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EARTHQUAKE BEHAVIOUR OF REINFORCED CONCRETE BUILDINGS WITH METALLIC DAMPERS

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Hevi Mustafa ABBAS

ABSTRACT

EARTHQUAKE BEHAVIOUR OF REINFORCED CONCRETE BUILDINGS WITH METALLIC DAMPERS

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M.Sc. in Civil Engineering
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Among the passive energy dissipation systems, one of the most used types is the metallic dampers such as Added Damping and Stiffness (ADAS) elements. In this study, the performance of reinforced concrete structures, which are regular in shape and symmetrical in plan, equipped with ADAS dampers in inverted V- bracing systems were investigated. For this, 6, 9 and 12 storey framed structures having three bays were used. The structural behaviours of the studied frames were determined by conducting the nonlinear static and time history analysis. A group of natural earthquake acceleration records compatible with seismic hazard levels of 2%, 10%, and 50% probabilities of exceedance in 50 years was utilized in the dynamic analysis. The results of the analysis in terms of the capacity curves of the structures, maximum storey displacement and interstorey drift demands, displacement time history, hysteretic behaviour, and plastic hinge formation were evaluated. The results exhibited that the ADAS dampers in inverted V bracing systems were effective in earthquake energy dissipation as observed in the behavior of the frames in the elastic region under the earthquakes compatible with the seismic hazard of 10% and 50% probability of exceedance in 50 years.

Keywords: Damper, Frame, Nonlinear analysis, Seismic behaviour, Seismic protection.

ÖZET

METALİK SÖNÜMLEYİCİLİ BETONARME BİNALARIN DEPREM DAVRANIŞI

ABBAS, Hevi Mustafa
Yüksek Lisans Tezi, İnşaat Mühendisliği
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Pasif enerji sönümleyici sistemler arasında en çok kullanılan türlerden biri, ADAS (Added Damping and Stiffness) elemanları gibi metalik sönümleyicilerdir. Bu çalışmada, ters-V çaprazlı sistemlerinde ADAS sönümleyicileriyle donatılmış düzenli şekil ve simetrik plana sahip betonarme yapıların performansları incelenmiştir. Bunun için, üç açıklığa sahip 6, 9 ve 12 katlı çerçeveler kullanılmıştır. Çalışılan çerçevelerin yapısal davranışları doğrusal olmayan statik ve zaman tanım alanında hesap yöntemleri kullanılarak tespit edilmiştir. Dinamik analizlerde, 50 yılda aşılma olasılığı %2, %10 ve %50 sismik tehlike seviyeleriyle uyumlu bir grup doğal deprem ivme kayıtları kullanılmıştır. Yapıların kapasite eğrileri, maksimum kat yer değiştirme ve katlar arası ötelenme talepleri, zaman tanımalanında yer değiştirme, histeretik davranış ve plastik mafsal dağılımları açısından analiz sonuçları değerlendirilmiştir. Sonuçlar, ters-V sistemlerinde ADAS sönümleyicilerinin 50 yılda aşılma olasılığı %10 ve %50 sismik tehlike seviyeleri ile uyumlu depremler altında, çerçeveninelastik bölgede davranışlarından da görüldüğü üzere, deprem enerjisini sönümlemede etkili olduğunu sergilemiştir.

Anahtar Kelimeler: Sönümleyici, Çerçeve, Doğrusal olmayan analiz, Sismik davranış, Sismik koruma.

To my beloved Father, Mother, and Sisters …

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LIST OF SYMBOLS/ ABBREVATIONS

ADAS	Added damping and stiffness
BRB	Buckling restrained brace
CYB	Confined yielding brace
FEMA	Federal emergency management agency
IVBF	Inverted-V braced frame
M3	Flexural moment hinges
Mw	The magnitude of ground motion
PGA	Peak ground acceleration
PMM	Axial force-biaxial moment hinges
RC	Reinforced concrete
TADAS	Triangular added damping and stiffness

CHAPTER 1

INTRODUCTION

1.1 General

In the past, structures were designed to resist small lateral loads by elastic behavior. During moderate to high earthquakes, building codes were intended mainly to provide life safety by preventing the building from collapsing. However, the design for life safety offered by current codes is fundamental but not sufficient. The seismic risk has increased in recent major earthquakes and the damage caused is far from being economic. In order to reduce structural damage and repair costs after severe earthquakes, a need for a design that controls structural and nonstructural damage is required. The key to achieving this, passive energy dampers were developed (Abdulrahman, 2014).

Metallic dampers is one of the oldest and most broadly used passive energy dissipation devices due to inelastic deformation of metals that can be effective to decrease dynamic response of structures subjected to strong earthquake excitations. The idea of using metallic energy dissipaters in design of earthquake goes back almost three decades began with the conceptual and investigational work by Kelly et al. in 1972 and Skinner et al. in 1975 (De la Llera, 2004; Bagheri, 2011; Wang et al., 2014). During the ensuing time, significant progress was made in the development of metallic dampers and proposed several new designs (Constantinou et al., 1998).

Metallic devices can be categorized into flexural types like hourglass shape (ADAS) and triangular shape (TADAS); shear types like YSPD, and axial types like buckling retrained brace (BRB). Devices are generally designed to be incorporated into the lateral-load-resisting system in the structural frames, but some are located between the beam and columns. Design of the metallic dampers involves many coveted engineering characteristics like as possessing adequate elastic strength and stiffness, such that device is not excited to inelastic region under service loads; having stable and large energy dissipative capability; and having reasonable resistance against low-cycle fatigue (Ghabraie et al., 2010).

One of the most used types of metallic dampers is Added Damping and Stiffness (ADAS) elements, and variations such as TADAS, they are among the displacement-dependent energy dissipation devices (Constantinou et al., 1998; Ghooch and Farokh, 2014). ADAS dampers are an assembly of steel plates that are designed to dissipate the energy through the flexural yielding deformation of mild-steel plates (Constantinou et al., 1998; Ghooch and Farokh, 2014). These parallel plate devices are typically installed in a building frame in such a way that the clamped end of the steel plates is attached to the top of a chevron bracing arrangement while the other end of the plates is attached to the floor slab above the bracing (Benedetti et al., 2014). ADAS and triangular metallic dampers based on the use of low-yield metals with triangular and hourglass shapes. These shapes are chosen to prevalence yielding throughout the device, thus the initial stiffness will be increasing (Wang et al., 2014).

The ADAS damper is adopted so as to have a constant strain variation over its height, thus ensuring that yielding happens concurrently and regularly over the full height of the damper (Bakre et al., 2006). ADAS dampers are a hysteretic device because its energy dissipation depends primarily on relative displacement within the device and not on its relative velocities. Generally metals have linear damping in the case when the stress amplitude fewer than the fatigue boundary and the hysteresis loop is an ellipsis (Orban, 2011; Bakre and Pujari, 2012).

The first structural achievement of metallic dampers took place in New Zealand, which got significant seismic response reduction of the structure. Another instance is that of the 6 storey Cardiology Hospital complex constructed in Mexico City in the 1970s, which suffered from many damage and the fall down of some part of the structures throughout the 1985 Mexico Earthquake. After the incident, the structure was retrofitted with 18 external steel trussed buttresses linked to the building floors through 90 ADAS devices. Nonlinear analytical results showed significant decrease in both inter-storey drift and base shear of the retrofitted structure, resulting from the combined effect of stiffening and increased energy dissipation through the ADAS devices (Gang and Hong, 2007). The ADAS elements also applied in U.S. for seismic retrofitting of a building in San Francisco in early 1992. Other configurations of ADAS devices, used mostly in Japan consist of bending type of honeycomb and slit dampers and shear panel type (Bayat and Abdollahzade, 2010).

Kelly and his colleagues in Berkeley, California, have examined the energy dissipation of dampers ADAS in a three story building on earthquake simulator. A more extensive experiment has been done by Whittaker et al. (1991) and his colleagues in Berkeley. Furthermore, Hanson and Xia have done a couple of numerical studies on parameters of ADAS in the shape of X. Tsai et al. (1993) and his colleagues also have done some experiments about numerical and lab studies on triangle parameters of ADAS (Khatoni et al., 2015). Due to possess more stiffness and damping compared with other devices like FVDs, the ADAS dampers were capable to raise the first floor stiffness by 30% (Hwang, 1997).

ADAS, TADAS, and rhombic damper are commonly installed in the direction of weak axis bending to provide energy absorption, whereas the other dampers (honeycomb, dual function DFMD, shear panel, and slit damper) absorb input energy of earthquakes in the direction of strong axis. Therefore, these dampers have a higher stiffness compared with ADAS or TADAS (Teruna, 2015). Other types of metallic dampers, such as lead and shape-memory alloys, have also been evaluated (Sakurai et al., 1992; Aiken et al., 1992). Some particularly desirable features of those devices are their stable hysteretic behavior, low-cycle fatigue property, long term reliability, and relative insensitivity to environmental temperature (Constantinou et al., 1998).

Also buckling restrained brace (BRB) is another type of metallic dampers that was defined by Calado et al. (2008). It is a bracing member consists of steel core plate or another section encased in steel tubes filled by concrete along its length. BRBs are tension-compression braces with hysteretic behavior. These braces have the ability to dissipate energy by yielding in tension and compression by resisting axial loads in the steel core and resisting buckling through the steel casing (Abdulrahman, 2014). Recent experimental testing of BRBs demonstrated their ability to provide a stable, repeatable and ductile response (Marshall and Charney, 2008).

1.2 Objective and scope

The aim of this study was to investigate the seismic response of the structures with inverted V- shaped braces and ADAS dampers. For that reason, 6-storey, 9-storey, and 12-storey reinforced concrete frame structures were designed. All the frame cases had the same plan and three bays on each direction. The finite element modeling and time history analysis were performed by means of SAP 2000 (CSI 2009). The seismic behaviour of the structures with inverted-V shaped braces and ADAS dampers were analyzed using different ground motions such as Cape Mendocino, Kocaeli, and Düzce earthquake records. Various parameters were considered to evaluate the seismic behaviour of the reinforced concrete frame structures. The finding of the study was discussed comparatively.

Chapter 1- Introduction: The aim and objective of the thesis are summarized.

Chapter 2- Literature Review: This chapter briefly gives the background on the previous studies about the damper and bracing systems, especially inverted V-bracing and ADAS damper included frames.

Chapter 3- Methodology: In the methodology, the analytical model of the structures with inverted V- braces and ADAS dampers are explained. The description of the analysis methods and the characteristics of the ground motion records are also provided.

Chapter 4- Results and Discussion: This chapter presents and compares the results obtained from the nonlinear static and dynamic analysis of each structural case.

Chapter 5- Conclusions: The conclusions are given in the light of findings from the overall results of the analysis.

CHAPTER 2

LITERATURE REVIEW

2.1 Damping in Structures

Damping is one among several methods that are able to dissipate vibration disturbances (Taylor, 1999). Damping is defined as the removal or dissipation of the vibration energy of a structure. The mechanism of damping is converting instantaneous strain and kinetic energy into heat and finally releasing it to the surrounding environment. In laboratory models which are a simple system the energy in most cases is released through repeated thermal stress impact of material as well as from the force of friction when a hard material starts to deform but in structures the energy can be dissipated by other methods. In a vibrating building the mechanisms could be a) Friction at the steel connections, b) Opening and closing of micro cracks in the concrete, c) Friction between the structural; and d) Non structural components such as the partition walls. The accuracy in conventional buildings of stiffness and hardness can be modeled with reasonable degree but not it's damping properties. Results of different experiments show that the damping forces are almost autonomous of frequency, which it is inconsistent with the inherent frequency, which depends on the viscous damping strength. Within lower levels of damping true viscous idealization is enough (Ibrahim, 2005).

2.1.1 Categories of Damping

Dampers can be categorized in many ways but the general categorization of dampers depends upon the classification of the material. The friction of the molecular depends on the force of the damping (known as structural or material damping). The surrounding medium is also a factor which classifies the vibration of the damping (known as nonmaterial damping). The percentage range of a structure with critical damping as an inherent damping is betweens (1%-5%); which can be counted as nonmaterial elements. A partition wall and outer claddings with cracks in reinforced concrete structures as well as structures composed of steel and wood. Table 2.1 shows the suggested damping ratios that used in different types of buildings (Ibrahim, 2005).

Table 2.1 Suggested values for damping ratios for various buildings (Ibrahim, 2005)

Stress Level	Type and Condition of Structures	Damping Ratio (%)
Working stress, less than ½ yield point	▪ Welded steel, prestressed concrete, well reinforced concrete (slight cracking).	2-3
	▪ Reinforced concrete with considerable cracking.	3-5
	▪ Bolted and/or riveted steel, wood structures with nailed or bolted joints.	5-7
At or just below yield point	▪ Welded steel, prestressed concrete without complete loss in prestress.	5-7
	▪ Prestressed concrete with no prestress left.	7-10
	▪ Reinforced concrete.	7-10
	▪ Bolted and/or riveted steel, wood structures with nailed or bolted joints.	10-15
	▪ Wood structures with nailed joints.	15-20

2.1.1.1 Material Damping (Internal damping)

Material damping depends on many factors: The materials type, stress amplitude, internal forces, the number of cycles, sizes of geometry, the quality of surfaces, and temperature. Damping fundamentally depends on the stress amplitude as:

$$D = J \cdot \sigma\, a^{n} \tag{2.1}$$

Where n and J are constants, D is energy dissipates per cycle, σ is stress amplitude, and a is the number of cycles.

The value of n can be between 2 and 4 but generally n= 2.3 is used. Damping increases with the number of cycles and finally a fatigue collapse happens. The J values are very different for the same material in accordance with different authors (Orban, 2011).

2.1.1.2 Nonmaterial Damping (External Damping)

Nonmaterial damping comes from limit effects. An important form is structural damping, which is produced by contacting of two dry surfaces that often occur between structural components such as joints like coulomb damping, or by the vibration response of a structure like acoustic radiation damping (Nashif et al., 1985).

a) Acoustic Radiation Damping

The structure vibration response depends on the event of its surrounding fluid, such as the atmosphere, stream or any other oil or gas or liquid. This method leads to important alterations in the form of natural frequencies and shapes. The influence of damping fluid density depends on its density and speed of sound waves in liquid and solid, and the mass of the structures itself (Nashif et al., 1985).

b) Coulomb Friction Damping

Also called dry friction, is developed from the proportional motion of two dry surfaces. These contacting surfaces are modeled by constant force relative to the normal load between the contacting planes and direction of the overall forces is generally versus the velocity vector at each moment. The damping force is nonlinear because it depends on the relative velocity. The coulomb damping is generally applied at joints. These dampers are representing a high rise energy dissipation potential in the joints at low cost and are easy to install and maintain (Nashif et al., 1985).

2.1.2 Dampers In Different Locations

Situating the damper in different locations of structures due to the different dynamic loads would increase the performance and accuracy of it. The dampers could be located in the following places in the structures (Ibrahim, 2005):

a) In a series and in parallel to an isolation systems

This is ideal for structures with a large displacement at its base.

b) Diagonal elements

This case is useful and effective for refurbishments where the damper is installed like a conventional diagonal brace.

c) Chevron bracing

The damper can be set up on both chevron brace legs which give the same effectiveness of as the diagonal brace.

d) Horizontal elements on top or bottom of chevron bracing

Locating the damper either at the bottom or top of a chevron braces horizontally.

e) Horizontally between adjacent structures

In order to prevent pounding between close structures, the dampers should be set up between both of the structures horizontally.

f) Toggle brace

Linking the damper to two steel linkage elements in addition to putting it diagonally is the so called Toggle brace. The effect of the damper increases once any inter story is drift is amplified in it direction. Not long time ago the Toggle –brace damper was used in United States. During the construction of the 37-storey Yerba Buena tower in San Francisco and 37-storey Millenium Place in Boston the so called Reverse Toggle brace damper was used in order to reduce the wind – induced vibration. This system was used by the two Engineers named Charney and McNarma in 2002 during the construction of the 39-story office building (Zhang et al., 2012; Huang, 2004).

g) The scissor-jack-damper system

The scissor-jack-damper is similar to the toggle damper but with a higher compactness. Testing the scissor-jack configuration on an earthquake simulator on a steel framed model structure shows a higher effectiveness. Beside its small size, the scissor-jack damper system resulted in more accurate damping and the seismic response of the examined structure went down as well. For understanding the scissor-jack damper better it is useful reviewing the conventional chevron and diagonal brace configuration, in which the dissipation of the device displacement is either less than (case of diagonal brace) or equal to (case of chevron brace) the drift of the story at which the devices are put on. Figure 2.1 illustrates the above mentioned locations of dampers (Sigaher and Constantinou, 2004).

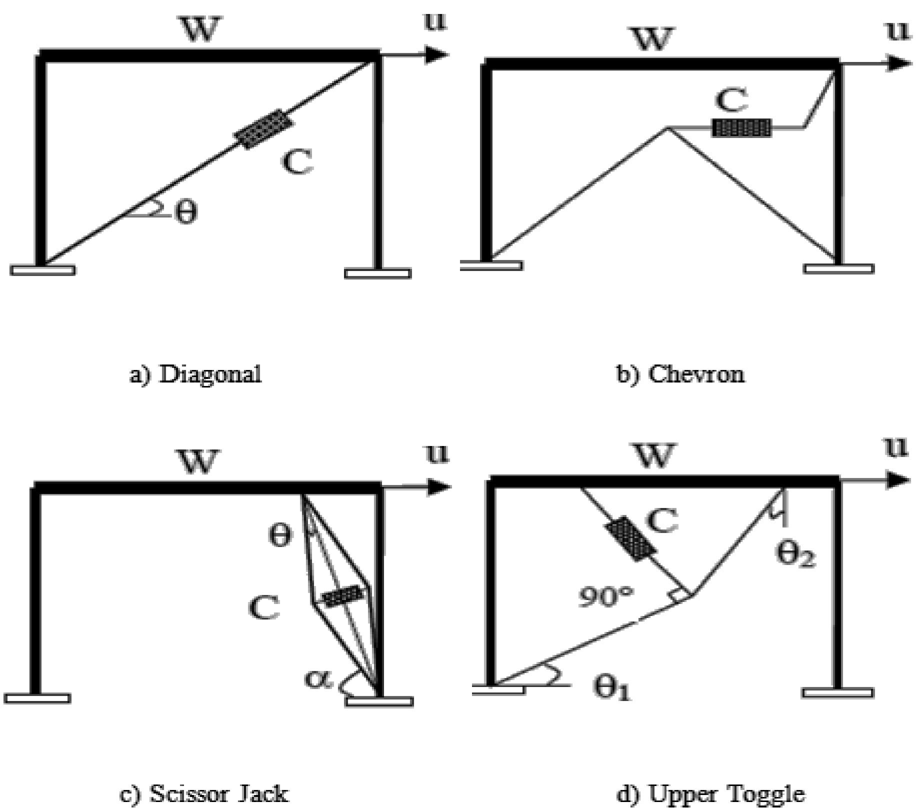


Figure 2.1 Different damper placements and their effectiveness (Sigaher and Constantinou, 2004)

2.2 Passive Energy Systems

To avoid damage to the structure elements, subsequently the 1989 Loma Prieta and 1994 Northridge seismic excitations stimulate the evolution of passive energy dissipation systems (Ramirez et al., 2002). Many researchers developed and applied passive energy dissipation devices (Li, Nan Li, 2013). Under moderate to high earthquakes, structures without energy dissipation can cause strong damage in all the structure components (Ibrahim, 2005). These devices are beneficial to use because they develop the internal damping through the energy dissipated by the inelastic deformation of these special devices (Colunga and Vergara, 1997). There are several methods to improve the characteristic of the structures under such earthquake events. Limit damaging deformations in structural elements, and repair costs is the main concept of this device; additionally these systems can minimize the ductility demand on the structure elements by reducing the accelerations and shear forces. This can be attained by the base isolation system, and seismically-controlled system approaches (Ibrahim, 2005).

Base-isolation system is a technique developed to minimize damage to the structures by absorbing seismic input energy and reducing the initial shock of horizontal ground displacement and experience far lower acceleration and much less damage. Also it dissipates lateral forces transmitted to the system during an earthquake (Tongaokar and Jangid, 1998). To restrain any possible resonance at the isolation frequency some damping can be applied in conjunction with the base isolation systems. It has been applied in New Zealand, India, Japan, Italy, and the USA. In Figure 2.2, apparent comparison of responses exhibited by a fixed-base structure and an isolated structure is presented (Symans, 2009).

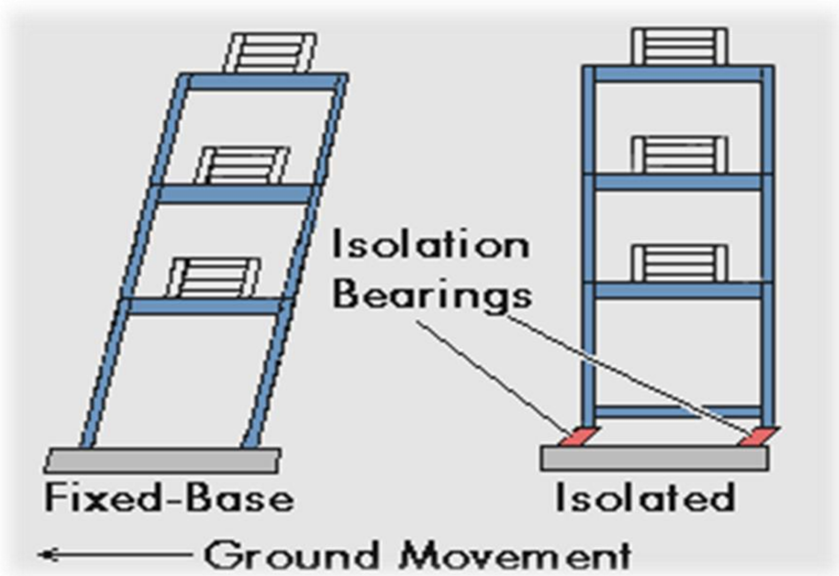


Figure 2.2 Base isolation system (Symans, 2009)

The first building obverses inter-story drift at upper floor levels, while the latter has deformation at the base level and uniform accelerations over the height of the structures (Symans, 2009). This system is suitable for low- to mid-height buildings (Chopra, 2007). Generally, there are several types of base isolation system (Abdulrahman, 2014):

- Elastomeric Bearings.
- Lead Rubber Bearings.
- Sliding Friction Pendulum Systems.
- The Double Concave Friction Pendulum.
- Triple Friction Pendulum.
- Combined Elastomeric and Sliding Bearings.
- Sliding Bearings with Restoring Force.

The seismically-controlled systems need power to develop the force that control seismic response of the structures by set of dissipating devices under different types of vibrations adequately located in the structure. In general, control systems devices can be classified into three classes (Soong and Spencer, 2000):

- Active control system.
- Semi-active control system.
- Passive control system.

