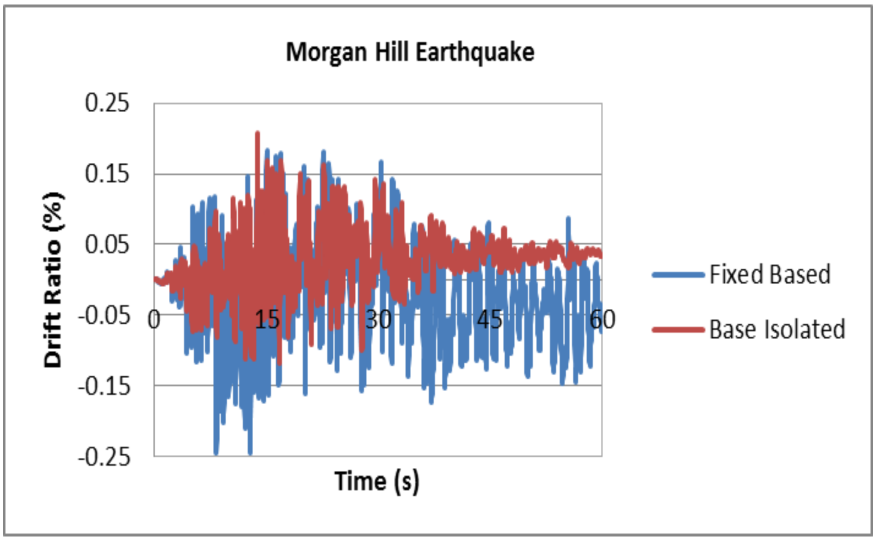


Figure 4.6 Drift ratio for top storey under Morgan Hill earthquake

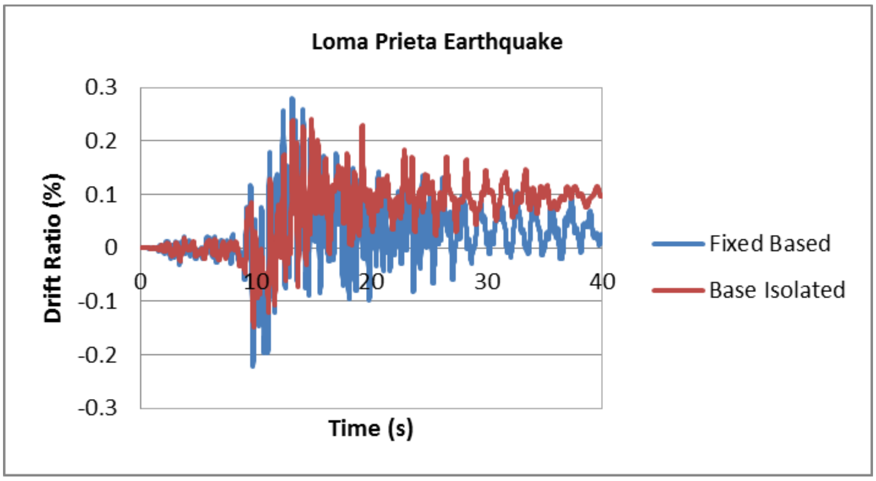


Figure 4.7 Drift ratio for top storey under Loma Prieta earthquake

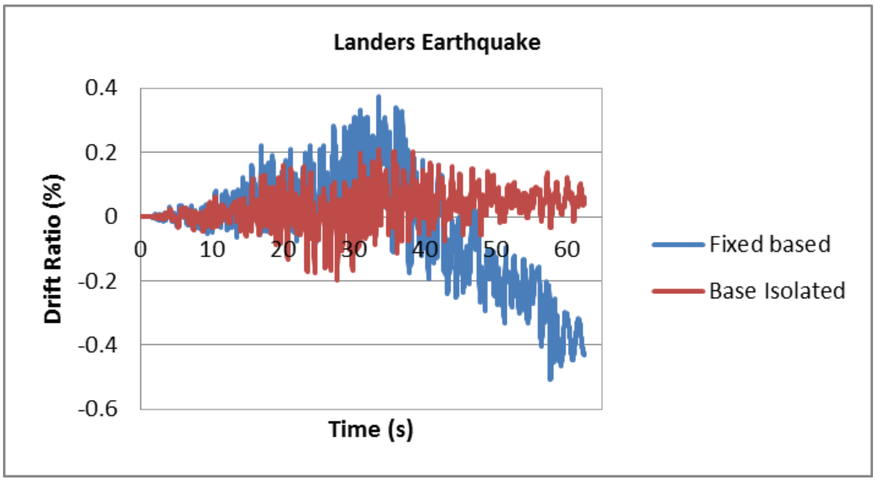


Figure 4.8 Drift ratio for top storey under Landers earthquake

4.4 The Base shear

The base shear was evaluated for different types of frames. The base shear could be described as the capacity of the structure to resist the lateral loads. Figures 4.9 to 4.12 show the comparison of the base shear for the fixed base and base isolated frames. As expected, a significant improvement was observed in the seismic performance of the original frame when choosing the right strategy for retrofitting. The use of a friction pendulum isolator dramatically reduced the value of base shear of the frames in different seismic performance levels. As seen from the figures, the frame with base isolation had considerably smaller base shear value in comparison to the original frame with fixed