Active and semi-active control systems works in another way from passive energy dissipation devise. These devices have the ability to prevent the structures from the seismic excitations. These systems works by sensor and actuator that needs a large external force to operate the actuators, but semi-active control systems needs a minimal provenance of power, the operation of the devise is the same as in active control system. Active and semi-active control systems are force delivery devices incorporated with real-time processing evaluators and sensor within the structures as defined by (Soong and Spencer, 2000).

Passive control systems are systems that require no external power sources and used to minimize the earthquake excitation response of the structures, these systems wherever base isolation and energy dissipating devices (or dynamic vibration). A diagram is shown in Figure 2.3 to simulate the mechanism of each system (Symans and Constantinou, 1999).

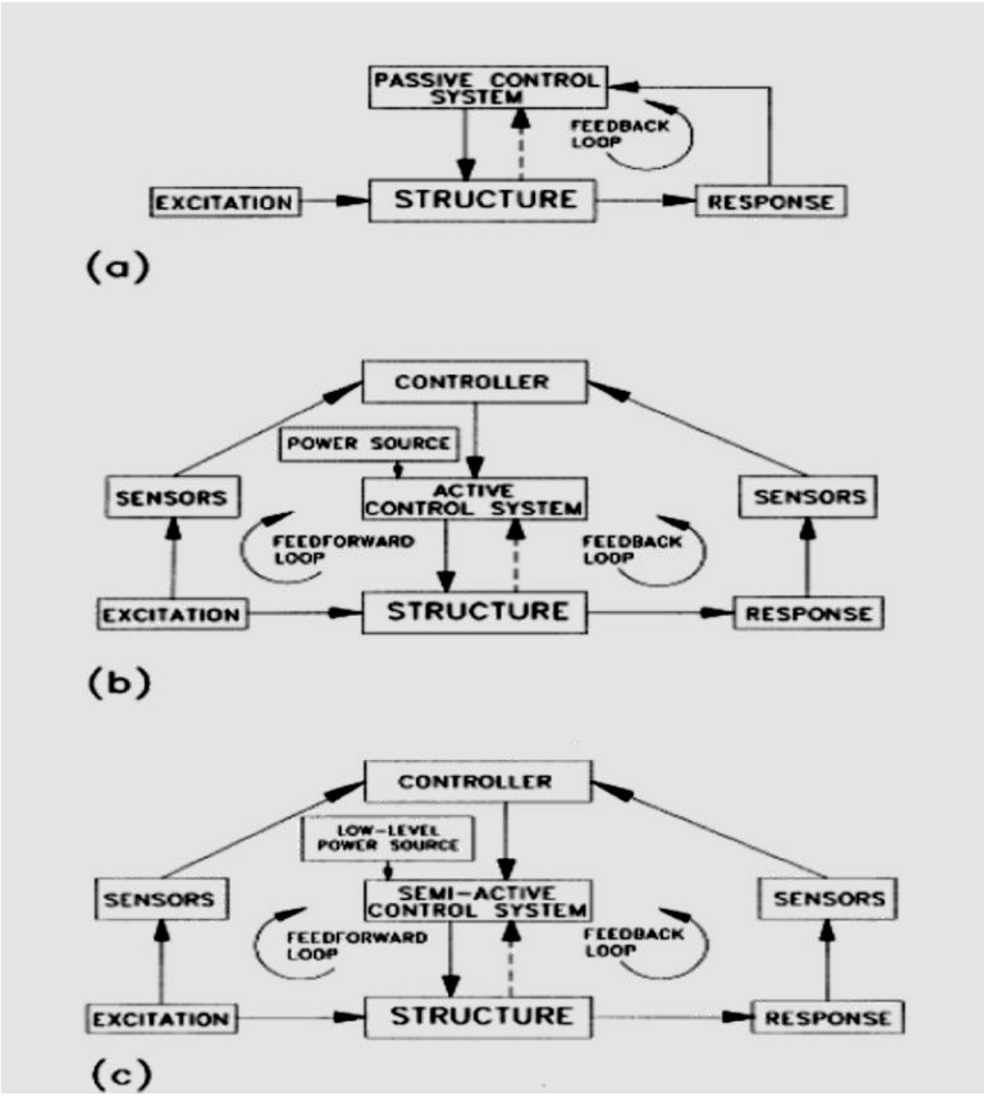


Figure 2.3 Diagram of structural control systems: (a) passive control system, (b) active control system, and (c) semi-active control system (Symans and Constantinou, 1999)

2.2.1 Groups of Passive Energy

2.2.1.1 Passive Dampers (with Direct Energy Dissipation)

The level of damping can be increased by passive system in a structure through a direct energy dissipation mechanism, through an orifice like a fluid with high viscosity or like (viscoelastic) by the shearing action of a polymeric / rubber material. Viscous damping devices (VDDs), friction system, and metallic dissipater are other class of passive system with a direct energy dissipater. The popularity of these mechanisms to structures has increased in nowadays. In Japan and as well as in the United States due to their advantage such that they need small area and it can be simply implemented into the existent frames. Their efficiency in areas with great amplitude and common earthquake has made them very useful and popular (Kareem et al., 1999).

a) Friction damper

Friction damper is one of the displacement-dependent devices. Keightley (1977) was the first one that study frictional dampers for structure implementations (Constantinou et al., 1998). After that, the friction damper was proposed by Pall (1979) to be utilized in structures for dissipating earthquake excitations, inspired from the concept of friction brakes which is applied in automobiles to minimize the motions of building as shown in Figure 2.4 (Nikam et al., 2014). The most advantages of these devices that have very high quality performance, easy to construct, large amount of energy dissipation ability, resistance to environmental temperature and their performance is fewer affected by applied load per cycles and frequency (Aiken, 1996). Friction dampers supply rectangular hysteresis loops which providing a fundamental increment in energy dissipation per cycles also minimizing drifts compared to moment resisting frames as shown in Figure 2.5 (Nikam et al., 2014). The devices vary in the materials used for sliding surfaces and also in their mechanical complexity (Aiken, 1996). Possible requirement for non-linear analysis due to non-linear behavior, permanent displacement in the absence of the restoration of the force and maintain their characteristics over time for long periods are the main disadvantages (Nikam et al., 2014).

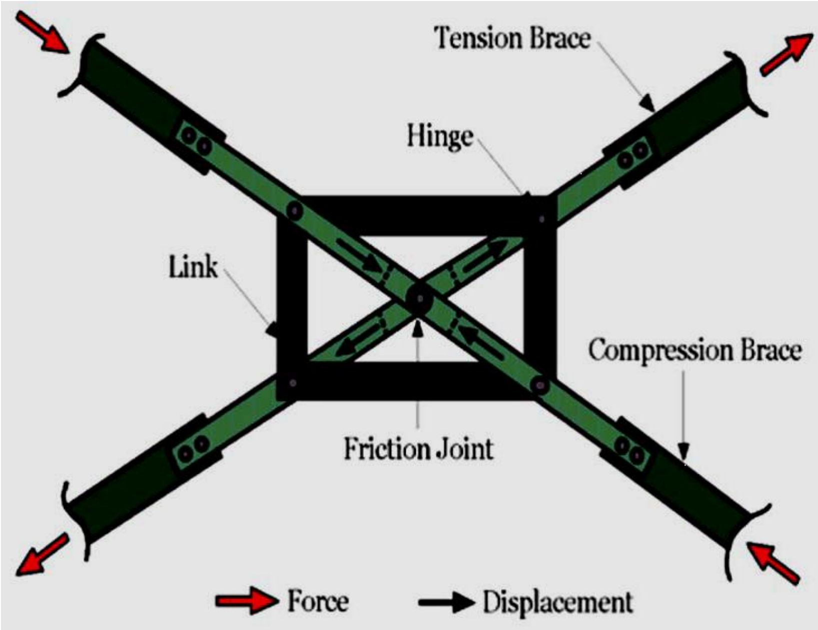


Figure 2.4 Pall friction dampers (Nikam et al., 2014)

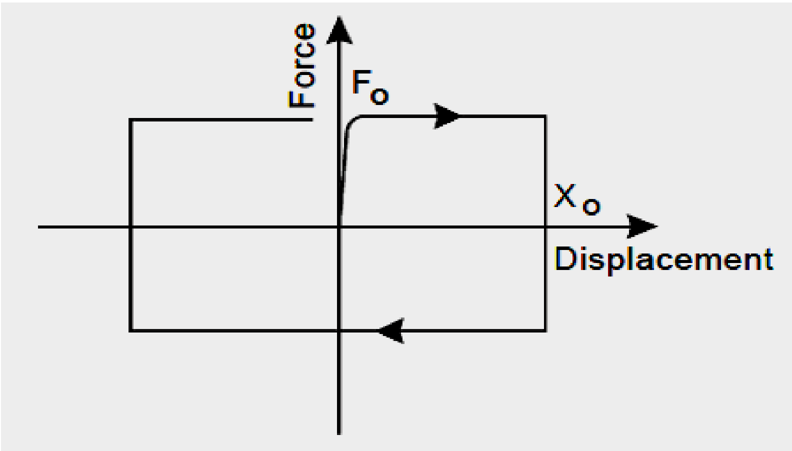


Figure 2.5 Hysteresis loop shapes for friction damper (Nikam et al., 2014)

Friction dampers appropriate for various types of construction have been developed for 1) shear walls of concrete, both precast and cast in place; 2) braced steel/concrete frames; 3) low-rise structures, and 4) clad-frame construction. Patented pall friction dampers are available for tension cross-bracing; single diagonal bracing; chevron bracing; cladding connections; and friction base isolators. These friction dampers meet a high standard of quality control. The friction dampers work on stick-slip phenomenon, in which slip load is the most significant parameter. Two most prominent types of the friction dampers which have successfully been utilized around the world are the Pall and the Sumitomo friction dampers. Sumitomo Metal

Industries of Japan developed friction dampers for railway applications. Recently, the application of these dampers was broadened to structural engineering. Many buildings in Japan integrate the Sumitomo friction dampers for earthquake protection. In general, the displacements are decreased in comparison to the moment resisting frames without these devices. However, the base shears and the accelerations are only altered; sometimes amplified due to the strengthening of new braces (Nikam et al., 2014).

Direct damping applications through friction systems allow plastic behavior through the provision of non-linearity while permitting the structure itself to remain elastic. These systems, carefully controlled by a sliding surface, feature a very large initial stiffness and the possibility of nearly perfect rectangular angular hysteretic behavior. Presently, such systems are in use in many buildings in Canada, which is characterized by friction braces and some in Japan that use of piston friction type dampers (Kareem et al., 1999).

b) Metallic damper

Metallic damper is the displacement-dependent devices which have the capability to dissipating energy, regular hysteric behavior, and resistance to thermal conditions. The resisting dampers force needs the non-linear analysis of the material and must be replaced after the earthquake can be shown as disadvantages of these devices (Sahin, 2014). The first hysteretic dampers were applied by Skinner et al. (1980) for seismic protection of buildings. It was applied to bridges and base isolated buildings. Metallic dampers were made up of mild steel plates (Soong and Spencer, 2002). They proposed special categories of these dampers like torsional beam, U-strip dampers and braced systems, some of these types are shown in Figure 2.6 (Constantinou et al., 1995). Moreover, the metal have linear damping when the stress amplitude fewer than the fatigue boundary as shown in Figure 2.7 (Orban, 2011).

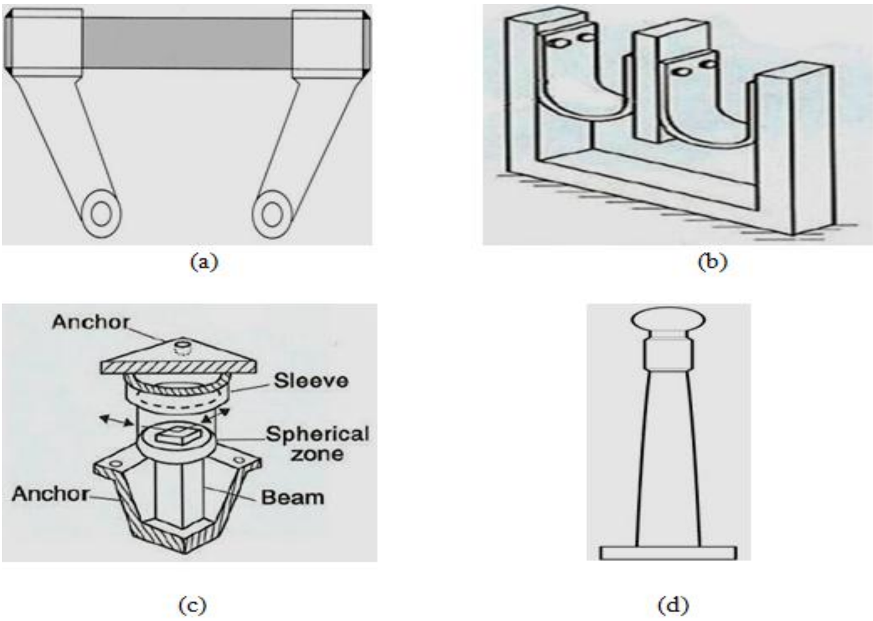


Figure 2.6 A number of hysteretic dampers a) Flexural beam damper (Type U) or uniform-moment bending-beam damper, b) U-strip damper, c) Flexural beam damper, and d) Steel cantilever (Type T) or Tapered rod damper (Constantinou et al., 1995)

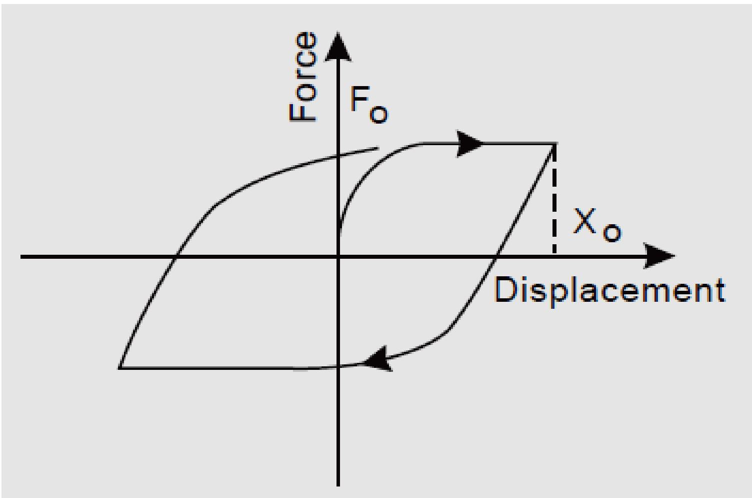
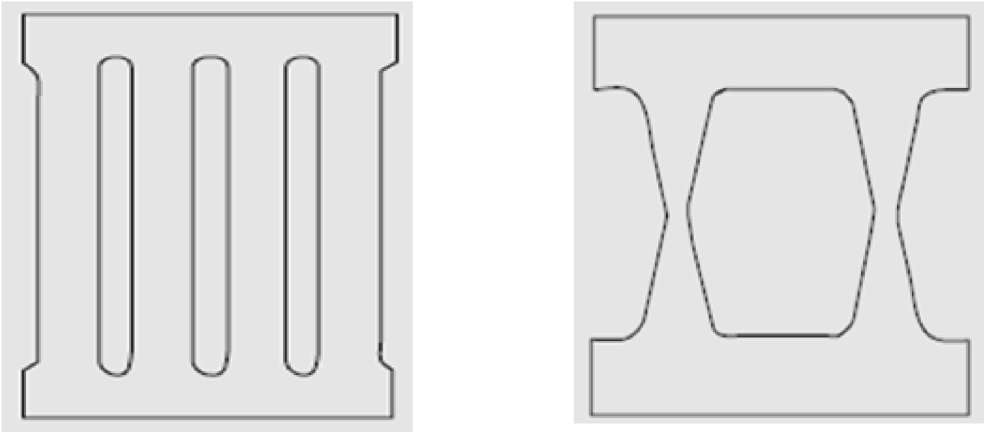


Figure 2.7 Hysteresis loops of metallic dampers (Orban, 2011)

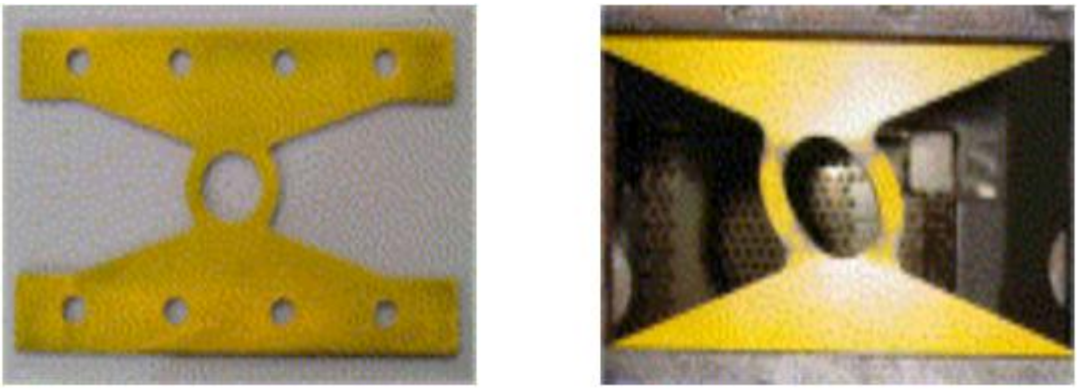
Metallic damper can be classified into two categories such as the shear type metal damper with the number of slots is subjected to shear deformations and dissipate energy through plastic shear yield metal plates, like the honey comb damper, slit damper, single round hole damper and double X- shaped metallic damper as illustrated in Figure 2.8 (a, b). The most commonly used of these types that are single round hole metallic damper and double X shaped metallic damper as shown in Figure

2.9 (a, b) and Figure 2.10 (a, b) respectively, also the other type is the bending type metallic damper uses flexural deformation of metallic plates under the out-of-plane bending, like the ADAS and TADAS metallic damper. Furthermore, the most recognized and used kinds of these dampers are the X-shaped and triangular metallic dampers (Li et al., 2014).



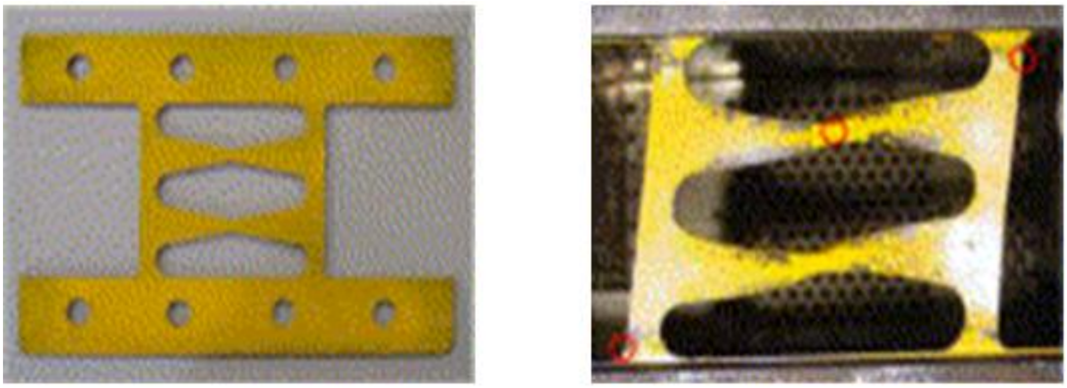
(a) Slit damper (b) Honeycomb damper

Figure 2.8 Type of hysteretic steel damper a) Slit damper and b) Honeycomb damper (Li et al., 2014)



(a) (b)

Figure 2.9 Single round-hole metallic dampers a) before deformation and b) after deformation (Li et al., 2014)



(a) (b)

Figure 2.10 Double X- shaped metallic dampers a) before deformation and b) after deformation (Li et al., 2014)

i. X-shaped plate dampers

These types of dampers used over the years. It consists of single or multiple constituents of thin metallic or layered plates of ‘X’ or ‘V’ shape. These types of dampers have the ability of maintaining many cycles of yielding deformations to reduce the input energy excitations (Bakre et al., 2006), and providing earthquake resistance for the structures. Also may be considered as a hysteretic device because its energy dissipation depends primarily on relative displacement within the device and not on its relative velocities (Pujari and Bakre, 2012). Kelly et al. (1972) and Skinner et al. (1975) were the first to consider the use of X-shaped plate dampers for dissipating input seismic excitations (Bakre et al., 2006).

The X-shaped metallic damper is not capable to give stable operational performance. Because it coming from the stress concentration at the corners of the steel plate. During the test of the hysteretic curves and deformation process indicated that the X-shaped damper is not efficient dissipating energy device, because it is the lack of sufficient deformation and energy dissipation capacity. In order to use this device economically, optimization of locations in a structure is an important issue and has been studied by several researchers (Pujari and Bakre, 2012). Another type of X-shaped dampers is called Added Damping and Stiffness (ADAS). ADAS devices have been used for dissipating seismic excitations through flexural bending deformation of steel plates of several buildings in Mexico City (Whittaker et al., 1989).

The characteristics of ADAS damper are a) easy to produce, easy to be use, b) and have stable performance under the effect of the seismic excitations, c) in addition to environmental conditions, and d) they can be replaced easily after the earthquake excitations, if needed (Rais et al., 2013). Damage to the building components will be avoided due to any plastic being inside the ADAS device during any event (Kareem et al., 1999). Additionally, as shown in Figure 2.11 (a, b); the device is composed of several parallel X-shaped types of steel plates designed to be installed in a structure frame in a way that the relative story drift causes the top of the device to move horizontally relative to the bottom. A large amount of energy can be dissipated by the ADAS device during an earthquake by yielding a large volume of steel. The properties of ADAS damper was studies by a few researchers named Whittaker et al. (1989); Su and Hanson (1990) at the University of California at Berkeley. To design safely ADAS devices the yield displacement should be above 10Δy (Δy is the yield displacement of devices). The device is formed in a way where the yielding happens over the total length of the device (Bagheri et al., 2011).

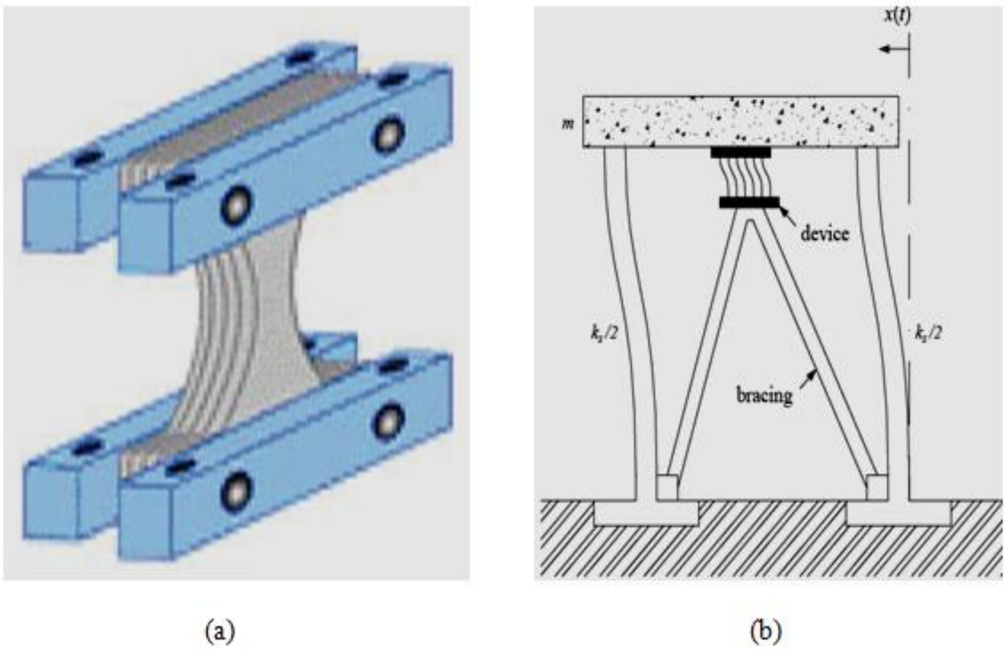


Figure 2.11 a) X-shaped ADAS device; and b) Typical building frame with ADAS element (Bagheri et al., 2011)

An experiment done by Whittaker (1989) on a three floor model structure using shake table test showed that the performance of the moment-resisting frame increasing its stiffness, increasing its strength, and finally increasing its capability to dissipate energy. Ratios of recorded inter story drifts in the structure with ADAS elements to inter story drifts in the moment-resisting frame were usually in the range of 0.3 to 0.7. Besides, an experiment has been done in Universities like California at Berkeley and Michigan which has shown that the effectiveness of the device is decreasing the seismic response to 3D structure samples. The number of the design of ADAS device in structures has gone down due to two reasons, which are building codes which many do not have yet a special provision for the design of structural systems with energy dissipation, and during the procedure of analyzing and designing the structure with ADAS device, it is not very known to every researcher but only to a small group. As well as calculated cost of the structure can also easily increase; this could be mentioned as on good reason for not using the ADAS device (Colunga, 1997).

Due to possess more stiffness and damping compared with other devices like FVDs, the ADAS dampers have the capability to increase the first floor stiffness by 30%. Also they increased the base shear by 20% in the comparative analysis of the retrofit of 10 storey office building in Almeda Park, Mexico city, therefore; in order for the ADAS dampers to be more cost effective, the necessity for additional stiffness must offset the cost of strengthening the structure itself (Hwang, 1998).

ii. Triangular shape plate dampers

Another example of metallic dampers that have received significant attention in recent years is triangular shape plate dampers. These types of dampers use plates of mild steel with triangular shaped plates so that yielding is spread almost uniformly throughout the material (Constantinou et al., 1998).

Triangular-plate added damping and stiffness (TADAS) devices are the derivation of ADAS elements developed by Tsai et al., in 1992/1993 as in Figure 2.12. In that system triangular plates are tighten to the plates by welding at one, while at the other end they are hinged by pins and slotted plate connection. Plates of this kind have been usually used for base isolation by Boardman et al. (1983) and Kelly et al.

(1972), before their application to building frames. Under cycling loading the TADAS damper has no stiffness or strengthens reduction due to the exhibit stable hysteretic performance. The behavior of TADAS dampers in braced frames is also verified through full scale tests (Tsai et al., 1993).

The triangular plate damper was first invented in New Zeeland. During earthquake, inter-story drifts cause to form a relative displacement between lower and upper part of TADAS dampers. This relative displacement results in yielding of steel plates and finally the energy of earthquake dissipates through this mechanism. Also if transverse force applied to the end of the triangular plate causes to form uniform curvature along the height of the plate. This behavior provides exploiting the total capacity of plates. Figure 2.13 shows an example of TADAS damper and its behavior during an earthquake. In the studies by Tsai and Hong (1992) and Tsai et al. (1993), it is shown a magnificent advantage in terms of predictably over conventional ADAS devices (Mohammadi and Garoosi, 2014).

Figure 2.12 Triangular plate dampers (Tsai et al., 1993)

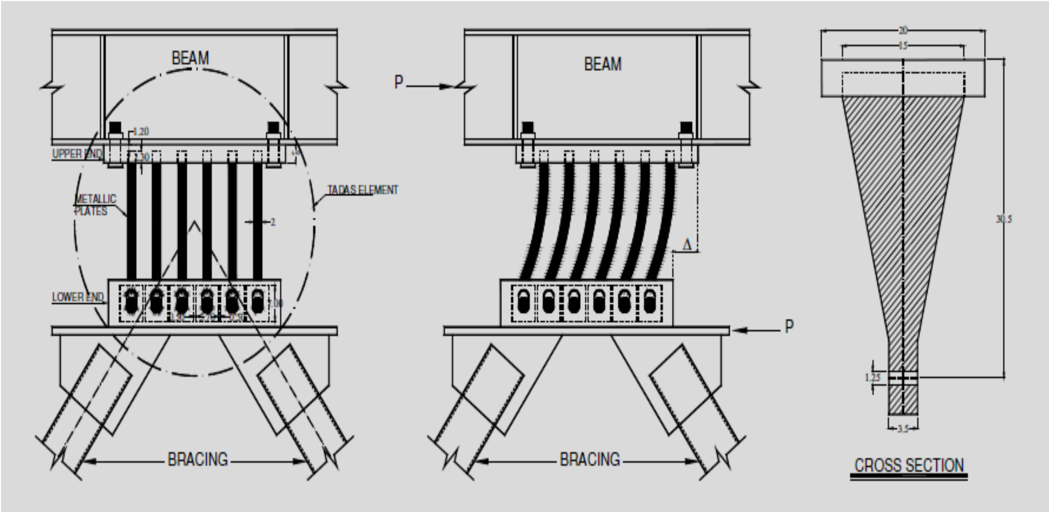


Figure 2.13 Behavior of TADAS damper during earthquake (all dimensions in centimeter) (Mohammadi and Garoosi, 2014)

iii. Application of metallic dampers

In San Francisco in the Wells Fargo Bank an ADAS adapter was installed which was giving protection to the structure against earthquake. In Tokyo (1990) in the Fujita corp., main office, the structure is 19 story steel frames; consist of 20 Lead Dampers in two directions. Likewise in Taisho Medicine Headquarter steel and reinforced concrete office structure of 38.75 m height that used Honeycomb Steel Damper. Also in Japan (1997) in Kobe Fashion Plaza (Kobe), the structure is steel frame of height 81.6 m, the Steel Dampers installed on 12th -18th Floors of the hotel (Kareem et al., 1999). ADAS devices were used in retrofitting Cardiology Hospital Building in Mexico City in 1990. The building consists of 6 storey reinforced concrete frame structure by nonlinear time history analysis using Drain-2D was used for the redesign. The results showed reduction in inter story drifts and base shear as well (Ibrahim, 2005).

iv. Methods for metallic damper applied by researchers

In order to apply optimization methods, the rate of rehabilitation along with preferred seismic performance of the structure should be reduced; also it is essential to achieve optimum allocation of control devices (Mohammadi and Garoosi, 2014).

For understanding the load deflection curve for the ADAS devices, Whittaker et al. (1989) found an easy way to define it. In Figure 2.14 (a, b) idealization of X-shaped equivalent plates are placed inside the actual profile of the ADAS. This method is gained the assumptions that the X-plates are rigidly restrained at their ends, that creates a double curvature, anti-symmetric by their mid height, and the width of the X-plates is half of their mid-height (b=1/2) (Colunga, 1997).

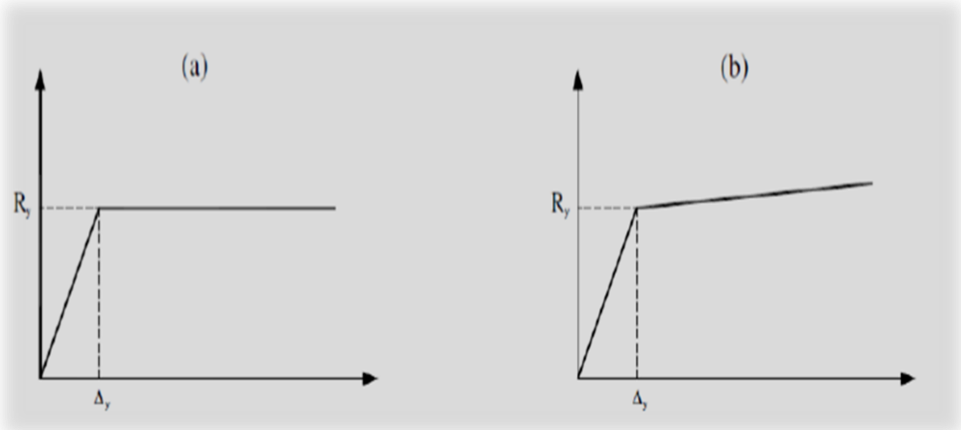


Figure 2.14 Idealized geometries for ADAS device a) Hourglass idealization, and b) Equivalent X-shaped (Colunga, 1997)

We can easily notice that the load-deformation curve of shear of ADAS can be market as an elastic plastic curve Figure 2.15 (a) or, as a bilinear one Figure 2.15 (b). The yielding point can be found by Whittaker et al. (1989) (Colunga, 1997).

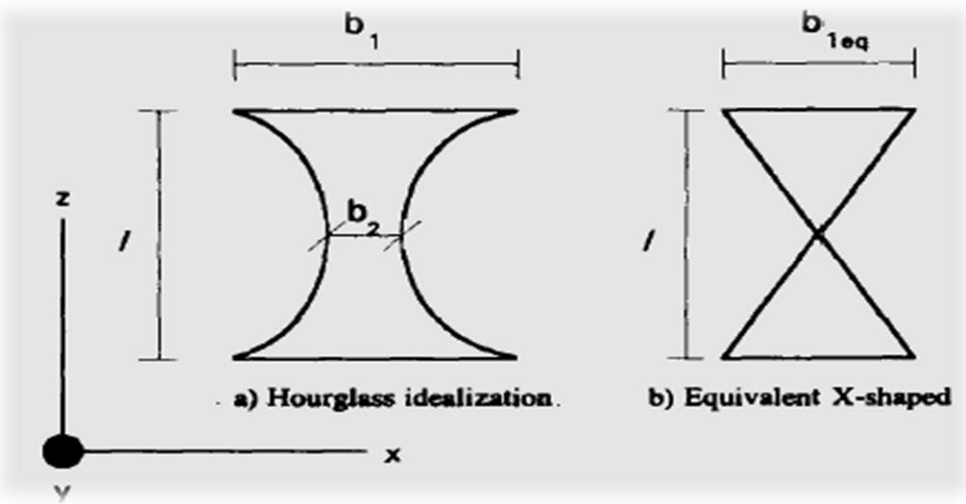


Figure 2.15 The commonly used load-deformation curve of shear of ADAS as a) elastic plastic curve, and b) bilinear curve (Colunga, 1997)

The ratio of the bracing member stiffness to the damper devise stiffness (B/D) of about two is used for the design of the ADAS device in all stories of the structures. Considering the cost of bracing members and yielding the ADAS devices without buckling or yielding of bracing members, these parameters has a significant result on the structural inelastic behavior (Xia and Hanson, 1992).

The study of an existing retrofit for a mid-rise steel building using additional stiff steel braced-frames against an alternate retrofit using ADAS (Added Damping and Stiffness) passive energy dissipation devices have comparative. The subject building located near Alameda Park in downtown Mexico City; is a ten storey office building that was built in the 1950s. The structure was damaged during the 1985 Michoacán earthquake because of resonant response with the site. The structure was later retrofitted using additional braced frames according to the seismic provisions of Mexico's 1987 Federal District Code. The retrofit scheme was planned to take the structure away from resonant responses and to inhibit structural damage. Carried out different sets of analyzes for compared alternatives: (a) 3D elastic analyses; (b) limit analyses and; (c) nonlinear dynamic analyses for putative ground movements for a *M*4"8.1 earthquake. Comparative studies have shown that the retrofit using ADAS device, have performance better than which uses steel bracing, but the initial cost of retrofitting with steel bracing is less than that of the ADAS retrofit and provides more strength (Colunga and Vergara, 1997).

The various parameters that can influence the device performance like the device yield force, the device yield displacement and the distribution of the devices have been study. Three ten storey moment frames with different ADAS elements were considered in the study. The response of these structure frames with the ADAS dampers was determined by using the program of DRAIN-2D and the following important conclusions were obtained a) the device yield force should be large enough to provide sufficient energy dissipation within the design ductility ratios, b) the device yield displacement should range between 0.0014 to 0.002 H, where H is the storey height, c) the ADAS devices yield forces can be distributed in proportion to the design shear forces while the stiffness of the ADAS elements can be distributed according to the story stiffness's of the frame without devices (Xia et al., 1990).

The performance of the ADAS dampers under simulated seismic excitations as well examined. Six ADAS dampers were attached to the three storey single bay ductile moment-resistant space frames. The first two natural vibration periods of the frame without the dampers are 0.74 and 0.22 seconds, respectively while the first two natural vibration periods of the frame structure with ADAS dampers and the chevron braces are 0.47 and 0.17 seconds, respectively. Three different simulated records were used including a soft soil synthetic (THSSR), 1985 Chile N10E and 1940 El Centro S00E. The results showed a significant improvement in structural response under different simulated seismic records (Whittaker et al., 1991).

Study the Parametric and experimental wok of earthquake behavior of structures with supplemental copper dampers and the numerical modeling of the stiffness and the load deformation curves based on the nonlinear behavior of the ADAS device (Liera, 2004; Colunga, 1997). The behavior and performance of steel structures equipped with ADAS and TADAS dampers and compared with traditional structures (Alehashem et al., 2008). Studying the influence of the different PGA’s and heights of structures on steel braced frame systems equipped with ADADS dampers and considering the idea of using separate metallic dampers for energy dissipation (Abdollahzadeh and Bayat, 2010; Kelly et al., 1972).

Optimum sizing of X-plate dampers for seismic response control of multistoried buildings has been investigated. In this study using square shaped building. The effect of damper parameters on the response quantities of the square building is studied for damper width and height in the practical range of 20–120 mm for four different thicknesses of the XPD in the range of 2–5 mm under real earthquake ground motions to obtain the optimum properties of the XPD. The role of the hysteretic energy dissipated by the XPD is also studied by using the percentage energy dissipation in the square shaped building as a criterion to decide the optimal range of combinations of the properties of the XPD and the following conclusions are drawn a) XPDs are very efficient in decreasing the earthquake response of square shaped building, b) the effectiveness of the XPD increases as the percentage energy dissipated by the XPD increases, c) there exists a range of sizes for which XPD gives maximum energy dissipated by XPD in square building, d) the energy dissipated in the square shaped building is dependent on the thickness of the XPD and input ground motion. As the thickness increases the amount of energy dissipated by XPD increases, and finally e) the percentage of energy dissipated by the XPD in square shaped buildings is higher for XPDs having lower values of *a* (half the height of XPD) and higher value of *b* (width of XPD) (Pujari and Bakre, 2012).

c) Buckling-restrained brace (BRB)

Buckling-restrained braced frame (BRBF) systems are currently used as primary lateral force resisting elements both in new construction and seismic retrofit projects (Benschoten et al., 2005). Buckling-restrained brace (BRB) are consists of a steel core, a confinement material and steel restraining tube. A typical BRB is sketched in Figure 2.16. The region between the tube and brace is filled with a concrete-like material and a special coating is applied to the brace to prevent it from bonding to the concrete. Thus, the brace can slide with respect to the concrete-filled tube. Bonding of the steel core to the concrete is prevented during manufacturing to ensure that the BRB components remain separate and composite action not allowed taking place (Robinson et al., 2013). The confinement provided by the concrete-filled tube allows the brace to be subjected to compressive loads without buckling (Symans et al., 2008). Based on the researches, the early beginnings of a brace restrained against buckling were in the 1970s in Japan (Marshall and Charney, 2008). The BRBF are designed using equivalent lateral force method (Deulkar et al., 2010). The use of BRBF systems has also been explored for bridge, blast, and lower seismic applications where the highly-ductile, non-buckling attributes of the BRB might still provide a significant benefit. BRBs have been utilized on several types of structures as part of a standard BRBF Frame. They are usage in building structures like office buildings, hospitals, retail, car parks, multi-story residential, schools, religious, stadiums and arenas, as well as non-building and industrial structures (Robinson et al., 2013). Recent experimental testing of BRBs demonstrated their ability to provide a stable, repeatable and ductile response (Marshall and Charney, 2008).

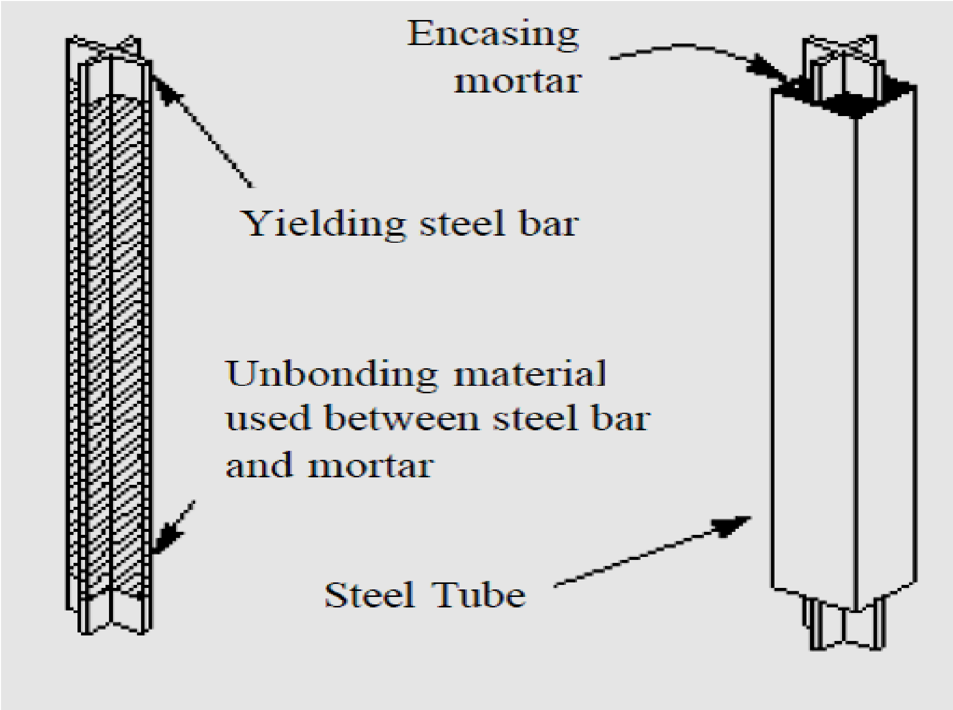


Figure 2.16 Atypical BRB (Robinson et al., 2013)

In most cases the core is dog-boned or has a tapered cross section so the yielding can be focused in the desired region. A variation of the typical BRB configuration called the confined yielding brace (CYB) replaces the mortar used to confine the steel core with a confined non-cohesive material. Because of this change, a debonding material is not required around the steel core. Additional modifications to the typical configuration include different geometric configurations of the steel core. Various core geometries are sketched in Figure 2.17 (a, b, c) (Marshall and Charney, 2008).

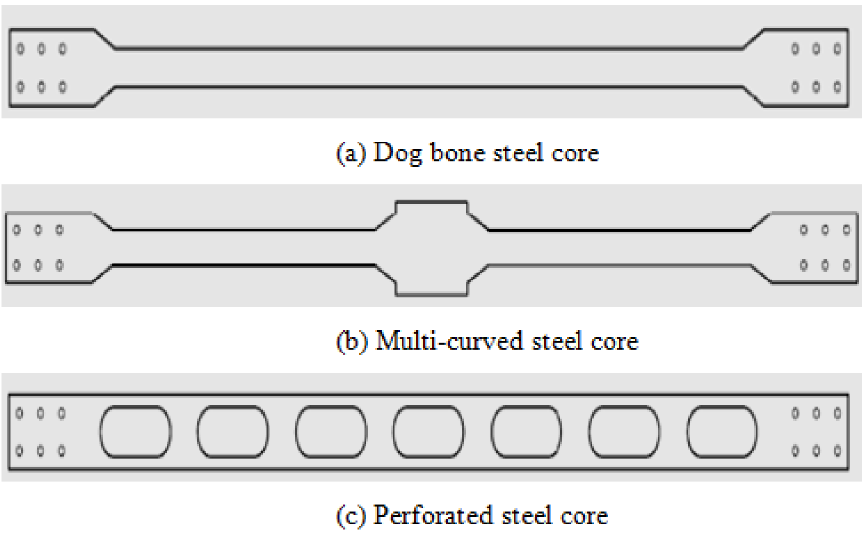


Figure 2.17 Steel core geometries for BRBs (Marshall and Charney, 2008)

The advantages of BRBs that have perfectly balanced hysteretic performance with compression yielding similar to tension yielding performance by achieving this during the decoupling of the stress resisting and flexural-buckling resisting sides of compression strength, the plastic hinges related with buckling do not take shape in properly designed and detailed BRBs, the BRBs permits very high compression strength, because there is no decrement in the existing material strength due to instability, the effective length of the core can be considered zero. By confining inelastic behavior to axial yielding of the steel core, the brace can achieve great ductility. Such analyses show that using braces with this type of hysteretic behavior leads to systems with very good performance. Drifts are expected to be significantly lower than the especially concentric braced frame (SCBF) due BRBs behavior (Deulkar et al., 2010).

d) Viscous damping

These device consist of a piston inside a pressurize cylinder with fluid medium like air, gas, water and oil that traveled between two internal chamber. As the building move by earthquake, the damper piston forces fluid from chamber to chamber converting the dynamic energy into heat energy which is absorbed and dissipated by the damper itself as shown in Figure 2.18, and Figure 2.19. The transverse energy to damped fluid minimizes the lateral movement and stress on structures which can lead to collapse. Chambers are configured in vary ways to accommodate different loads, building size and shapes (Zahrai and Mousavi, 2012). In viscous damping, the velocity of the vibrating body is relatively to the damping force (Zheng et al., 2002). The flexibility of viscous dampers can provide the ideal stiffness or repeal the unideal one by initial design (Constantinou et al., 1995). The original exertions have been tilting toward converting the applications developed for the armed forces and heavy industry to the civil engineering field (Mata et al., 2008).