base. For example, for the building under Hector Mine earthquake (Figure 4.9), the maximum base shear of the original frame was about 471 kN while the frame with friction pendulum isolator was approximately 288 kN. Moreover, the time history analysis by using Morgan Hill earthquake resulted in the maximum base shear of about 431 kN for the original frame while in the base isolated frame was about 266 kN. Furthermore, for the frame under Loma Prieta earthquake (Figure 4.11), the maximum base shear of the original frame was evaluated as about 546 kN while in the base isolated frame was about 353 kN. Similarly, for the frame under Landers earthquake (Figure 4.12), the maximum base shear of the original frame was about 582 kN while in the base isolated frame was on average 367 kN.

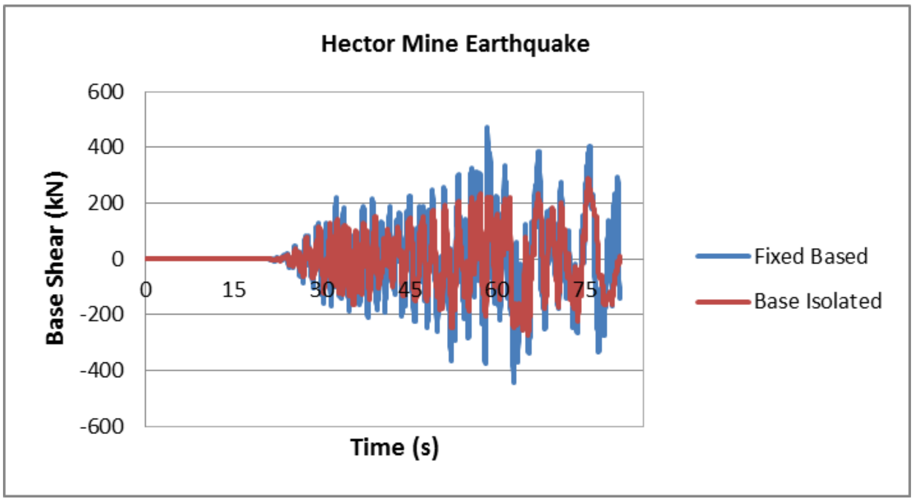


Figure 4.9 Base shear vs. time for the different frames under Hector Mine earthquake

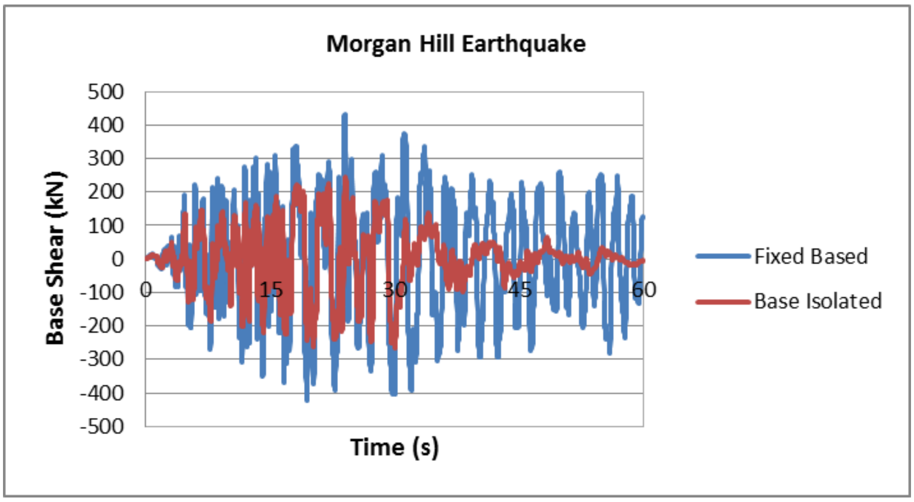


Figure 4.10 Base shear vs. time for the different frames under Morgan Hill earthquake