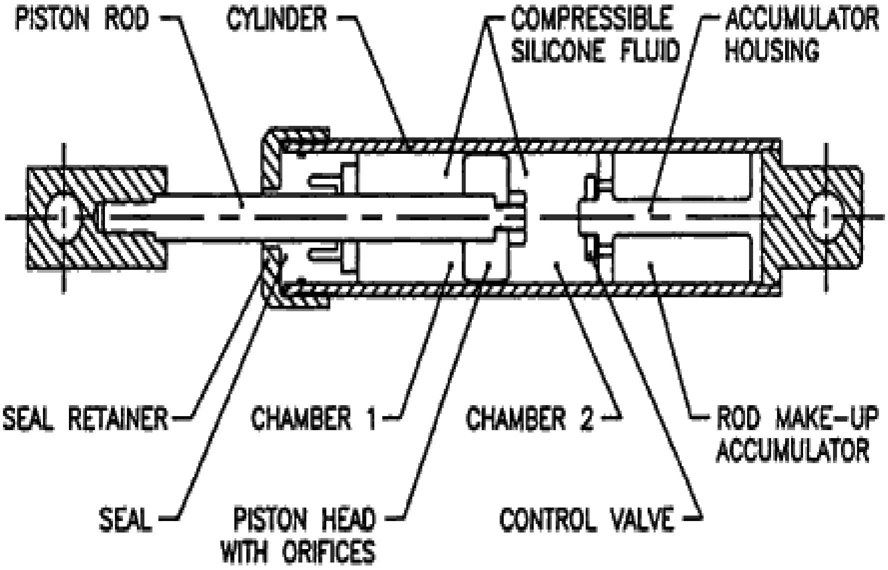


Figure 2.18 Schematic views illustrating an operation of viscous dampers (Zahrai and Mousavi, 2012)

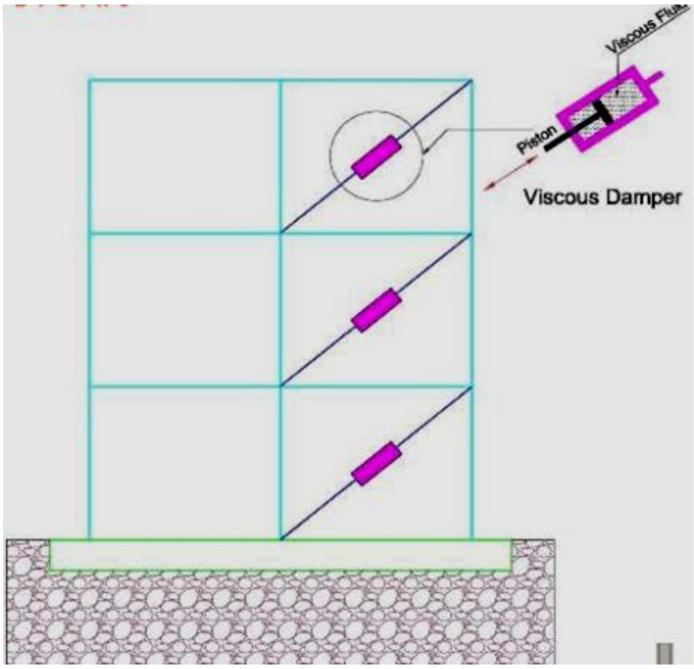


Figure 2.19 A typical building frame with viscous dampers elements (Zahrai and Mousavi, 2012)

e) Viscoelastic dampers

Viscoelastic dampers are one of the most important energy dissipation systems for wind and earthquake loading compared to other dampers which are used over 25 years for reducing wind-induced vibrations due to it is stiffness and damping (Constantinou et al., 1998).

Viscoelastic damper consider being velocity-dependant device. These devices contain viscoelastic material like acrylic copolymers, or glassy materials which are dissipated in the form of shear deformation under low seismic vibrations and they are depends on environmental temperature conditions, the strain level, and frequencies of vibration (Kareem et al., 1999). The reason for using polymers is that it is stiffness and damping characteristics commonly depending on the composition and operation (Mortezaei and Zahrai, 2009).

The viscoelastic dampers consist of pads of viscoelastic material that bonded to steel plates, such that relative displacement of steel plates will induce shear stress in the viscoelastic material as shown in Figure 2.20. The steel plates are attached to the structure within chevron or diagonal bracing where displacement is expected. The performance of viscoelastic damper in earthquake has been investigated during experimental studies (Milani, 2014). Researches pointed to that the damper properties remain comparatively constant with strains under 20% for the same temperature and frequency (Kareem et al., 1999).

Viscoelastic dampers has been applied in four buildings in the United States in the World Trade Center Towers in New York (1969) for reducing wind induced excitations, After that in 1982, VE dampers were included into the 76 storey in Columbia Sea. First structure in Seattle for reducing wind induced vibrations (Constantinou et al., 1998), also used in many structures in Japan: Seavans South Tower in Tokyo (1991), the Old Wooden Temple, Konohanaku Symbol Tower (1999), the Sogo Gymnasium in Chiba (1993), the Goushoku Hyogo Port Distribution Center (1998), and the Torishima Riverside Hill Symbol Tower. In addition, the Chientan Railroad Station in Taipei, Taiwan (Kareem et al., 1999).

A few researchers before investigated the alternative frequency and time domain approaches to evaluate the correlation coefficients for structures with viscoelastic dampers, the optimal parameter and location of VE dampers by simplex method, the mechanical properties for viscoelastic dampers and experimental test results of a five storey steel structures with viscoelastic dampers, and the experimental and analytical study on the seismic retrofit of 1/3-scale reinforced concrete structures by viscoelastic damper (Palmeri, 2006; Dong Xu et al., 2003; Won Min et al., 2004; Soong et al., 1995).

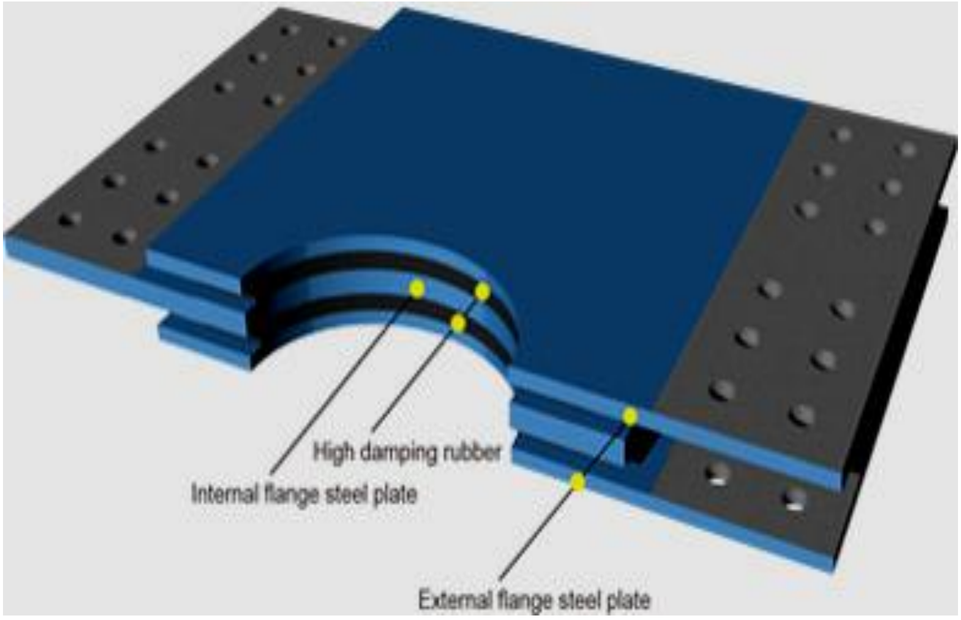


Figure 2.20 Viscoelastic damper (Milani, 2014)

2.2.1.2 Passive Dampers (with In-Direct Energy Dissipation)

In most cases, auxiliary damping could be supplied through the incorporation of some secondary system capable of passive energy dissipation. As a sample of this, the motion of building can be counteracted by attaching it to an extra mass and damper by a spring. Such passive system was embraced for their simplicity and ability to reduce the structural response. Among the passive devices that impart indirect damping through modification of the system characteristics, the most popular concept is the damped secondary inertial system, which will be discussed below. The damper is imparting indirectly to the structure by choosing the proper frequency response (Kareem et al., 1999).

a) Tuned Mass Dampers

Tuned mass dampers are a simplest form of passive energy devices that has been used over the years in many tall buildings and towers to dissipate input energy of the structures during the wind induced vibrations (Kareem et al., 1999; Avila and Goncalves, 2009). TMD was first suggested by Frahm (1909) to reduce undesirable vibrations in ships; also used to minimize fatigue effects (Tuan and Shang, 2014; Nawrotzki, 2008). Generally it consists of a single mass-spring-dashpot method linked elastically to the primary structure, usually near the top of building to maximize its effectiveness which determined by their dynamic features and the amount of added mass they use during transmitting the vibration energy to the structures for reducing its excitations, Figure 2.21 (Kareem et al., 1999; Avila and Goncalves, 2009).

The tuned mass damper has many advantages compared with other damping devices: accuracy, effectively for controlling the acceleration and displacement responses, and low cost maintenance (Nagarajaiah, 2009; Thockchom et al., 2014). The negative side of a TMD device is the sensitivity for the alteration in regulation the TMD frequency to the controlled frequency of a potentially unbecoming structure. Another negative point of a TMD is the size of the tuned absorber mass, but this disadvantages and negative limitations can be solved by conceptions of seismic isolation with TMD principles and conversion of a structural system into a TMD system through a specially designed (Chey et al., 2008). The additional damping introduced by the TMD is also dependent on the ratio of the damper mass to the effective mass of the building in a particular mode vibration. TMDs weigh 0.25%-1.0% of the building's weight in the fundamental mode (typically around one third) (Nawrotzki, 2008). The single tuned mass dampers are less efficient and weak than a multiple tuned mass dampers which consisting of several tuned mass dampers with a uniform frequencies for minimizing unwanted structure vibrations. The stiffness and damping of these types remain constant but the mass is varying (Li, 2000; Avila and Goncalves, 2009). The response of the primary system is not much influenced by small errors in the values of system parameters used in the TMD design (Avila and Goncalves, 2009).

TMD systems can generally be divided into two groups: vertically and horizontally working devices. Both vertically and horizontally direction TMDs are mainly arranged in structures like bridges or floors. They can be used to control the bending modes of the buildings. The arrangement is also achievable to limit torsional modes especially when they are arranged in pairs. The design of the tuned mass dampers depending on whether it is concrete or steel mass, the mass shapes, available space for the installation, target frequencies and damping as well as architectural restrictions in order to minimize this vibration (Nawrotzki, 2008). TMDs have been especially applied in many structures like CN Tower in Canada, the John Hancock Building in Boston, USA, Center-Point Tower in Sydney, Australia, and the once tallest building in the world named Taipei 101 Tower in Taipei (Tuan and Shang, 2014). There are many kinds of TMDs in use in Japan, usually utilize oil dampers (Kareem et al., 1999). Many authors studied and investigated the researching such as the performance of TMDs for response reduction of structures under near-field and far-field seismic excitation for 3, 9, and 20-story structure, the mitigating effects of a TMD on the structural dynamic responses of Taipei 101 Tower, under the action of winds and remote (long distance) seismic excitation, Giuseppe Carlo Marano investigated a comparison between different optimization criteria for tuned mass dampers design, presentation a method of estimating the parameters of tuned mass dampers for seismic applications, and presents the optimal design theories and applications of tuned mass dampers (Moghaddam et al., 2006; Tuan and Shang, 2014; Sadik et al., 1997; Lee et al., 2006).

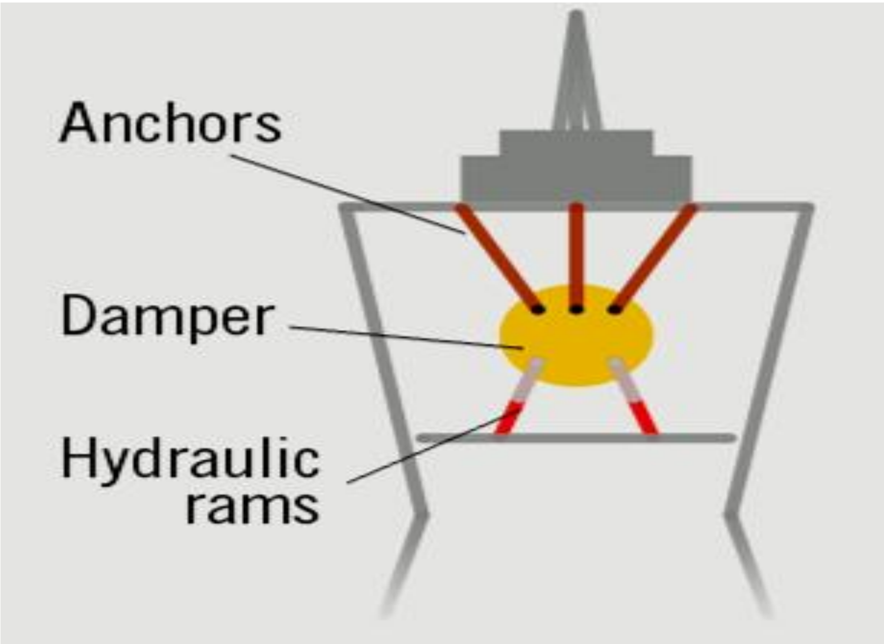


Figure 2.21 Tuned mass damper (Kareem et al., 1999; Avila and Goncalves, 2009)

b) Tuned Liquid Dampers (TLDs)

Tuned liquid dampers first used in space satellites and marine vessels (Limin, 1991). The effectiveness of TLD is increased by using multiple tuned mass dampers

(MTLDs) in which number of liquid sloshing tanks are increased to reduce the dynamic response of the structures. These MTLDs can be used for tall buildings to minimize the wind and earthquake excitations (Umachagi et al., 2013). Authors studied and investigated the researching such as a new approach facilitates the mitigation of wind caused by the movement of buildings using tuned liquid dampers , and experimentally the tuned liquid dampers behavior with a slopped bottom of an angle 30^{0} with the horizontal (Kareem, 1990; Gardarsson et al., 2001). TLD are applied in Nagasaki Airport tower, Yokohoma Marine Tower, Shin Yokohoma Prince Hotel and Tokyo international airport tower. Tuned liquid dampers (TLDs) are classified into two main categories such tuned sloshing dampers (TSD) and tuned liquid column dampers (TLCDs) (Kareem et al., 1999).

i. Tuned sloshing dampers (TSD)

Tuned Sloshing Dampers are generally come in the rectangular or circular shape. Their main function is to control the vibration by installing them in the top most floor of the building (Kareem, 1990; Nanda, 2010). Depending on the water wave theory, TSD can classify to shallow water type or deep water type according to the water height in the container, If h (water height)*L or D (length or diameter of water tank in the direction of excitation) >0.15 the TSD said to be deep water type, and if it's <0.15, it said to be shallow water type. The scheme of TSD is shown in the Figure 2.22. The under controlled building frequencies will determine whether depth of liquid in a container is deep or shallow (Venkateswara, 2013).

As a passive energy dissipation device TLD presents several advantages over other damping systems such as (i) Low installation and RMO (Running, Maintenance and Operation) cost, (ii) Fewer mechanical problem as no moving part is present, (iii) Easy to install in new and existing structures as it does not depends on installed place and location, (iv) It can be applied to control a different vibration type of multi-degree of freedom system which has a different frequency for each other (v) Applicable to temporary use (vi) Non restriction to unidirectional vibration (vii) Adjusting the depth of liquid also container dimensions for controlling the natural frequency of TLD, and (vii) Water present in the damper can be used for firefighting purpose (Venkateswara, 2013).

Along with the above mentioned advantages, there are some drawbacks too associated with TLD system. The main drawback of a TLD system that, not all the water mass involve in counteracting the structural motion. This leads to the addition of more weight without getting the any benefit. Again low density of water makes the damper bulky, and hence increases the space required housing it. As is the case for Tuned Mass Damper, there exists an optimal damping factor for TLDs. Since usually plain water is used as the working fluid, it gives a lower damping ratio compared to the optimal value. In order to overcome the drawbacks and to achieve the optimal damping ratio, several methods are proposed such as (i) Installing the TLD at proper position, (ii) Wave breaking in shallow water TLDs, (iii) Addition of floating beads as surface contaminants, (iv) Using submerged nets and screens , (v) Using Slopped bottoms for TLD, (vi) Enhancement of bottom roughness by using wedge shaped bottom with steps and with holes, (vii) Using a conical TLD, and (viii) Inserting poles (Venkateswara, 2013).

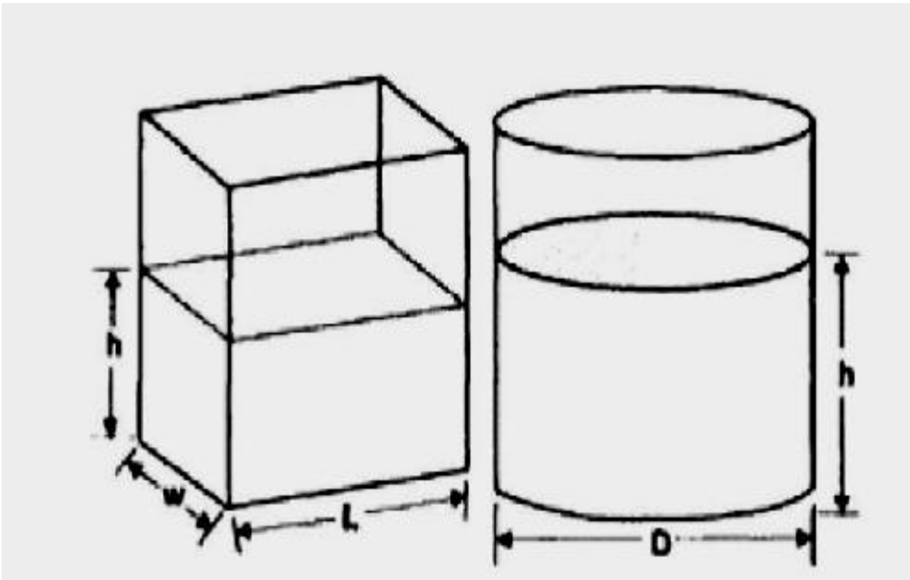


Figure 2.22 Tuned sloshing damper (Venkateswara, 2013)

ii. Tuned Liquid Column Dampers

Tuned liquid column dampers (TLCD) combine the effect of liquid motion in a tube, which results in a restoring force using the gravity effect of the liquid, and the damping effect caused by the hydraulic pressure loss because of the orifices that installed inside the container, Figure 2.23 shows the schematic of a TLCD. When two LTDC dampers are moving in different direction, then a so called Double Tuned Liquid Column Damper (DTLCD) is formed. The motion into two different directions increases its effectiveness eliminating the limitation of regular unidirectional TLDC. The advantages of TLCD are any shape can be taken in order to fit it into the structure, its behavior is well studied and known, adjusting the orifice opening the TLCD damper can be under control, adjusting the liquid column in the tube, and its frequency can be controlled as well (Venkateswara, 2013).

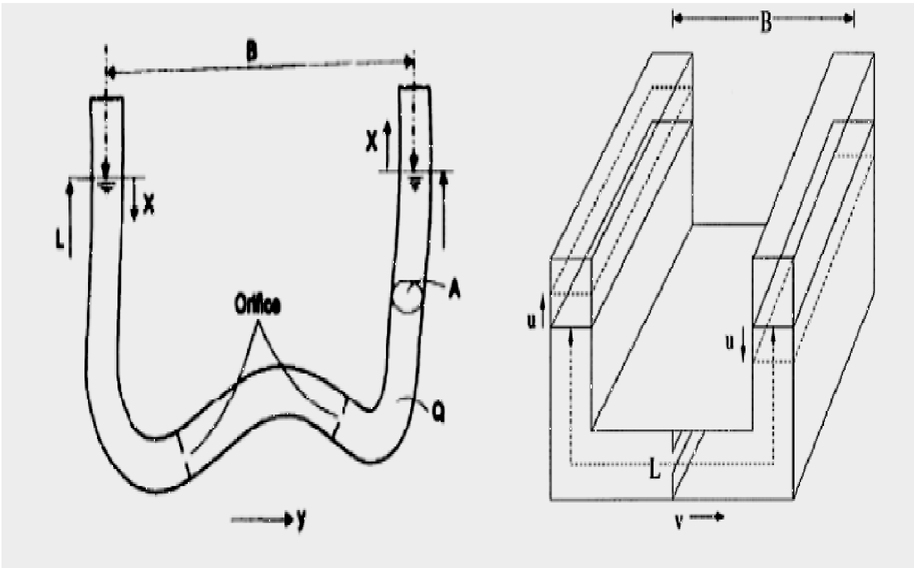


Figure 2.23 Tuned liquid column damper (Venkateswara, 2013)

CHAPTER 3

METHODOLOGY

3.1 Analytical Model of Structures

The reinforced concrete (RC) frame models used in this study were 6-storey, 9-storey, and 12-storey simple frames with ADAS devices and inverted V- bracings that utilized for reducing the adverse effect of the earthquake ground motions. The model structures with regular in shape and symmetrical in plan of 6 m span, 3 m story height, and slab thickness of 12 cm were used. The plans were the same at all stories and loading of all frames were also similar. The fixed foundations for the columns at basement level were considered for all cases. These frames were designed as three bays on each direction, and they worked as a rigid diaphragm that connected the frames at each floor level and only two dimensional analysis was created for each structure in order to make the explanation of the results of the analysis easier. Typical floor plan of the case study RC models for 6, 9, and 12 storey structures are shown in Figure 3.1**.**

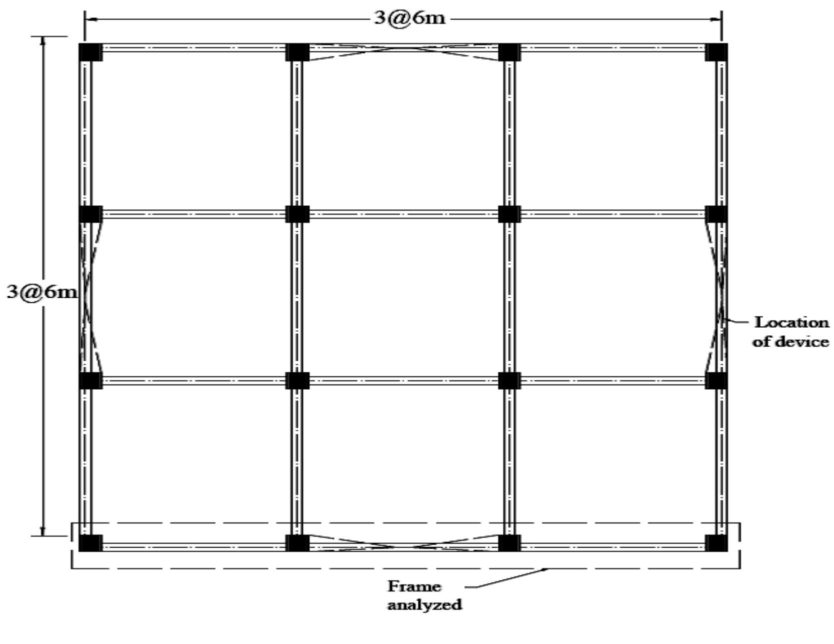


Figure 3.1 Typical floor plan of the case study RC building models for 6, 9, and 12 storey structures

The external and internal frames of the structures consisted of three bays; only one bay (middle bay) consists of braces with ADAS dampers. The dimensions of the beams were 60 cm in height and 30 cm in width for all frame cases while the dimensions of the columns varied for each storey and for all frames as shown in Tables 3.1 and 3.2. Long aspects of the columns were placed in the external axes while in the internal axes, short aspects of columns were located in the direction parallel to the x axis. The live load and additional dead load uniformly distributed on the slabs were considered as 2 kN/m^2 and 1.5 kN/m^2, respectively.