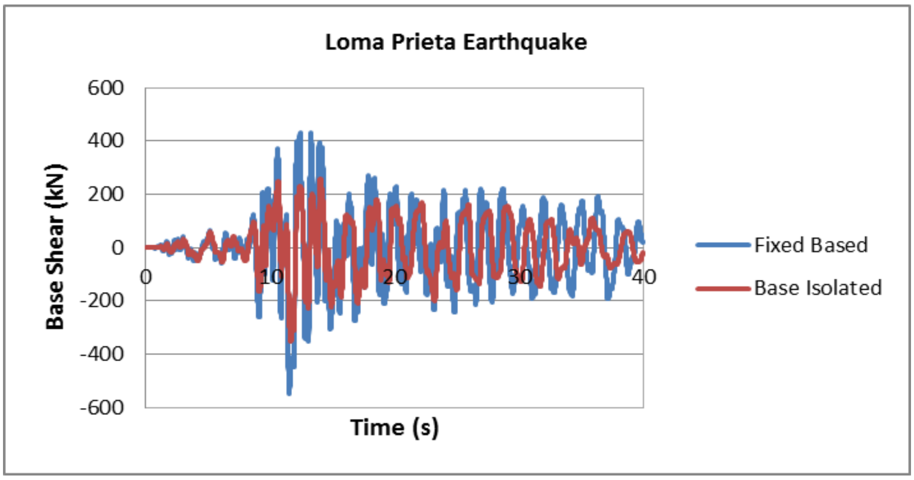


Figure 4.11 Base shear vs. time for the different frames under Loma Prieta earthquake

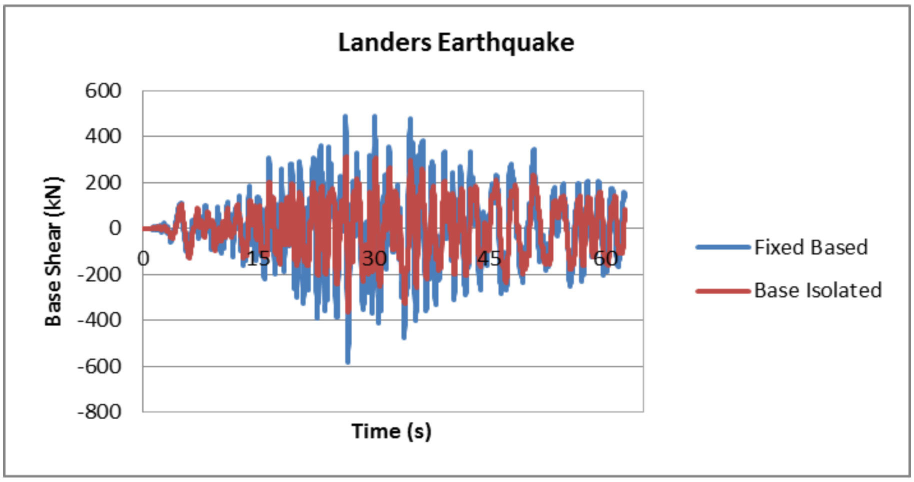


Figure 4.12 Base shear vs. time for the different frames under Landers earthquake

So, from the previous figures, it could be concluded that the structural behaviour of the frame with base isolation in terms of the base shear was better than the fixed based for all earthquakes. The difference between the maximum base shear of the fixed base and base isolated cases can be observed clearly in Figure 4.13.

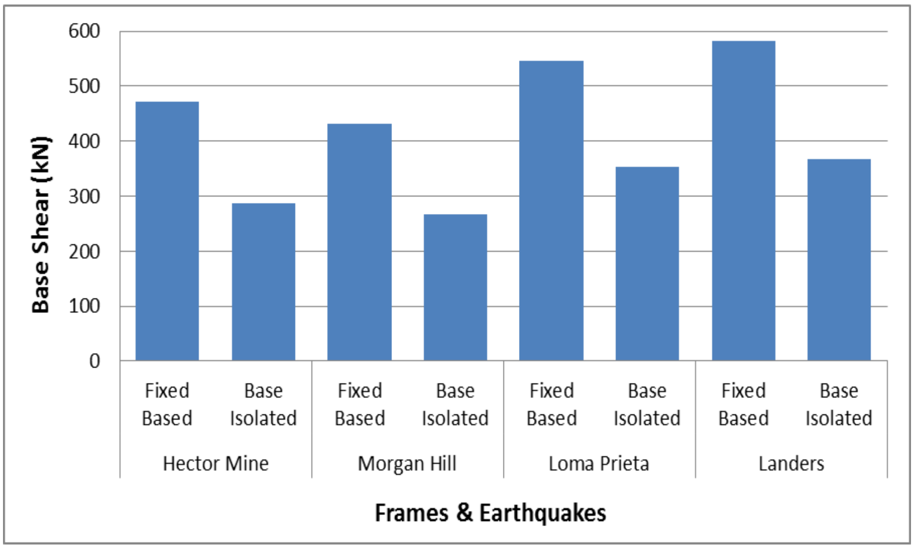


Figure 4.13 Comparison of the maximum base shear values for the fixed base and base isolated frames under analyzed earthquakes