Table 3.1 Dimensions of the columns for the 6-storey structure

Storey level	1	2	3	4	5	6
Dimensions of the columns (cm)	55x55	55x55	55x55	50x50	50x50	50x50

Table 3.2 Dimensions of the columns for the 9 and 12-storey structures

Storey level	1	2	3	4	5	6-12
Dimensions of the columns (cm)	60x60	60x60	60x60	55x55	55x55	50x50

RC structures (6, 9, and 12 storey frame structures) were designed with inverted V-shaped braces and ADAS dampers in order to compare their seismic response with each case. The braces were connected to the bottom flange of the beam of the RC structure, and the ADAS dampers were fixed between beams and braces and inserted in an interior bay (with pin connections) of the structure for all structure cases. For the inverted V-shaped braces element, tube sections were used by the following dimensions: outside depth, outside width, flange thickness, and web thickness of 6 cm, 6 cm, 1 cm, and 1 cm were used, respectively for all frame cases. Figure 3.2 reveals the typical building frame with ADAS element. It was consider that the bracing elements remain elastic and behave linearly unlike ADAS element. A bilinear elastoplastic shear spring with kinematic hardening and Wen plasticity was used to model the nonlinear behavior of ADAS elements (Bagheri et al., 2011).

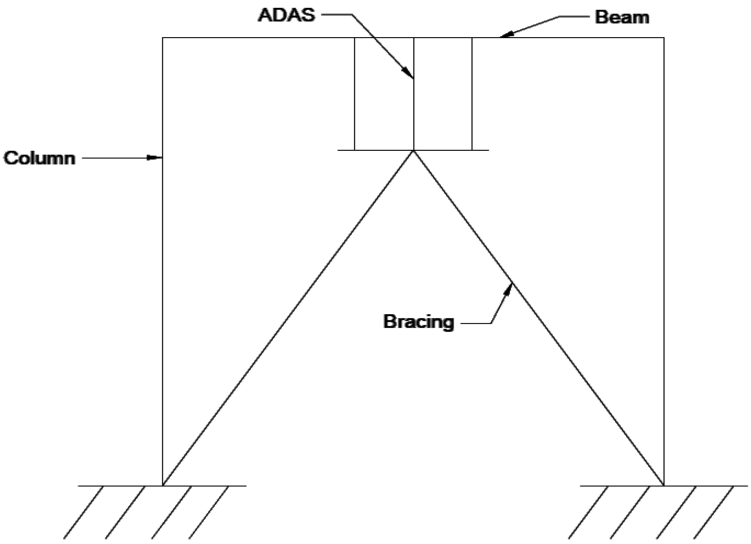


Figure 3.2 Typical building frame with ADAS device

The modulus of elasticity of concrete was 25 GPa and that for steel was 200 GPa. A concrete compressive strength of f′c = 28 MPa and steel yield strength of f_y = 350 MPa were considered. Figures 3.3, 3.4, and 3.5 show the frames for the 6 storey, 9 storeys, and 12 storeys, respectively. Buildings were supposed to have a uniform mass distribution over the height and non-uniform distribution of the lateral stiffness.

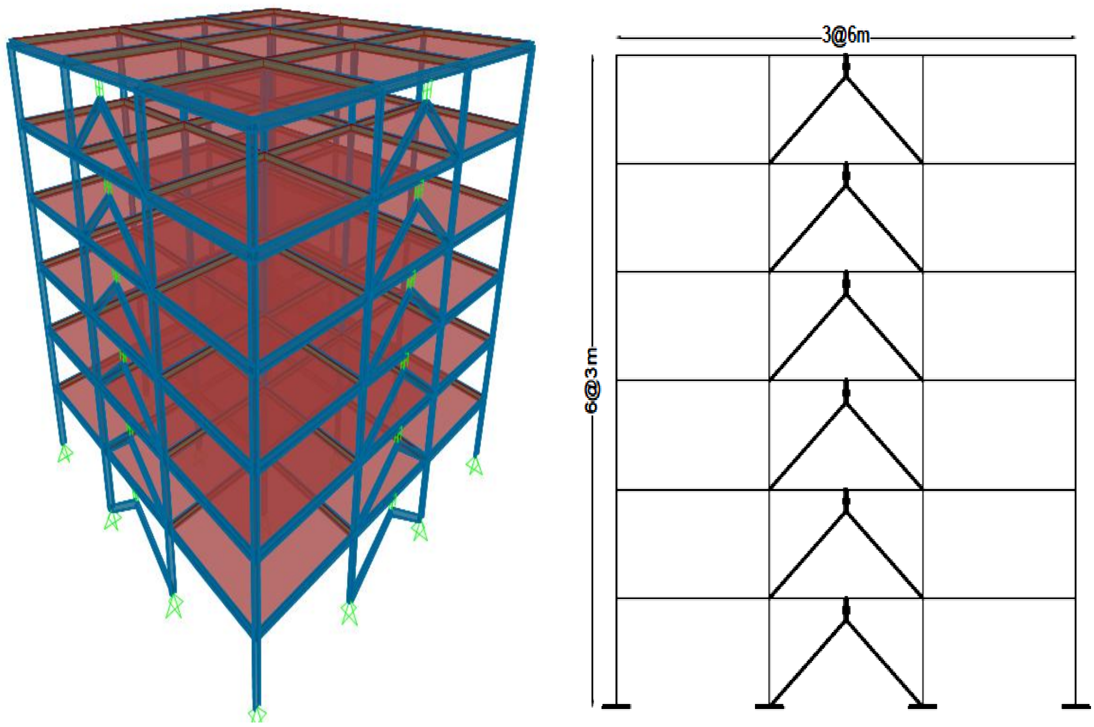


Figure 3.3 Six storey frame structure with ADAS dampers and inverted V- braces

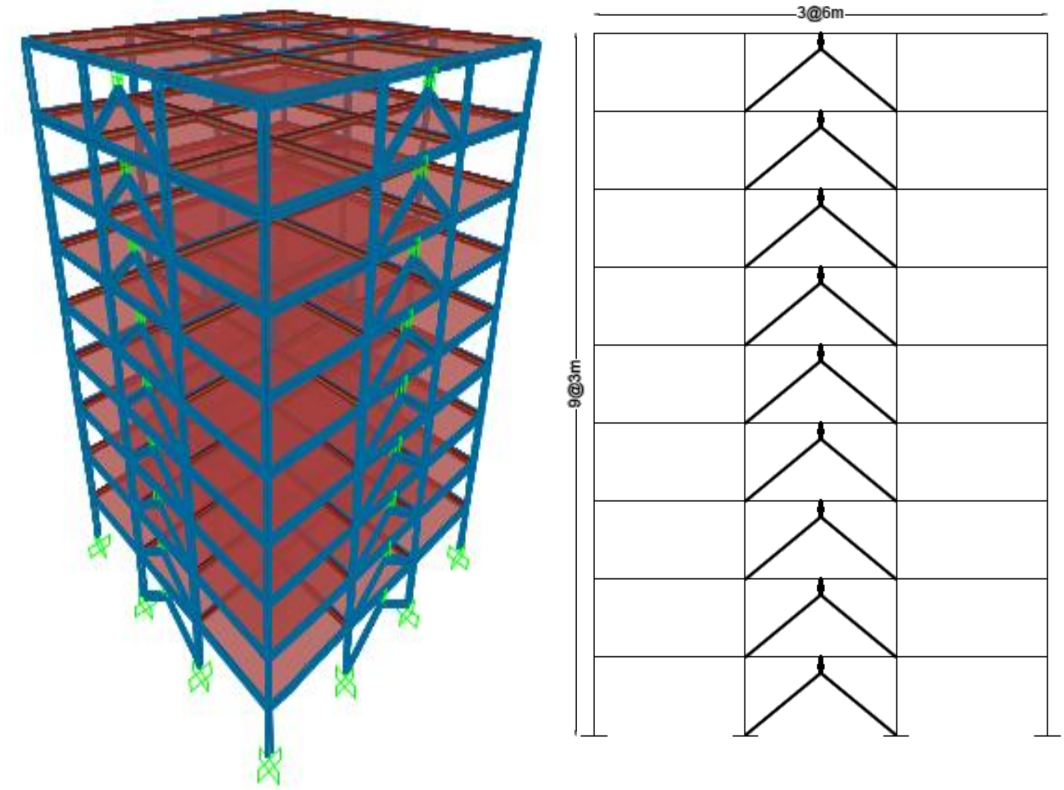


Figure 3.4 Nine storey frame structure with ADAS dampers and inverted V- braces

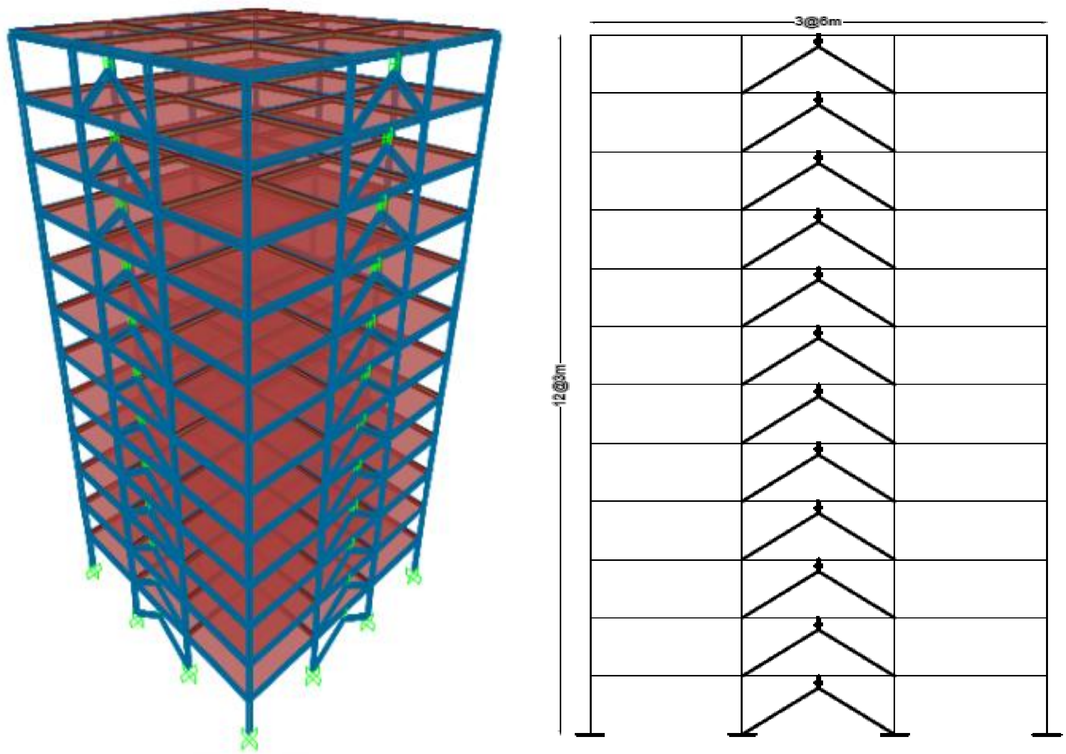
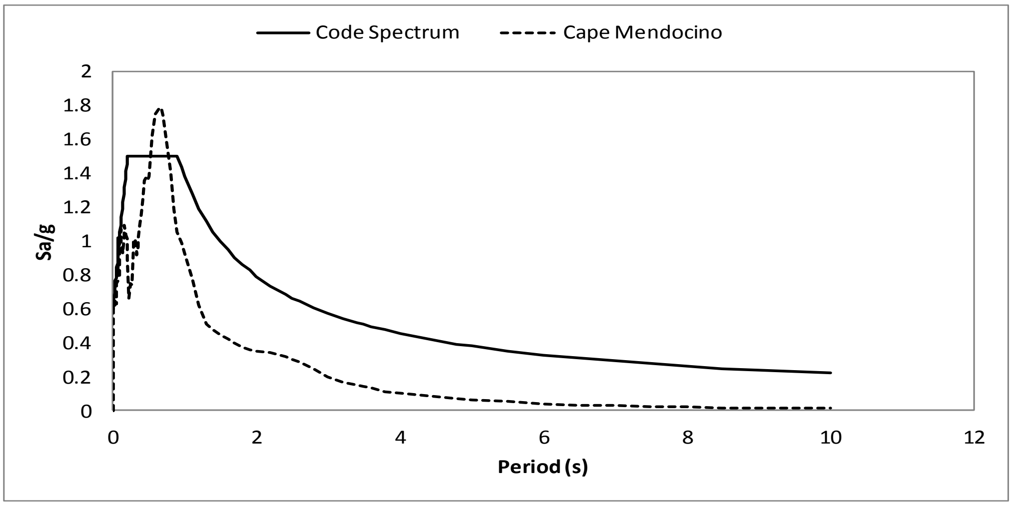


Figure 3.5 Twelve storey frame structure with ADAS dampers and inverted V-braces

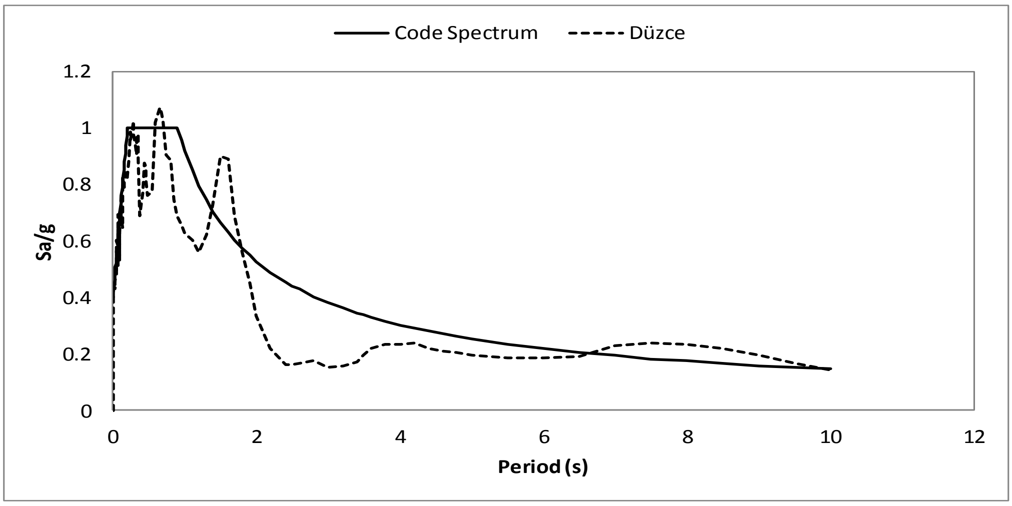
3.2 Nonlinear Analysis

Nonlinear static and dynamic analyses were performed by using the structural analysis program of SAP 2000 version 14 (CSI, 2009) to evaluate the performance of the frame structures equipped with inverted V-shaped braces and ADAS dampers under different ground motions. Nonlinear analysis method might be considered as easier for a large scale problem when the seismic force and deformation demands could be evaluated for the performance of the structures during the earthquake. In this regard, nonlinear static pushover analysis could be a more fairly accurate method of analysis which recommended by Eurocode and FEMA through which the structure of the building was subjected to incremental lateral forces with distribution in rise until it reached the desired displacement. It covered a sequential series analysis of cracks, yielding, the plastic hinge formation, and the failure of the various structural elements. The earthquake-induced forces almost signified the equivalent lateral loads. In the current study, the displacement-base shear relation was plotted to show the pushover response of the structure. According to FEMA 356 (2000), the hinge properties of structural elements were identified with taking the type and mechanism of failure until reaching the target displacement by increasing the lateral forces. The capacity curves that associated with roof displacement versus base shear for 6, 9 and 12 storey frames with inverted V-shaped braces and ADAS dampers were attained at the end of the nonlinear static analysis.

The analysis results obtained from the nonlinear static method were evaluated by means of the dynamic time-history analysis in which the real ground motion records were used for accessing the response variability of the structures under loading according to the function of time. The nonlinear dynamic time history analysis was realized by following the direct integration methods. The seismic response of the structures with inverted V- braces and ADAS dampers were verified under various acceleration earthquakes. For the nonlinear dynamic analysis of the frames, a group of the natural earthquake acceleration was used (PEER, 2011). The design code spectrum and elastic spectral acceleration of the ground motion at 2%, 10%, and 50% probability of exceedance in 50 years are given in Figure 3.6.



a)



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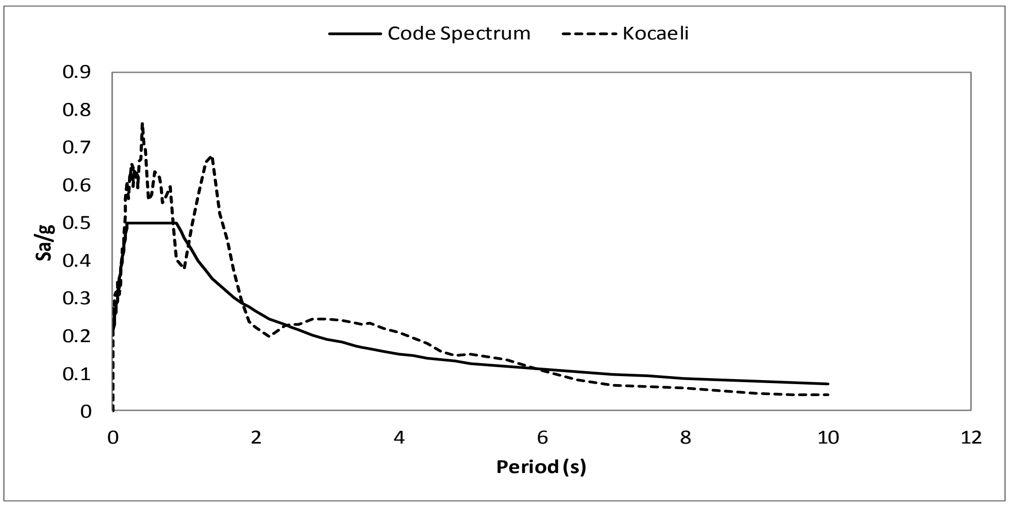


c)

Figure 3.6 Elastic spectral acceleration of the ground motion: a) 2%, b) 10%, and c) 50% probability of exceedance in 50 years

Furthermore, the properties of the natural ground motions such as the magnitude (Mw), the peak ground acceleration (PGA), the peak ground velocity (PGV), the peak ground displacement (PGD), and the characteristics of the site are given in Table 3.3.

Table 3.3 Characteristics of the ground accelerations used in the analysis

Earthquake Record	Year	Magnitude (M_w)	Mechanism	Vs30 (m/s)	PGA (g)	PGV (cm/s)	PGD (cm)
Düzce	1999	7.14	Strike-Slip	517.0	0.43	106.2	142.0
Cape Mendocino	1992	7.01	Reverse	712.8	0.62	81.9	25.5
Kocaeli	1999	7.51	Strike-Slip	297.0	0.28	48.2	43.0

As previously mentioned the plastic hinge properties were executed using SAP 2000 as illustrated in FEMA (356). Figure 3.7 shows a force-deformation relationship for a typical plastic hinge. The assigned values to each of these points showing in the figure varied depending mainly on the type of element, material properties, and axial load level on the element. Labeled IO, LS, and CP shown in the figure indicated for Immediate Occupancy, Life Safety, and Collapse Prevention, respectively.

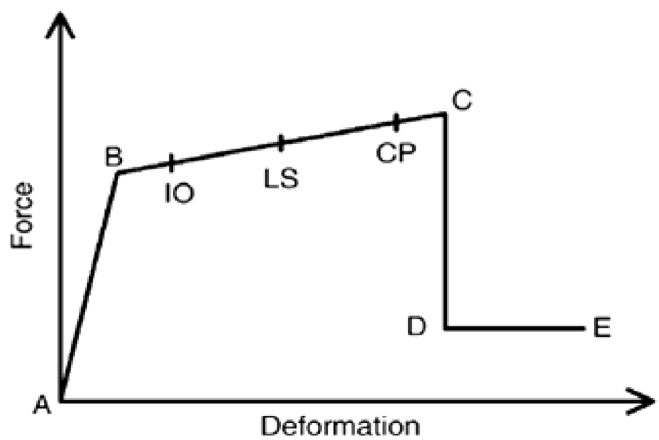


Figure 3.7 A typical force-deformation relationship for a plastic hinge (FEMA 356, 2000)

In both of the static and dynamic analysis cases, the post yield was simulated to all the structural elements by concentrated plastic hinges. Elastic behavior happened along a member length, and subsequently deformation within hinges happened, which was modeled in separate locations. Inelastic behavior was achieved by integration of plastic strain and plastic curvature, which lied within the boundaries of the hinge length, usually on the order of element depth (FEMA 356, 2000). To capture plasticity distribution along the member length a series of hinges were modeled. Multiple hinges were also coinciding at the same location. Plasticity were associated with force-displacement behaviors (axial and shear) or moment-rotation (torsion and bending). The nonlinearity was taken into account by adopting plastic hinges with hysteretic relationships based on FEMA 356 (2000) at each end of the beam and column members.

In this study, only two types of hinges are used to simulate the plastic hinge formation through the nonlinear behavior of the structure. The first is the coupled axial and moment hinge (P-M2- M3) which was assigned to the column elements. The hinge properties of this type were created based on the interaction surface that represented where yielding first occurred for different combinations of axial force, minor moment, and major moment acting on the section. The second type was the moment hinge (M3) which was assigned to the beam elements. Table 3.4 shows the fundamental period of different RC frames with inverted V braces and ADAS dampers.

Table 3.4 Fundamental periods of the frames with inverted V- braces and ADAS dampers

Type of frame	T1 (s)	T2 (s)	T3 (s)
6 storey frame	0.35	0.11	0.06
9 storey frame	0.52	0.18	0.10
12 storey frame	0.72	0.24	0.14

At the six storey buildings, free vibration periods for the frame with inverted V-braces and ADAS dampers were determined. The first three modes of free vibration period obtained as T_1= 0.35 s, T_2=0.11 s, and T_3=0.06 s. However, the free vibration periods for the 9 storey frame was obtained as T_1=0.52 s, T_2=0.18 s, and T_3=0.1 s; while for the 12 storey frame, the first three modes of free vibration period was obtained as T_1=0.72 s, T_2=0.24 s, and T_3=0.14 s.

CHAPTER 4

RESULTS AND DISCUSSION

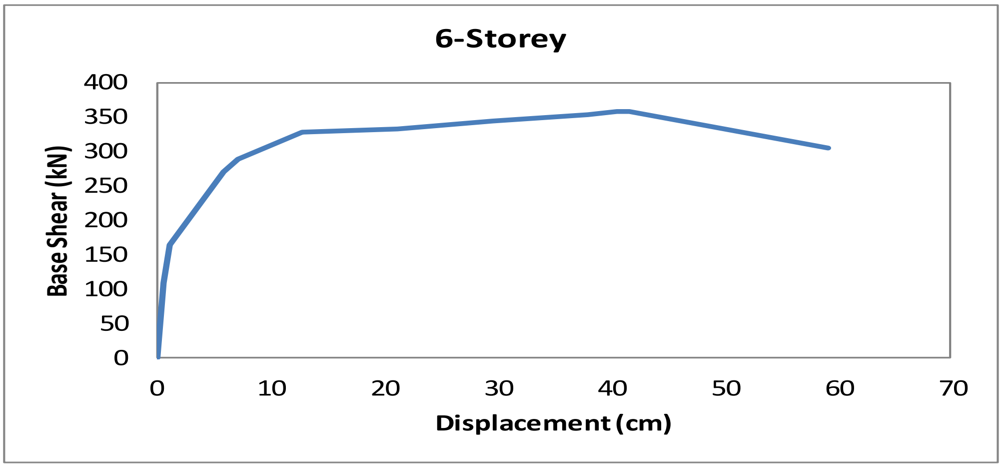
4.1 General

In this part, the results for the structures with inverted V-shaped braces and ADAS dampers based on the nonlinear static and time history analyses were given and discussed comparatively. To this aim, three different frame structures having various numbers of storeys were subjected to the earthquakes of 2%, 10%, and 50% probability of exceedance in 50 years. The performance characteristics in terms of capacity curves, variation of storey displacement, roof drift, inter storey drift ratio, hysteretic curve, and plastification were evaluated and provided below.

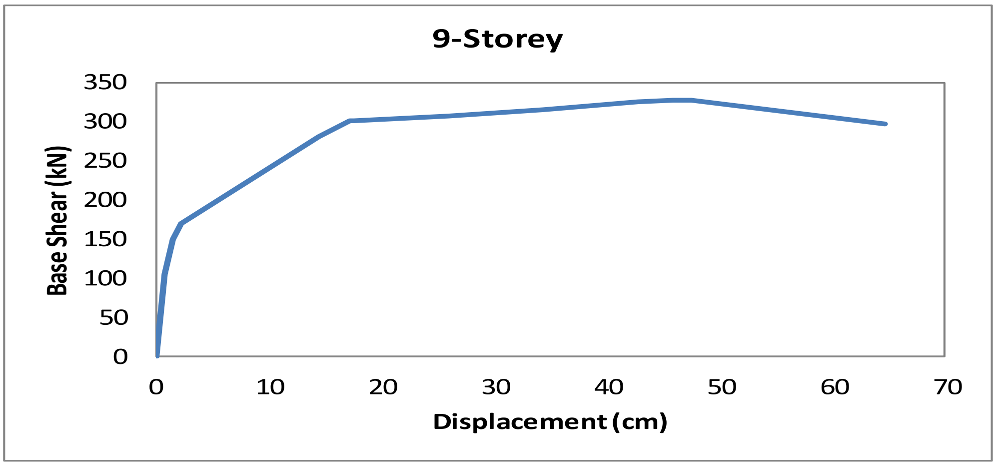
4.1.1 Capacity Curves

The capacity curve is the envelope response curve which yields the optimal response and gets the drift limits and the maximum base shear. It can be approximated with bilinear scheme for the typical RC structure. The capacity curves that resulting from the analysis of frames with ADAS dampers and inverted V- braces for all three frame cases of 6, 9, and 12 storeys were evaluated and illustrated in Figure 4.1 (a, b, and c).

The figures showed that the frame with inverted V-shaped braces and ADAS dampers had different lateral load carrying capacities. For instance, the maximum base shear of the 6 storey frame was obtained as 357 kN while for 9 storey frame the maximum base shear was about 328 kN. In the case of the 12 storeys frame, it was about 291 kN. The comparison of three frame cases with inverted-V braces and ADAS dampers indicated that the lateral load carrying capacity between frames 6 and 9, also between frames 6 and 12 implied about 1.23 and 1.09 times difference, respectively. Moreover, the structural systems with inverted V braces and ADAS dampers dissipated a large part of the input seismic excitations, and particularly the shear forces of the storeys were considerably decreased. It was also observed that the frame cases of 9 and 12 storeys had similar initial stiffnesses due to adding steel braces and ADAS damper systems.



a)



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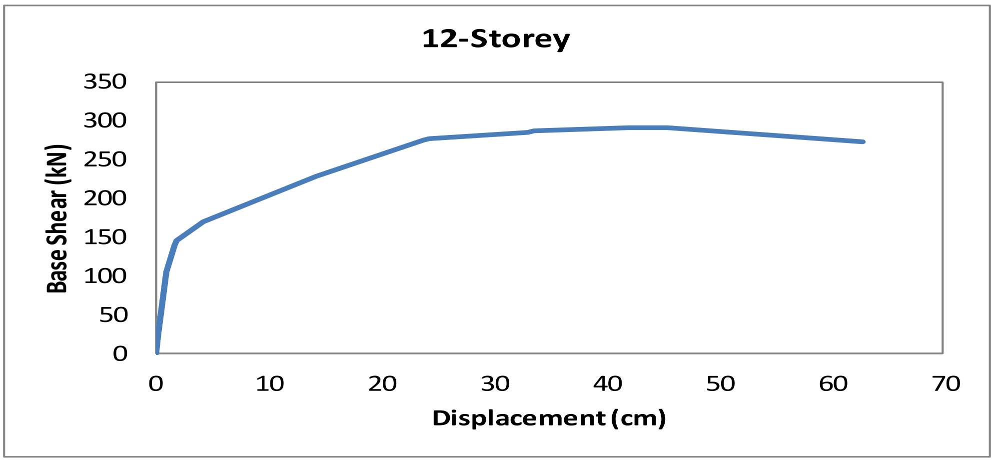


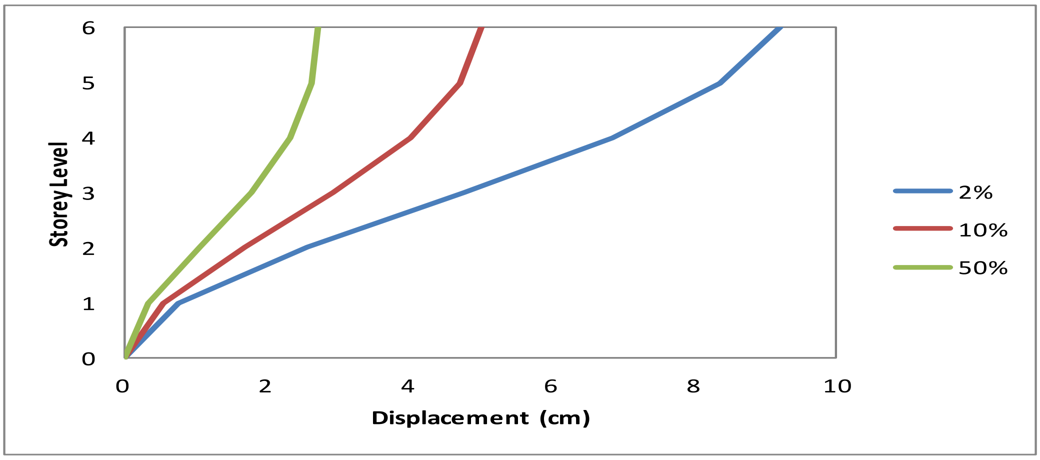
c)

Figure 4.1 Capacity curves of a) 6 storey, b) 9 storey, and c) 12 storey frame structures

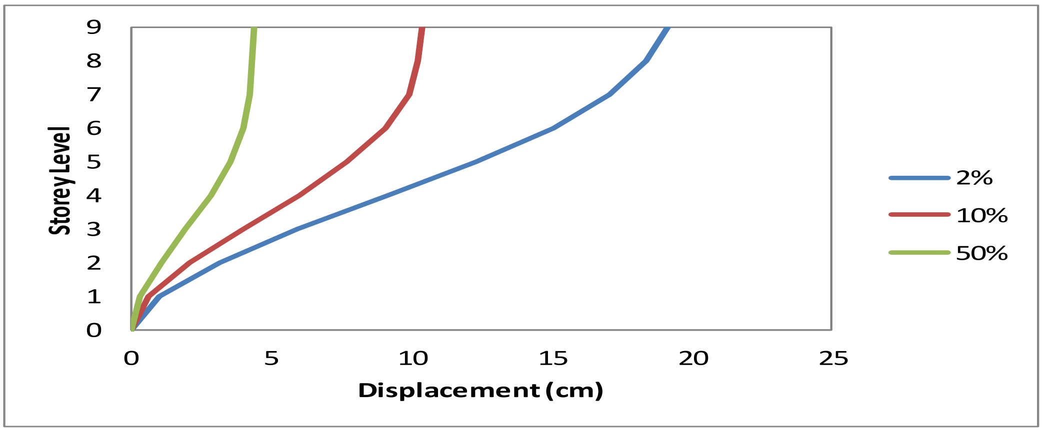
It was shown that the capacity curves were almost trilinear in all the conditions of the structures with inverted V- braces and ADAS damper, because in the beginning, the structure was in a elastic stage and supplied a linear elasticity slope and some structure members like beams and columns cracked, and some of them yielded like bracing; this happened when the shear base began to increase and then led to a change in the slope of the capacity curve. With the use of inverted V – braces and ADAS dampers, the structure had the capability of carrying more lateral load and also more energy absorption capacity.

4.1.2 Variation of Storey Displacement

Figure 4.2 illustrates the variation of storey displacement for the 6, 9, and 12 storey RC frames and each of frames subjected to the earthquakes having 2%, 10%, and 50% probability of exceedance in 50 years. It was evident from the curves that all the studied frames were significantly affected by the given seismic records. Furthermore, the inverted V-shaped braces and ADAS dampers that used for the purpose of upgrading structural response of the frame significantly reduced the maximum storey displacement value which influenced by frame type and number of storeys. For example, in the case of 6 storey frame having 50% earthquake probability, the maximum storey displacement was smaller than other frames; by increasing number of storeys and decreasing the probability ratio of exceedance in 50 years, the maximum storey displacements were also increased according to the results that obtained from the nonlinear time history analysis.



a)



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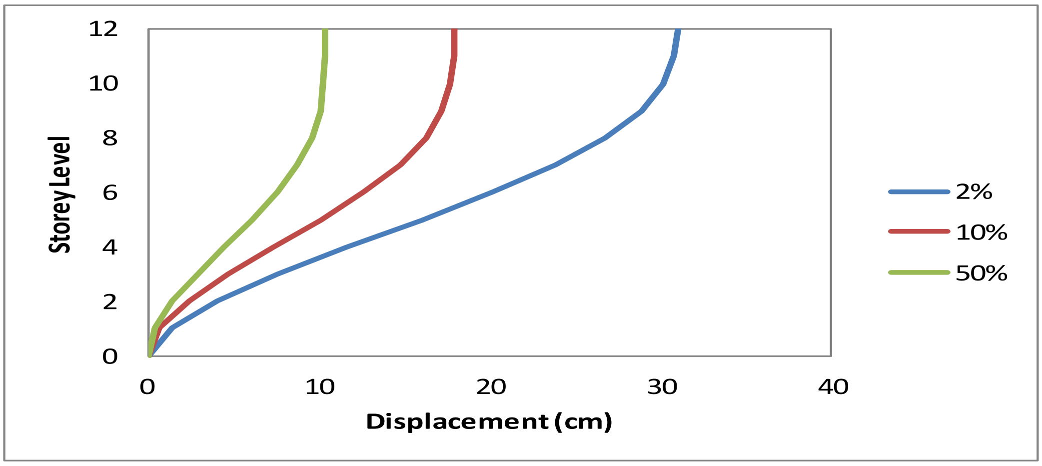


c)

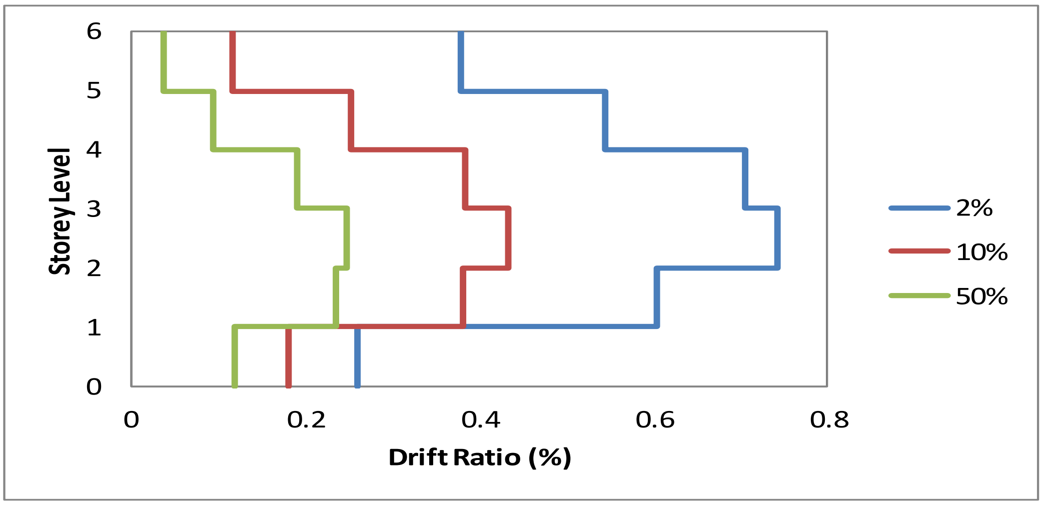
Figure 4.2 Variation of storey displacement of a) 6 storey, b) 9 storey, and c) 12 storey buildings under earthquakes having 2%, 10%, and 50% probability of exceedance in 50 years

In the case of the 6 storey building under the earthquake with 2% probability (which represents Cape Mendocino earthquake), the obtained maximum displacement of the frame structure was 9.2 cm, while the maximum displacement for the structure under the earthquake with 10% probability (which represents Düzce earthquake) was obtained as 5.02 cm, and also the structure under the earthquake with 50% probability (which is Kocaeli earthquake) had maximum displacement of 2.73 cm as shown in Figure 4.2 (a). For the 9 storey frame structure under the earthquake with 2% probability, it was found that the maximum displacement was about 19.15 cm, while under the earthquake with 10% probability, it was obtained as 10.34 cm, and finally under the earthquake with 50% probability, it was obtained as 4.39 cm as illustrated in Figure 4.2 (b). Like two frame buildings discussed above, the maximum storey displacement for the case of 12 storey building subjected to the earthquakes of 2%, 10%, and 50% probabilities were obtained as 31.1 cm, 18.0 cm, 10.4 cm, respectively Figure 4.2 (c). Accordingly, based on the results, it was appeared that the use of inverted V-shaped braces and ADAS dampers provided lower displacement in 6 storey frame building than other two frame buildings. Overall, the structures added with inverted V-shaped braces and ADAS damper performed better in terms of the maximum displacement value. Moreover, it was worthy to rote that the storey number of the structure had significant effect on such response characteristic.

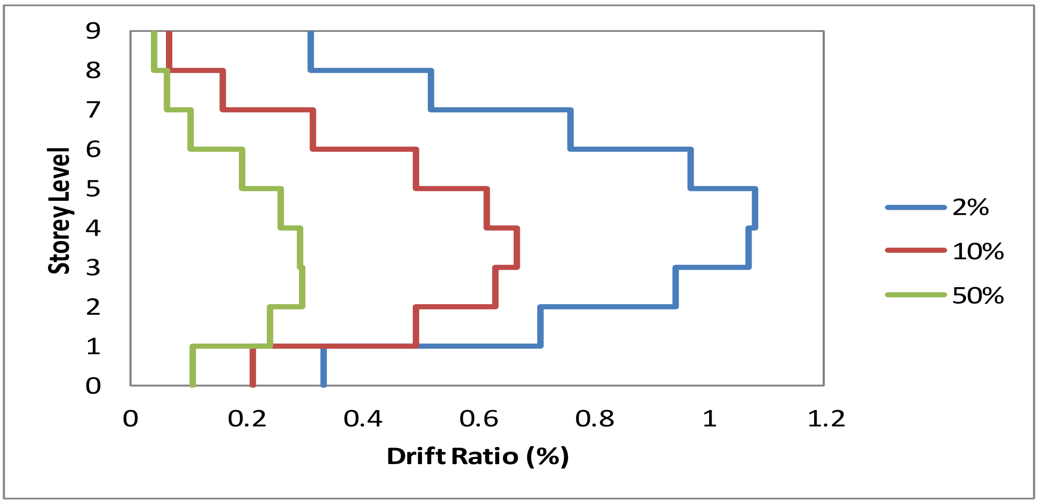
4.1.3 Interstorey Drift Ratio

The interstorey drift ratio is one of the significant response parameters that can be used on a large scale in determining the seismic performance of the structure through various ground movements. Figure 4.3 illustrates the schemes for the interstorey drift ratio of the different structures equipped with inverted V- braced and ADAS dampers. The analysis results showed that using the ADAS damper systems with inverted V- braces significantly improved on the first storey drift and reduced the interstorey drift ratio for the structures under different seismic motions. For instance, for the six storey frame as shown in Figure 4.3 (a), under the earthquakes with 2%, 10%, and 50% probabilities, the maximum interstorey drift ratio of the inverted V-braced and ADAS damped frames was obtained as 0.74%, 0.43%, and 0.25%, respectively. Under the earthquake with 2%, 10%, and 50% probabilities, the maximum interstorey drift ratio for the 9 storey frame with inverted V- braces and

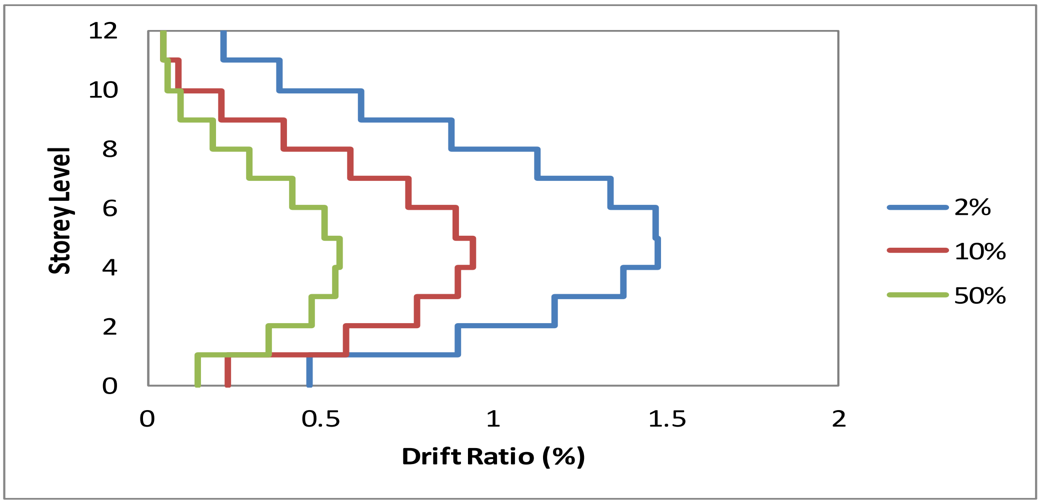
ADAS dampers, as seen in Figure 4.3 (b), was obtained as 1.08%, 0.68%, and 0.29% respectively; while the maximum interstorey drift ratio of the inverted V-braced and ADAS damped 12 storey frames was evaluated as 1.48%, 0.87%, and 0.56%, respectively, as shown in Figure 4.3 (c). As noted from the results, the addition of the inverted V braces with ADAS damper systems to the structures caused a significant reduction in the proportion of drifts especially in the first storey due to the effect of adding stiffness and damping.



a)



b)

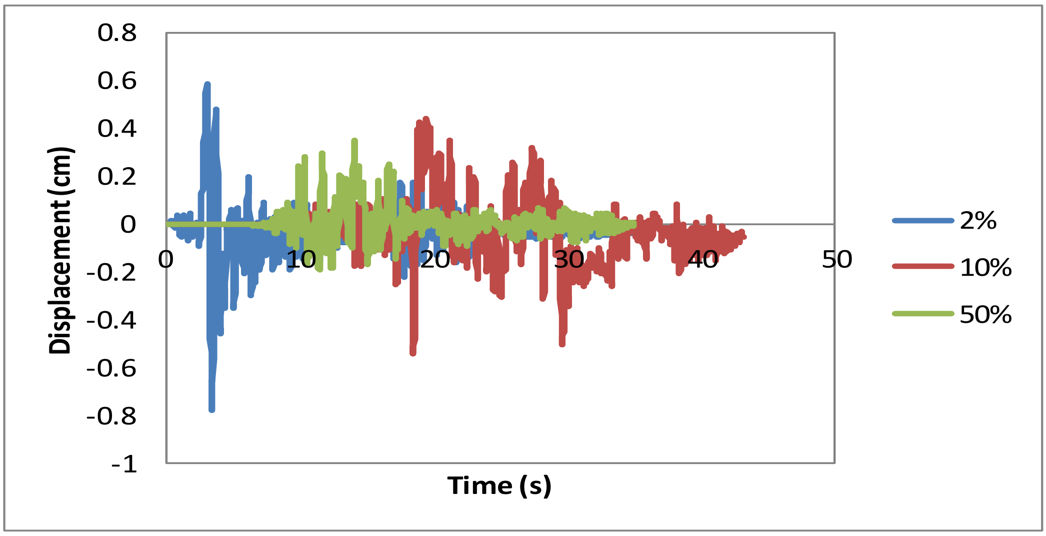


c)