4.5 The Storey bending moment

Bending moments are also significant parameters and required during the design. Figures 4.14 to 4.17 show the comparison of the maximum moment for the frames with fixed base and base isolation under different earthquake ground motions. The analysis of the results exhibited that the use of isolation systems decreased considerably the values of bending moment of the existing frame under all earthquake ground motions. For instance, as seen in Figure 4.14, for the building under the Hector Mine earthquake, the maximum moment of the fixed base frame was achieved as 50.1 kNm while the maximum moment of the base isolated frame was obtained as 38.4 kNm. In addition, as observed in Figure 4.15, for the building under the Morgan Hill earthquake, the maximum moment of the fixed base frame was found as 49.4 kNm while the maximum moment of the base isolated frames was obtained as 41 kNm. Also in Figure 4.16, for the building under the Loma Prieta earthquake, the maximum moment of the fixed base frame was achieved as 49.3 kNm whereas the maximum moment of the base isolated frame was obtained as 43.9 kNm. Furthermore, in

Figure 4.17, for the building under the Landers earthquake, the maximum bending moment of the fixed base frame was found as 57.7 kNm while the maximum moment of the base isolated frame was obtained as 48.9 kNm. As a result, under the impact of earthquakes such as Hector Mine, Morgan Hill, Loma Prieta and Landers, the bending moments decreased significantly when only the base isolation was applied.

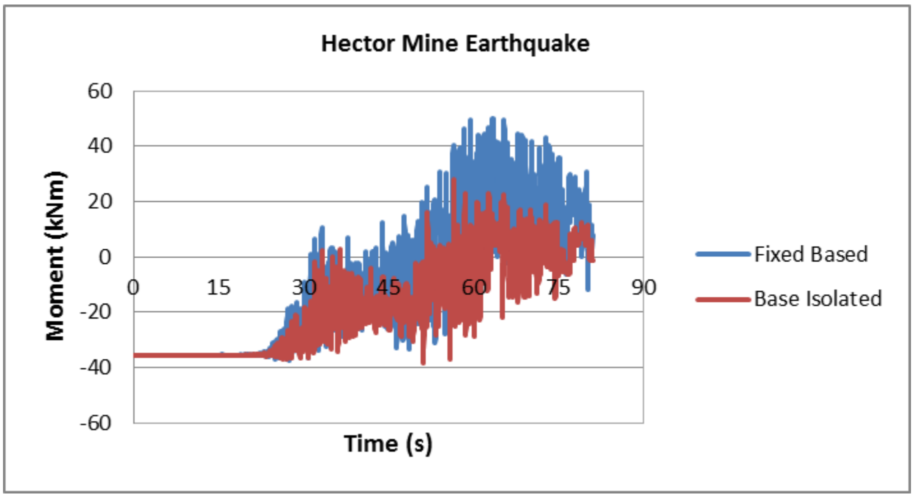


Figure 4.14 Variation in the bending moment of a top storey column with time under Hector Mine earthquake

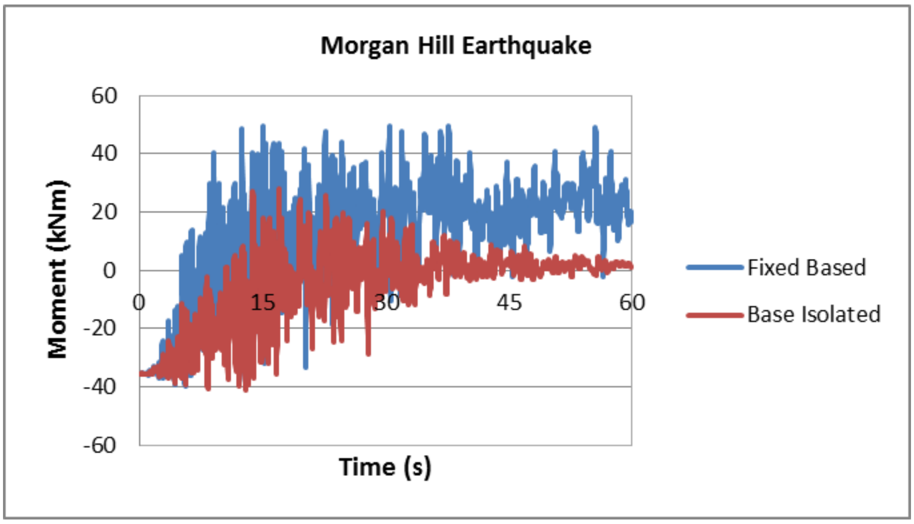


Figure 4.15 Variation in the bending moment of a top storey column with time under Morgan Hill earthquake