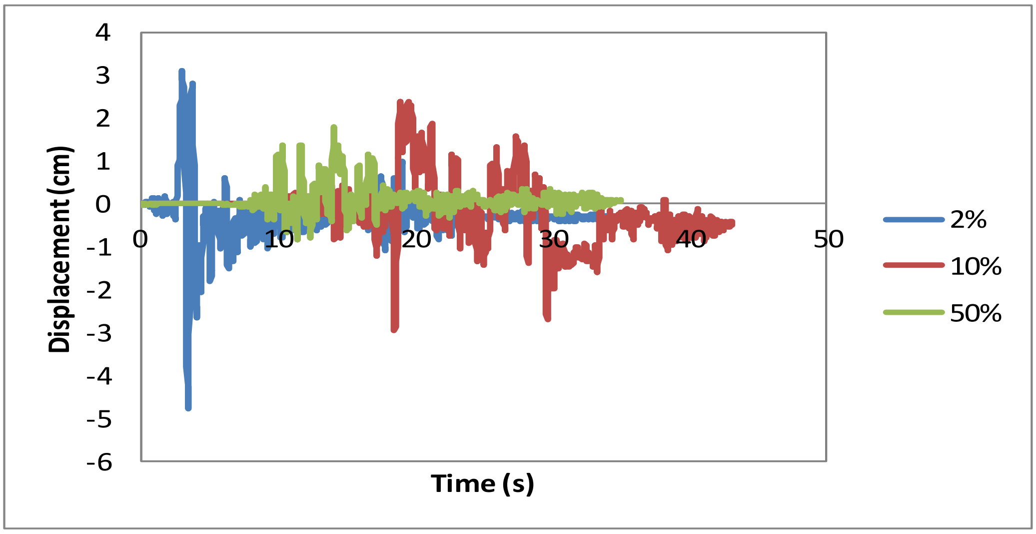
Figure 4.3 Interstorey drift ratio of a) 6 storey, b) 9 storey, and c) 12 storey buildings under earthquakes having 2%, 10%, and 50% probability of exceedance in 50 years

4.1.4 Displacement Time History

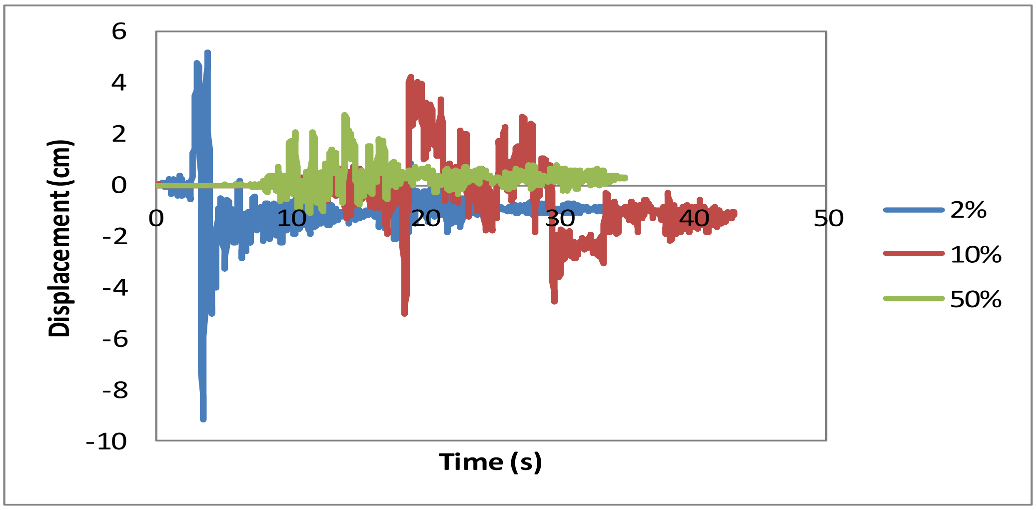
The curves obtained from the time history analysis indicated the variation of displacement with time for 6, 9, and 12 storey frame structures under three different earthquakes with 2%, 10%, and 50% probabilities (Figures 4.4 to 4.6).



a)



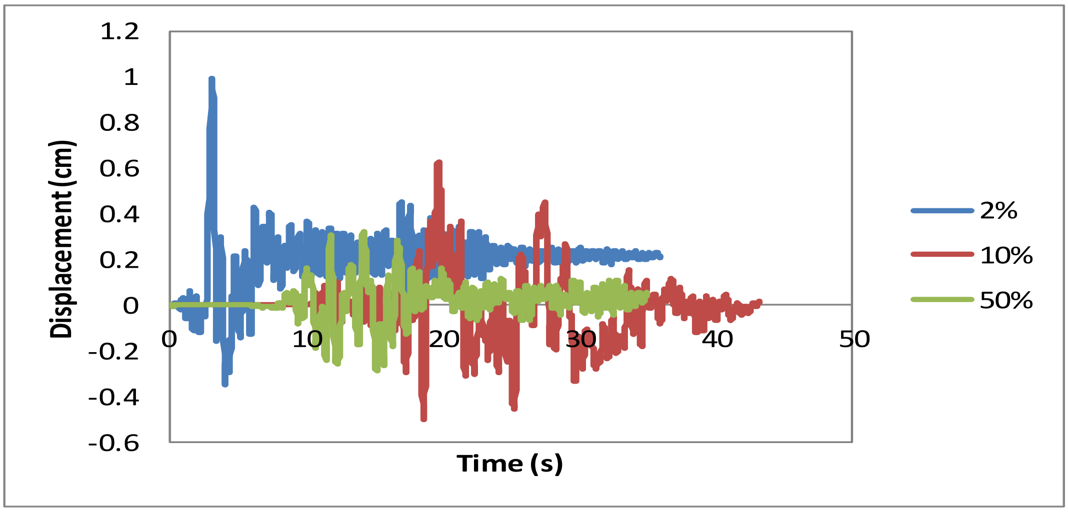
b)



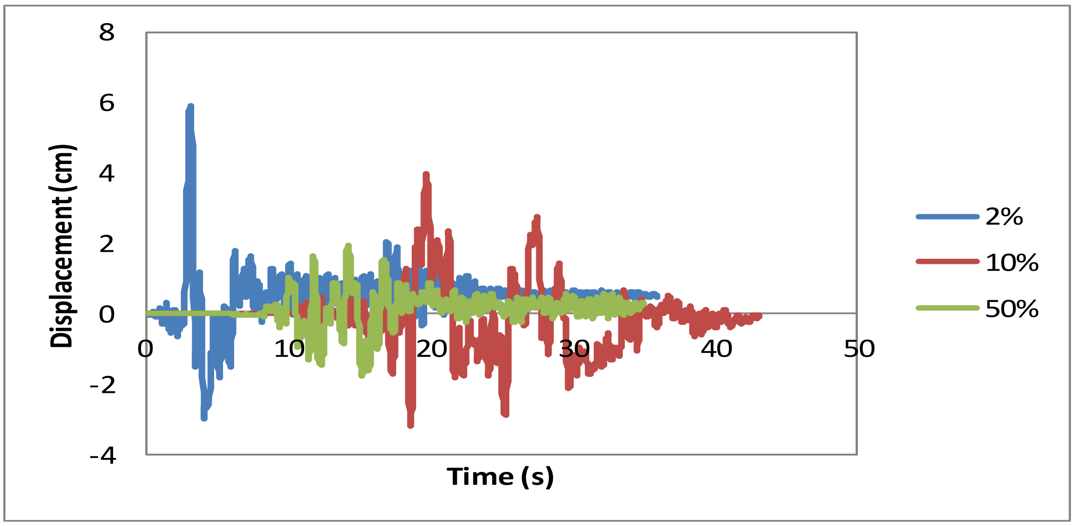
c)

Figure 4.4 Displacements versus time of a) first storey, b) third storey, and c) top storey for 6 storey building under the earthquakes having 2%, 10%, and 50% probability of exceedance in 50 years

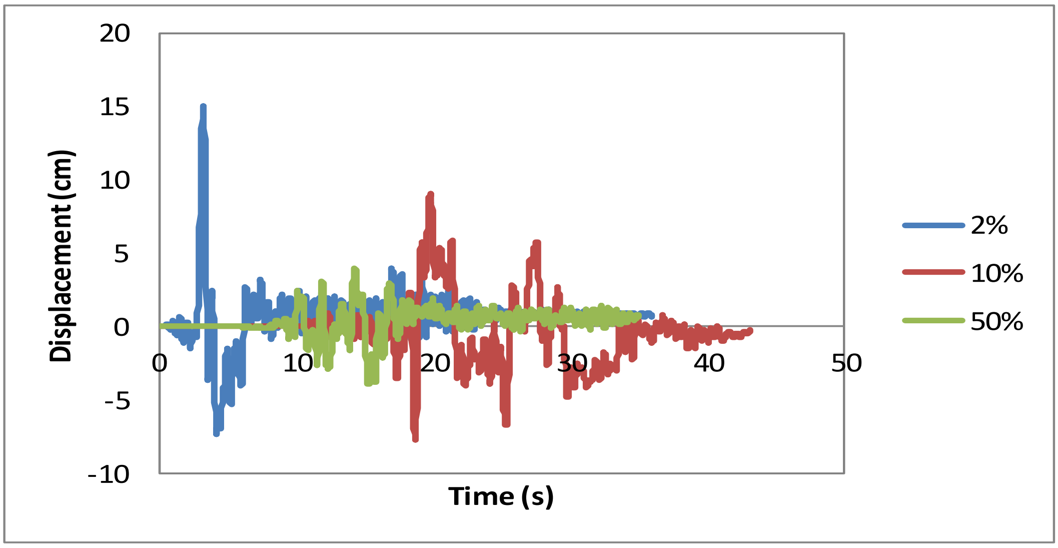
As it was clear from Figure 4.4 obtained from the nonlinear time history analysis, the maximum first storey displacement under the earthquakes with 2%, 10%, and 50% probabilities for the 6 storey frame were obtained as 0.77 cm, 0.54 cm, and 0.35 cm, respectively, while in the third storey it was obtained as 4.78 cm, 2.94 cm, 1.78 cm, respectively. The maximum top displacement of the six storey frame were obtained as 9.2 cm, 5 cm, and 2.73 cm under the earthquakes having 2%, 10%, and 50% probability of exceedance in 50 years, respectively. As a result, with increasing the seismic hazard level and storey level, the displacement demand increasing remarkably. For instance, the first storey displacement demand increased about 2 times with the change in the seismic hazard level of 50% to 2% probability of exceedance. Similarly, in the case of top storey displacement demand, this increment almost was computed as about 4 times.



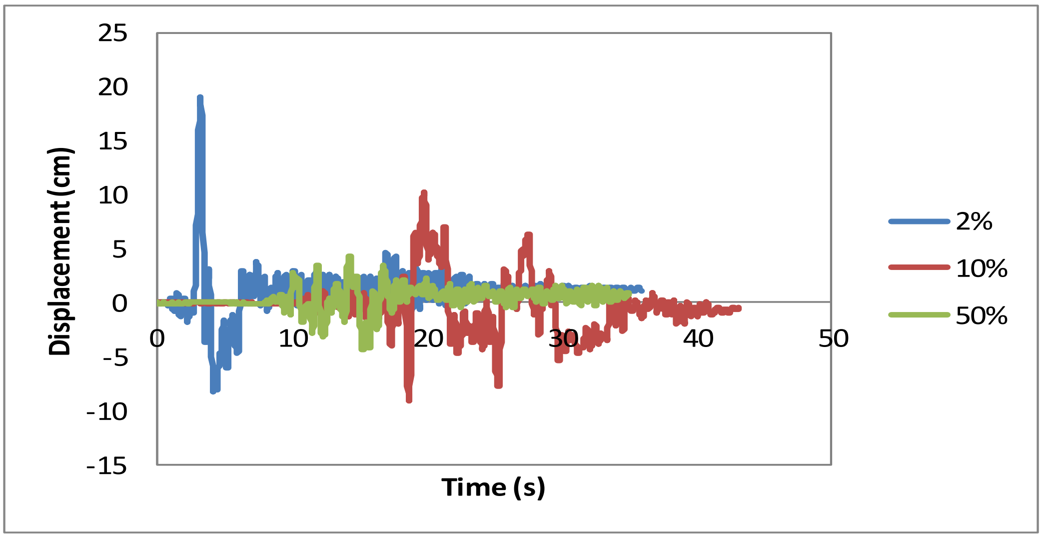
a)



b)



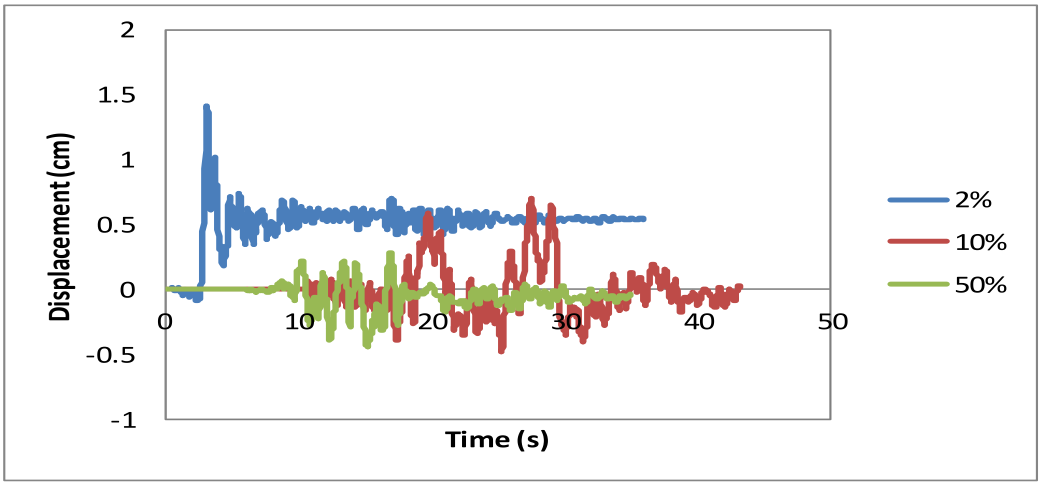
c)



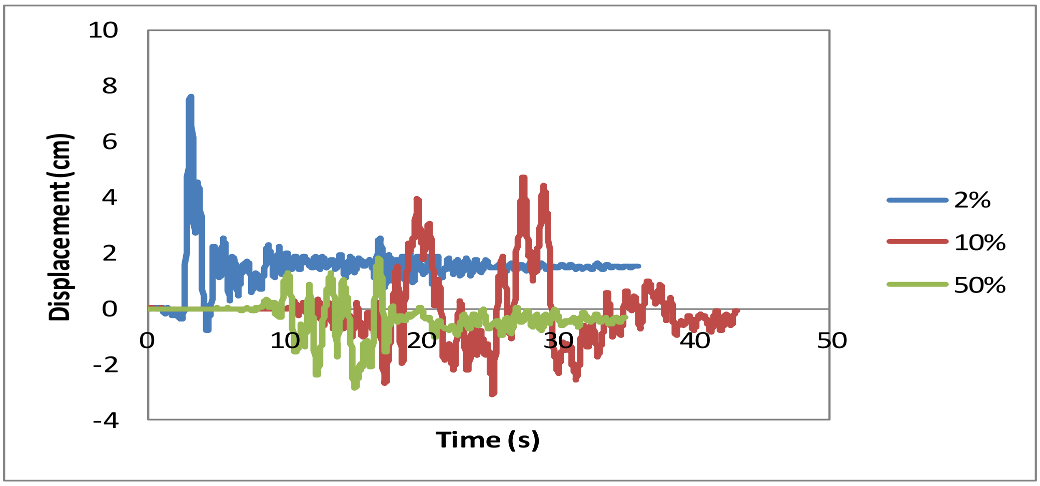
(d)

Figure 4.5 Displacements versus time of a) first storey, b) third storey, c) sixth storey, and d) top storey for 9 storey building under the earthquakes having 2%, 10%, and 50% probability of exceedance in 50 years

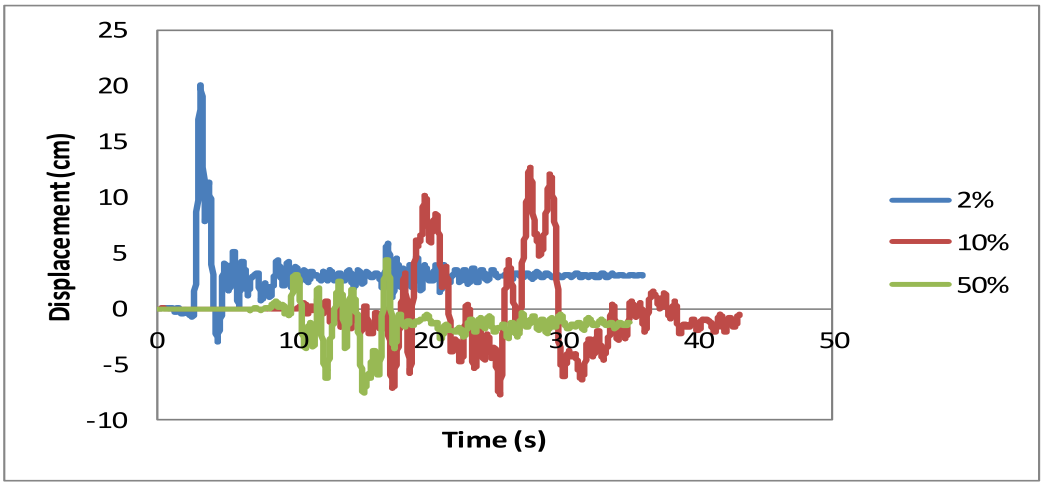
For the 9 storey frames as shown in Figure 4.5 above, the maximum first storey displacement under the earthquakes having 2%, 10%, and 50% probabilities of exceedance in 50 years were 0.99 cm, 0.63 cm, and 0.32 cm, respectively. Also for the third storey was obtained as 5.93 cm, 3.98 cm, and 1.93 cm, respectively; while the maximum sixth storey displacement were obtained as 15.05 cm, 9.08 cm, and 3.97 cm, respectively. Moreover, the top displacement was obtained as 19.15 cm, 10.34 cm, 4.38 cm, respectively under the same earthquake probabilities. As observed from the result, the maximum displacement demand increased about 3 times between seismic hazard intensity of 50% to 2% possibilities for the first displacement storey, while the maximum top displacement demand was increased about 4 times under the earthquakes between 50% to 2% probabilities of exceedance in 50 years.



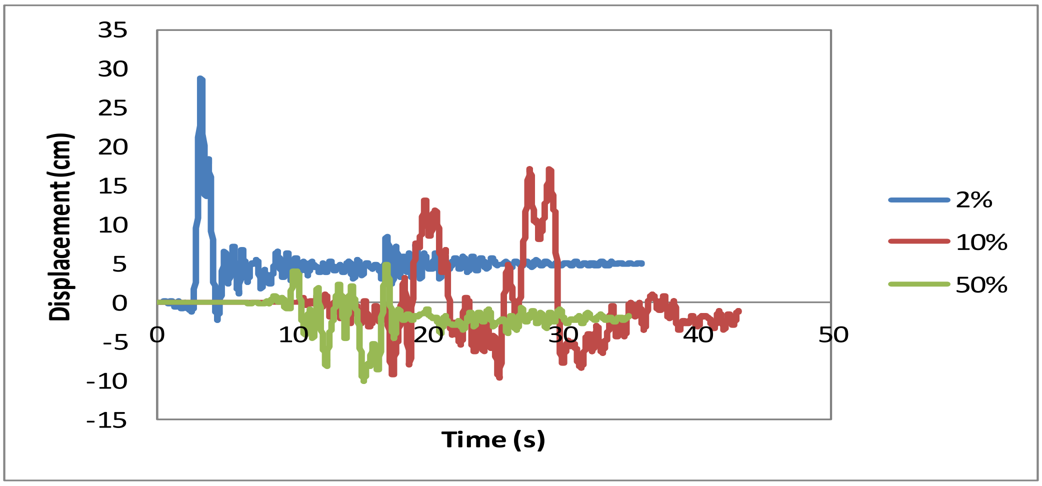
(a)



(b)



(c)



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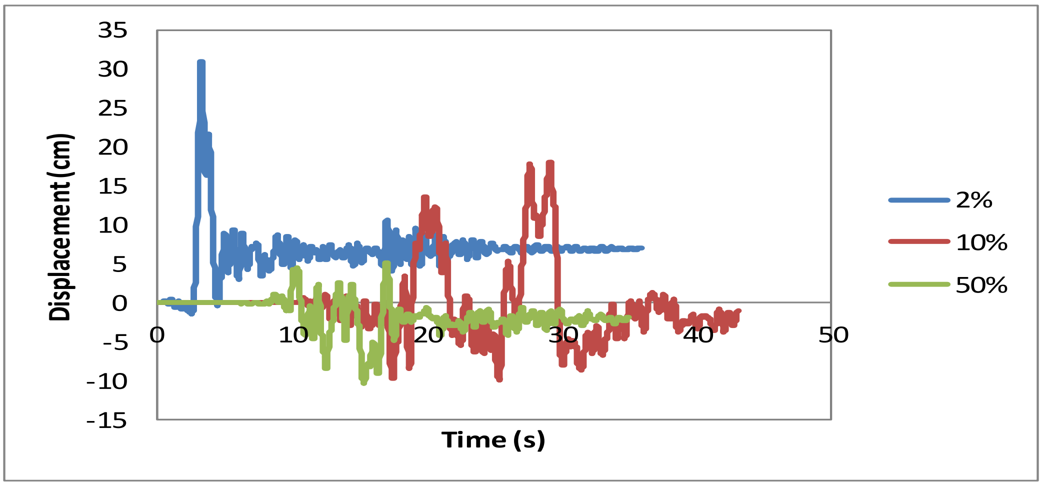


(e)

Figure 4.6 Displacements versus time of a) first storey, b) third storey, c) sixth storey, d) ninth storey, and e) top storey for 12storey building under the earthquakes having 2%, 10%, and 50% probability of exceedance in 50 years

As shown from the Figure 4.6 above for 12 storey frame structure, the maximum roof displacement for first storey were obtained as 1.41 cm, 0.70 cm, 0.43 cm, respectively under earthquake having 2%, 10%, and 50% probability of exceedance in 50 years, and also the maximum displacement for the third storey under the same earthquake probabilities were 7.63 cm, 4.75 cm, 2.90 cm, respectively while for the sixth storey was obtained as 20.17 cm, 12.69 cm, and 7.62 cm, respectively. Furthermore, for the ninth storey were obtained as 28.87 cm, 17.22 cm, and 10.07 cm, respectively and finally the maximum top displacement were 31.09 cm, 17.89 cm, and 10.40 cm, respectively under the earthquakes having 2%, 10%, and 50% probability of exceedance. As a result, the first and top storey displacement increments were almost equally obtained about 3 times between seismic hazard intensity of 50% to 2% possibilities of exceedance in 50 years.

From the result above, it was observed that using steel braces with ADAS systems in the structures had a significant role in reducing the value of the displacement demand and also the displacement-time history curves were considerably affected by the number of storey and the type of frame. For comparing the all cases, the deformed shapes of each frame were given in Appendix A.