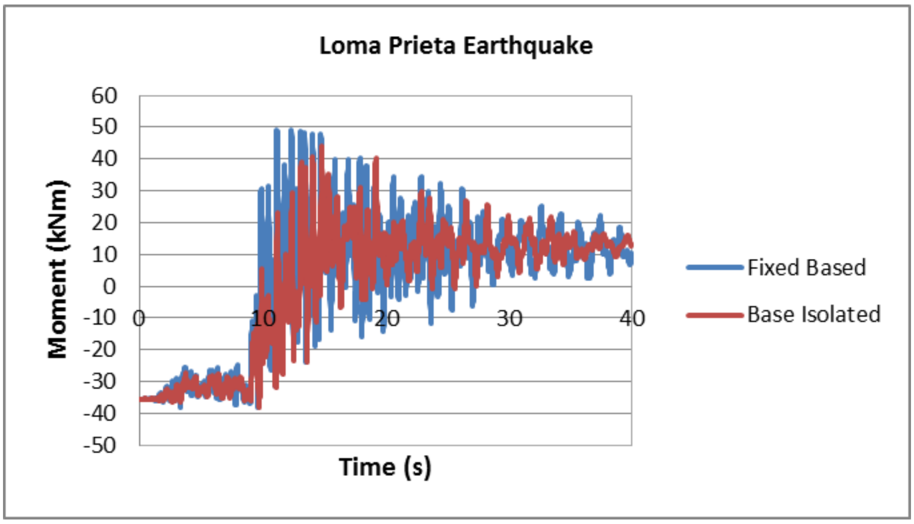


Figure 4.16 Variation in the bending moment of a top storey column with time under Loma Prieta earthquake

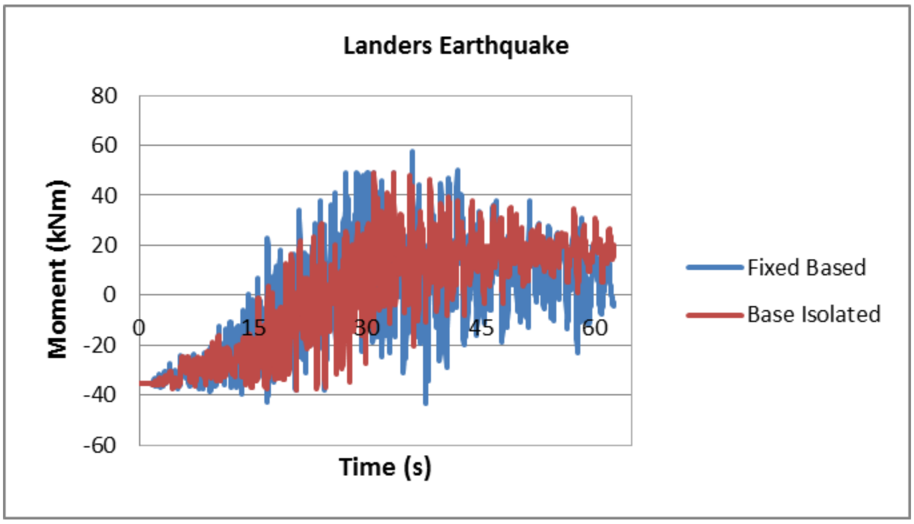


Figure 4.17 Variation in the bending moment of a top storey column with time under Landers earthquak

CHAPTER 5

CONCLUSIONS

This study achieved the modelling, analyzing, and designing of the fixed and base isolated RC frames through the nonlinear time history analysis. The performance characteristics were evaluated in terms of storey displacement, drift ratio, base shear, and storey bending moment. The base isolation system of friction pendulum elements was chosen to seismically isolate the frame having 9 storey levels. From the analysis of the results obtained in this study, the following conclusions can be drawn:

- The storey displacement was observed to be increased due to the inclusion of base isolation into the existing frame. The value of the maximum storey displacement was also increased with increasing the number of storey of the frames.

- In the analysis of interstorey drift ratio, the results showed a remarkable reduction in the drift ratio, especially in the top storey of the base isolated frames. For all earthquakes, the analysis results indicated that the base isolated frame performed better than the fixed base frame in the term of drift ratio.

- The base shear value of the existing frame had a tendency to decrease significantly after using the base isolation system. The base isolated frame had an ability to absorb more energy under the earthquakes through increasing the value of displacement and thus would reduce the generation of the lateral forces on the structure and then diminish the risk of failure in the frames. So, the frame with base isolation was better than the frame with fixed base to be used in the terms of the base shear.

- It was observed that there was a significant difference in the maximum base shear of the fixed base and base isolated frames. For example, for Hector Mine earthquake, the percent difference was about 39%, in Morgan Hill earthquake, the percent difference was about 38%, for Loma Prieta earthquake, it was about 35%, and in Landers earthquake, such value was on an average 37%.

- The analysis of the results of the storey bending moment revealed that using the base isolation as a retrofitted system reduced the values of the bending moment significantly in the top storey in comparison to the fixed base frame. This also returns to the ability of isolators to dissipate the seismic energy that expected from the superstructure.

- It was also pointed out that with the use of base isolation about 23%, 17%, 11%, and 15% reductions in the top storey bending moment of the column were obtained under Hector Mine, Morgan Hill, Loma Prieta, and Landers earthquakes, respectively. This indicated that the base isolated frame had better structural responce than the fixed based frame in terms of the storey bending moment under all ground motion records.

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APPENDIX

Appendix A: Deflected shapes

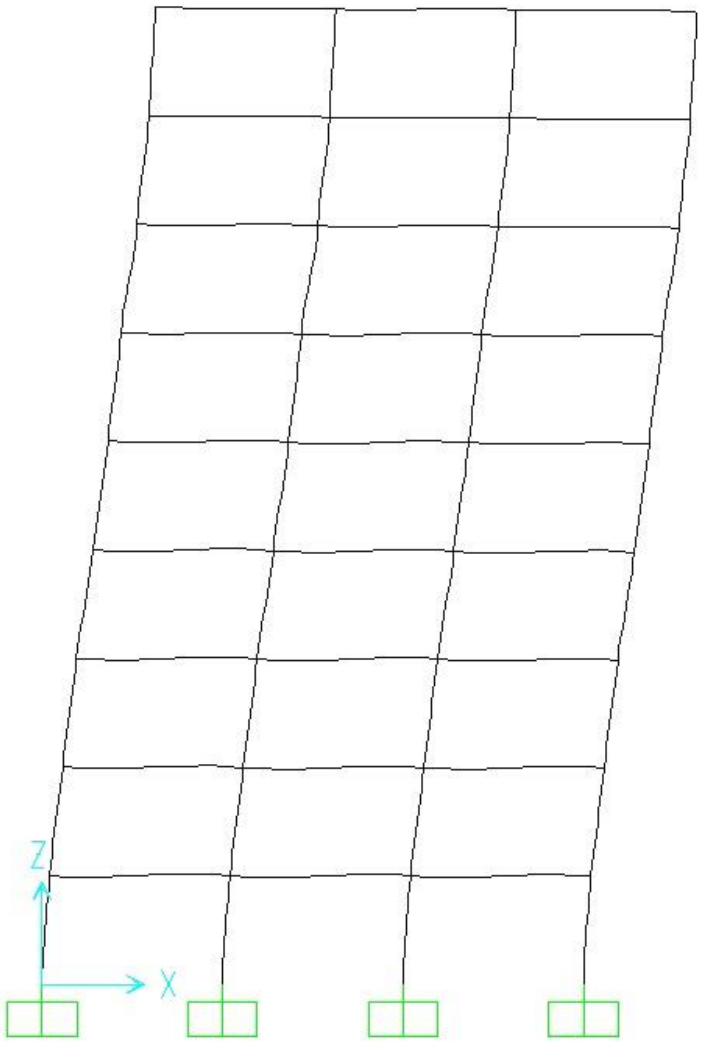

Figure A1 Mode shape of fixed base frame at T1=1.5 s

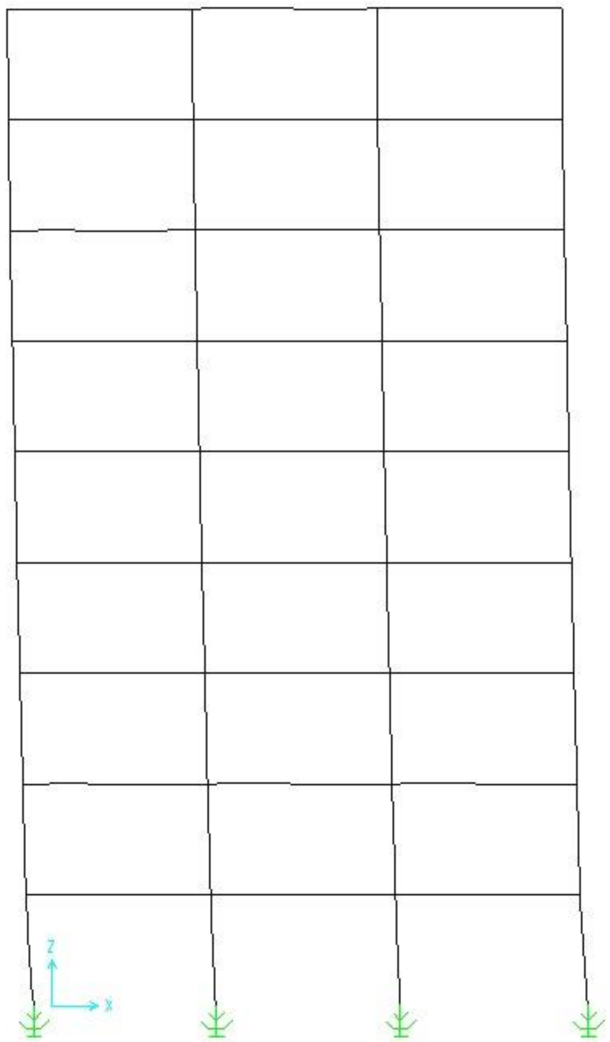

Figure A2 Mode shape of base isolated frame at T1=2.5 s

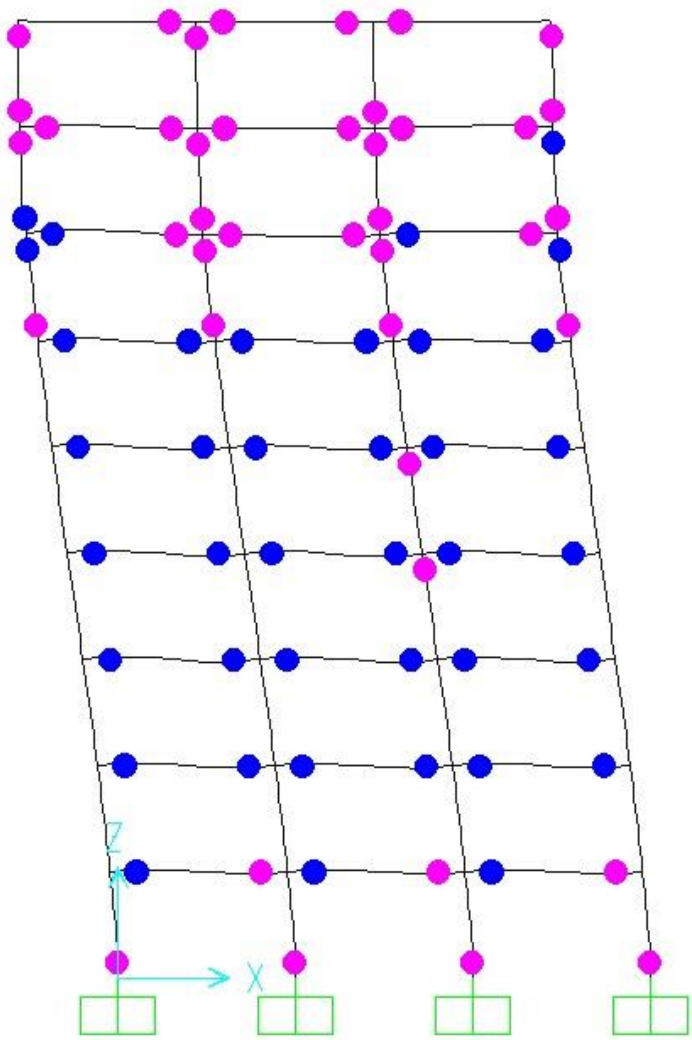

Figure A3 View of hinge formation of the fixed base frame for the Hector Mine earthquake

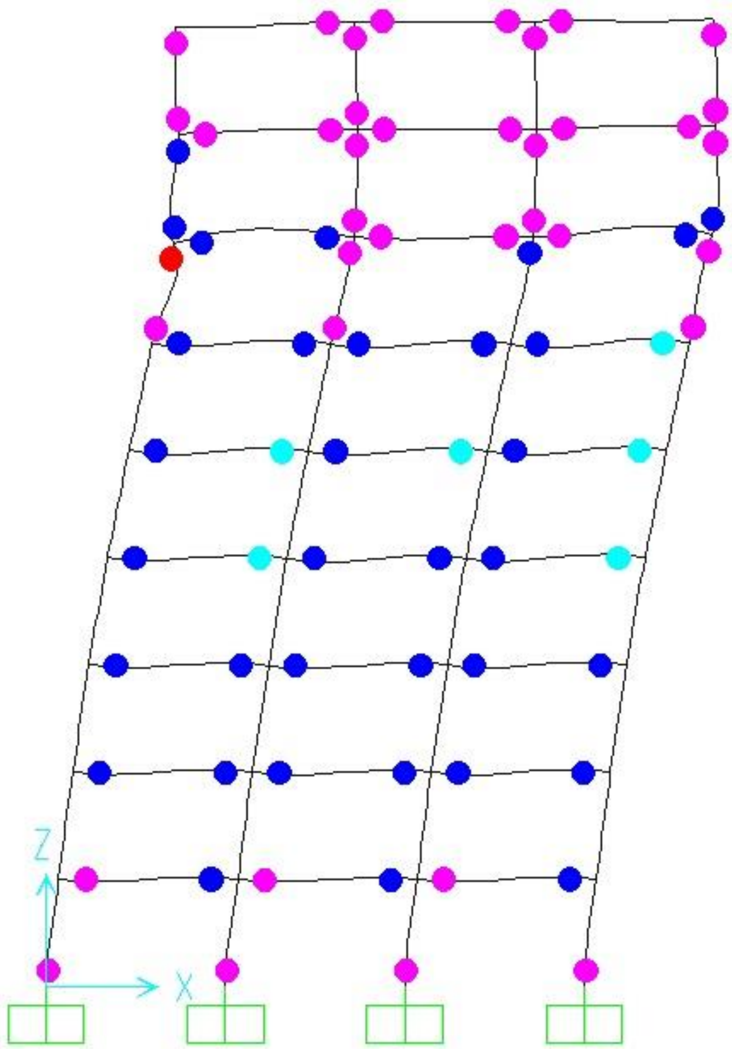

Figure A4 View of hinge formation of the fixed base frame for the Morgan Hill earthquake

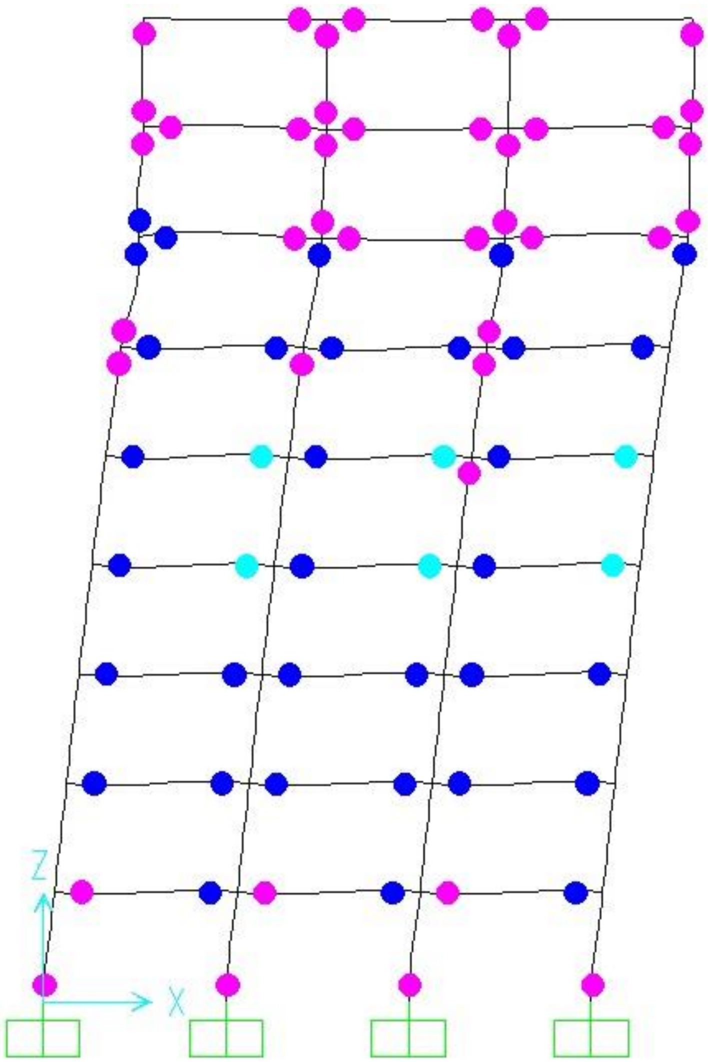

Figure A5 View of hinge formation of the fixed base frame for the Loma Prieta earthquake

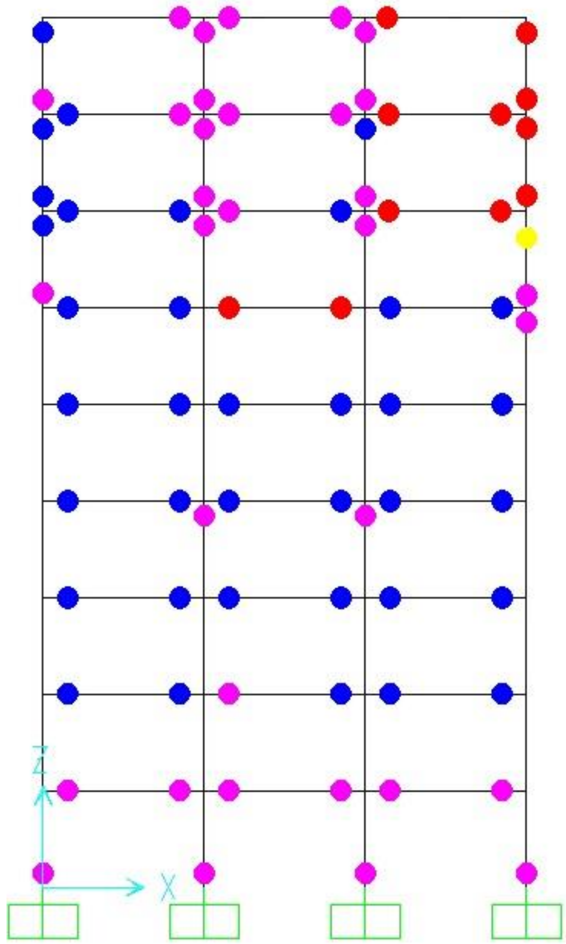

Figure A6 View of hinge formation of the fixed base frame for the Landers earthquake