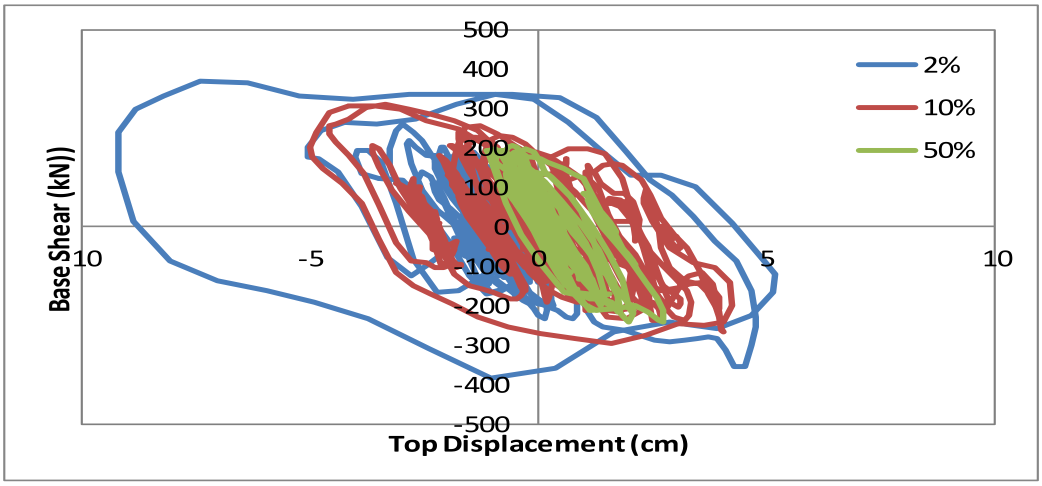
4.1.5 Hysteretic curves

Figure 4.7 presents the force-displacement hysteretic curves for 6 storey, 9 storey, and 12 storey frame buildings under the earthquakes having 2%, 10%, and 50% probability of exceedance in 50 years. The difference in the sizes of the loops is an indication of the energy dissipation in yielding at these three intensity levels. The horizontal axis represents the ratio of lateral displacement to the height of the plates.

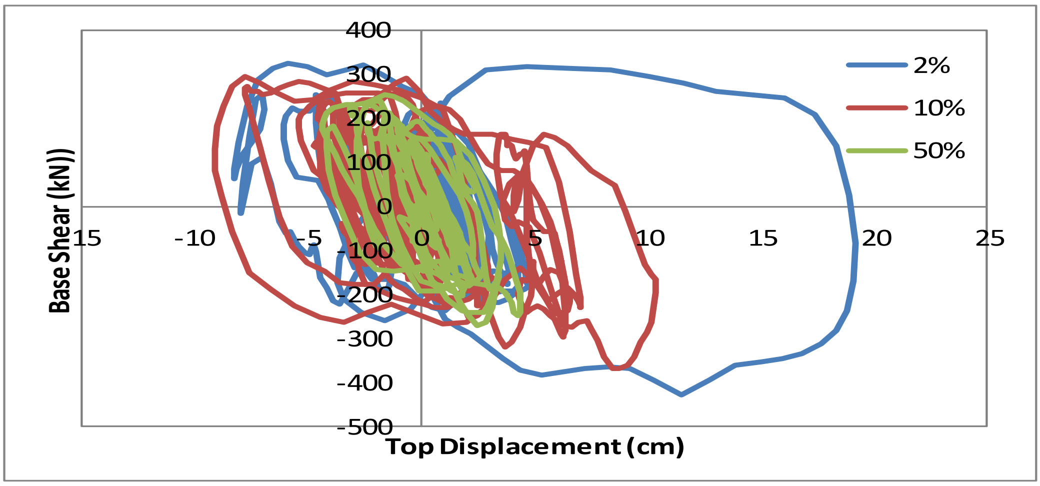
As observed from the figure, there were a large number of cyclic load reversals early failures. For example, the 6 storey frame structure under the earthquake with 2% probability sustained a large displacement as the yield force was reached 370 kN while under the earthquake with 10% probability, there was less degeneration both in stiffness or in strength. Moreover, under the earthquake with 50% probability the device dissipate energy in short displacements also with shorter shear force.

For the 9 storey frame structure under the earthquake with 2% probability, it was found that the hysteretic energy demand was great, while under the earthquake having 10% probability, there was less energy dissipation capacity and also under the earthquake with 50% probability, it was found that the energy strength demand was less than other and dissipated in short displacement. Like other cases, the 12 storey frame structure revealed the stable hysteretic performance and much energy dissipated capacity because the area of the hysteretic loops was large under the earthquake with 2% probability, while the dissipation energy was less because the hysteretic loops was smaller under earthquake with a 10% probability comparing with earthquake having 2% probability. Furthermore, the strength was less under the earthquake with 50% probability.

It was observed from the hysteretic curves that the damper provided large and stable hysteretic loops without degeneration of either strength or stiffness, and the building had a larger effect for mitigating displacement rather than shear force. Also it was observed that the hysteretic curves under the earthquakes with 2% probability was greater than other curves in all frame cases, therefore, it shows a more energy dissipation. In general it can be said that adding the damper to the desired structure can lead to the improvement of the seismic parameters of that frame in the time of earthquake. It is clear that the device enter its plasticity stage to dissipate energy through their plasticity deformation.



(a)



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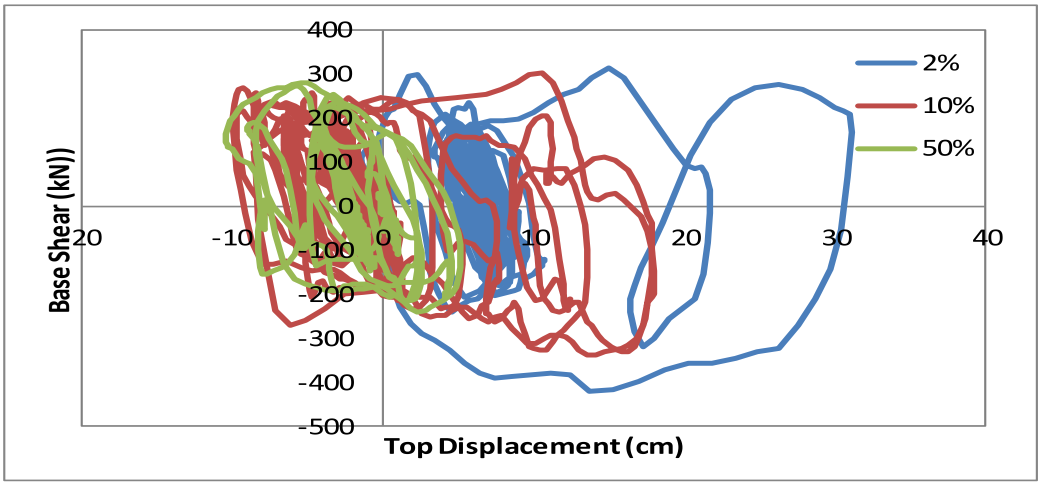


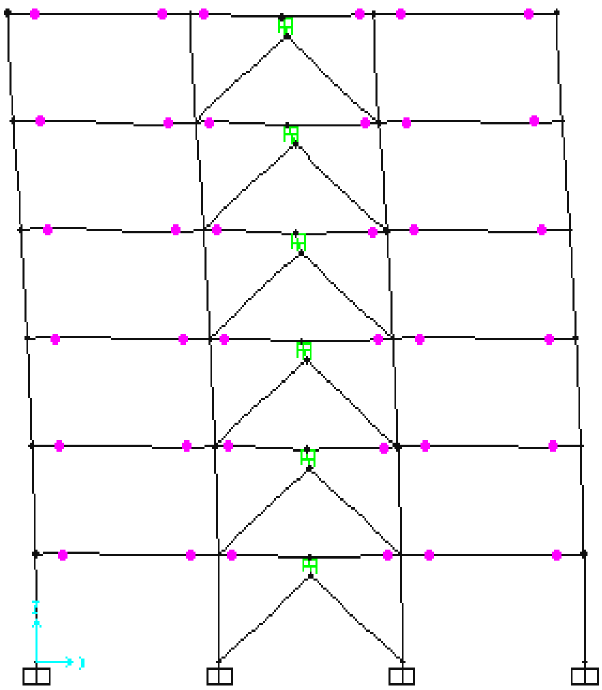
c)

Figure 4.7 Hysteretic curves of a) 6 storey, b) 9 storey, and c) 12 storey buildings under the earthquakes having 2%, 10%, and 50% probability of exceedance in 50 years

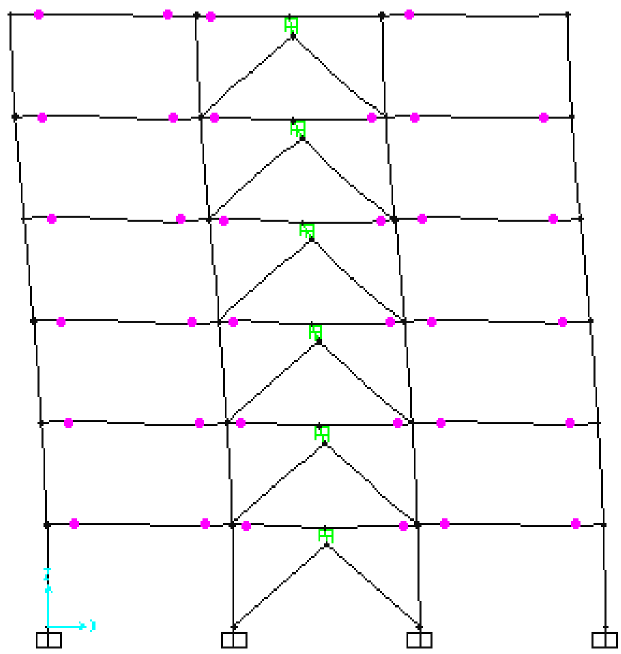
4.1.6 Plastic Hinge Formation

The occurrence of the plastic hinges in the structural members refers to failure due to the applied load. The plastic hinges are assigned to both ends of all beams, and column, according to FEMA356 (2000). Number of the total plastic hinge formations and a variety of the plastic hinges in structural elements like beams, columns and bracings for 6, 9, 12 storey buildings under the earthquakes having 2%, 10%, and 50% probability of exceedance in 50 years illustrated in Figures 4.8, 4.9, and 4.10, respectively.

At the first, Figure 4.8 for the 6-storey RC structure under the earthquake having 2% probability of exceedance in 50 years exhibited that all the hinges at the yielded state, while under earthquake having 10% probability, all the hinges at the yielded state except for two locations in the beams. Moreover, under the earthquake having 50% probability, observed that the hinges at the yielded state except for some locations in the beams. Plastic hinge formation started in the beam ends at lower storeys, and then spreaded to upper storeys.



(a)



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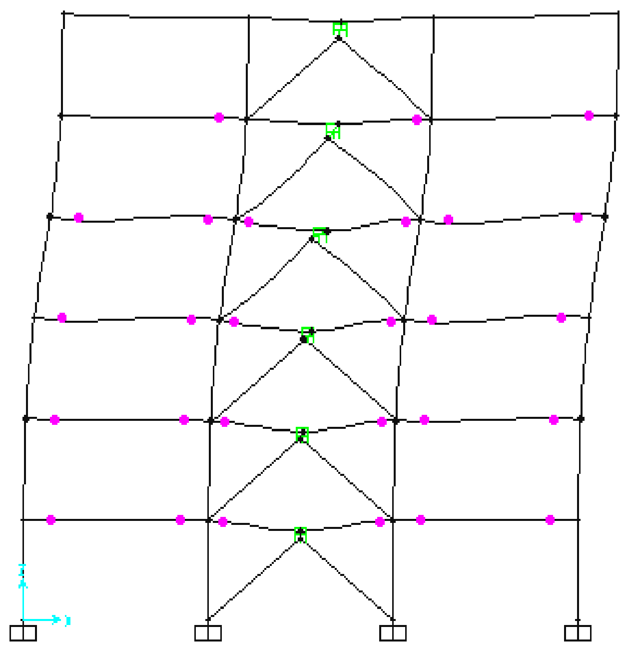
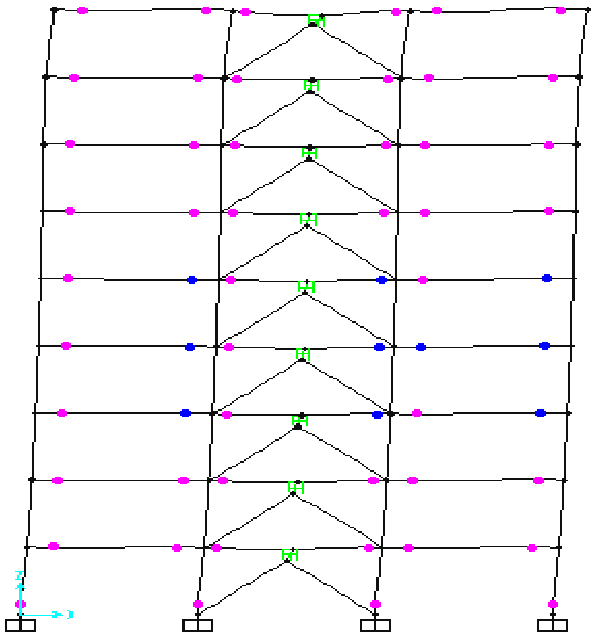


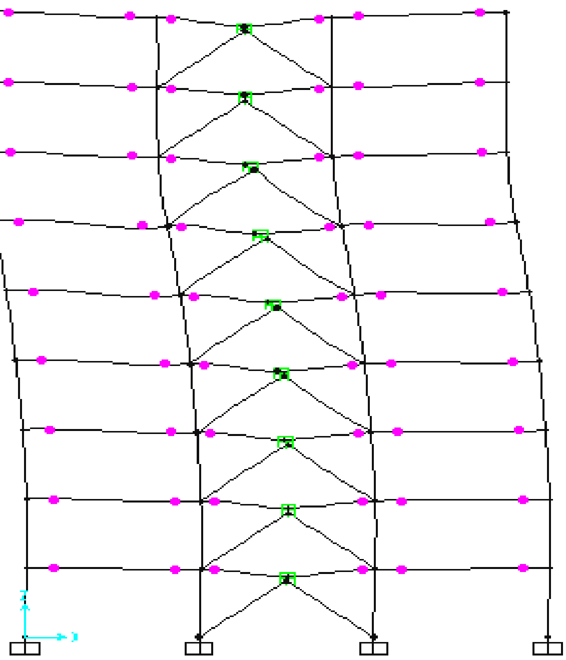
(c)

Figure 4.8 Plastic hinge formations of 6 storey buildings under the earthquakes having a) 2%, b) 10%, and c) 50% probability of exceedance in 50 years

For 9-storey frame structures given in Figure 4.9, the development of the plastic hinges under the earthquakes having 2% probability had some more yielding hinges in the top floors except in the middles that were in the immediate occupancy and just lower columns were yielded, while under the earthquakes having 10% and 50% probability of exceedance in 50 years, all the hinges were yielded and they were similar except in three locations in top floors of the beam.



(a)



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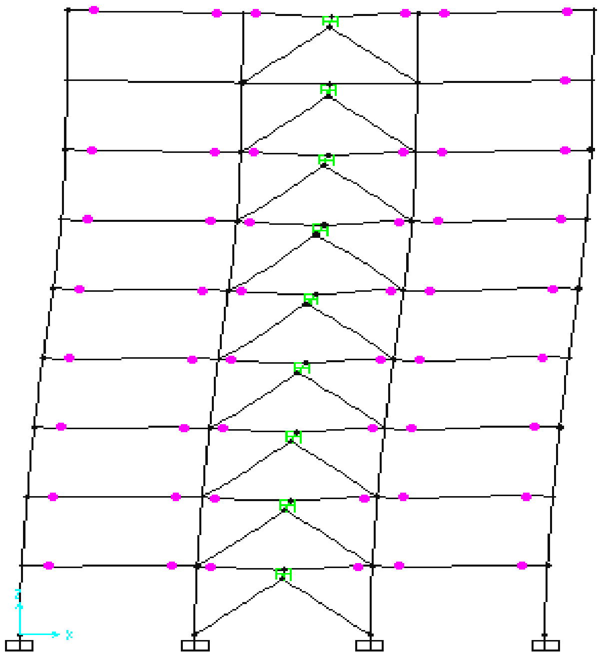
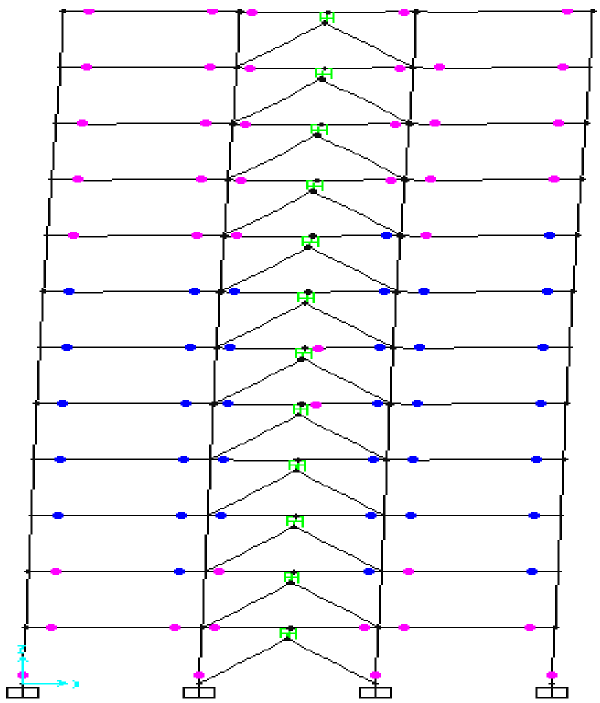


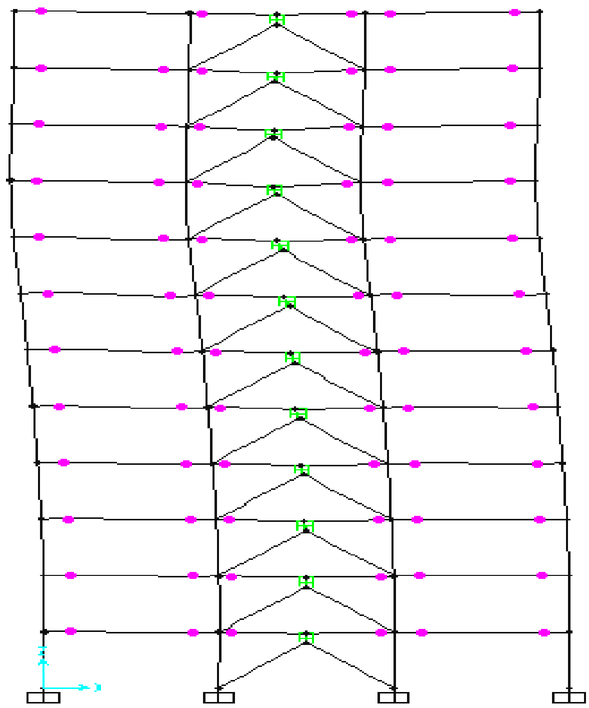
(c)

Figure 4.9 Plastic hinge formations of 9storey buildings under the earthquakes having a) 2%, b) 10%, and c) 50% probability of exceedance in 50 years

Moreover, Figure 4.10 illustrates the plastic hinge formation for the 12-storey frame structures. Under the earthquake having 2% probability of exceedance in 50 years, more hinges in the beam were yielded in the top and bottom floors while at the middle floors had some higher rotations, except that just in lower columns there was yielded hinges. Under the earthquakes having 10% and 50% probability of exceedance in 50 years, all hinges were yielded and they were almost similar except for one location.



(a)



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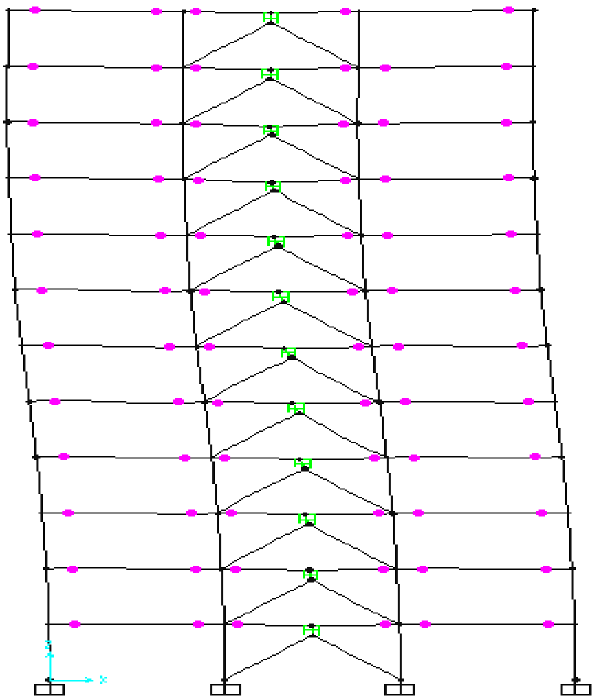

(c)

Figure 4.10 Plastic hinge formations of 12 storey buildings under the earthquakes having a) 2%, b) 10%, and c) 50% probability of exceedance in 50 years

CHAPTER 5

CONCLUSIONS

The analytical study examined the structural behaviour of the buildings equipped with inverted V- braces and ADAS dampers. The performance characteristics evaluated on the basis of nonlinear static and time history analysis. From the study results, the following conclusions can be summarized:

- It was observed from the capacity curves that the frame with ADAS dampers and inverted V- bracing systems had great capacity and particularly the shear forces of the storeys were considerably decreased. The 6 storey and 9 storey frames had higher base shear forces than 12 storey frames.

- It was evident from the variation of storey displacement curves that the studied frames were significantly affected by the given seismic records. Furthermore, the inverted V- bracings and ADAS dampers that used for the purpose of upgrading structural response of the frame significantly reduced the maximum storey displacement value which influenced by frame type and number of storeys.

- The analysis results showed that using the ADAS damper systems with inverted V- bracings improved on the first storey drift and caused a significant reduction in the proportion of drifts due to the effect of adding stiffness and damping.

- Seismic behaviour of the structure with ADAS dampers seemed to depend largely on the design criteria that were used in the model and the characteristics of the earthquake record.

- It was observed from the hysteretic curves that the damper provided large and stable hysteresis loops without degeneration of either strength or stiffness. Also, it was pointed out that the hysteretic curve under the earthquakes with 2% probability was greater than other curves in all frame cases. Therefore, it showed a more energy dissipation, it was clear that the device entered its plasticity stage to dissipate energy through their plasticity deformation.

- The results showed that the installation of ADAS dampers in the structures decreased the plastic hinge formation, also, it was observed that almost hinges were at the yielded state and occurred in the beam, except in two locations for cases 9, and 12 storey frames that occurred in the lower columns under 2% earthquake probability of exceedance in 50 years. This type of dampers played a significant role in ductility improvements and absorbing the seismic.

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Appendix A: Deflected shapes

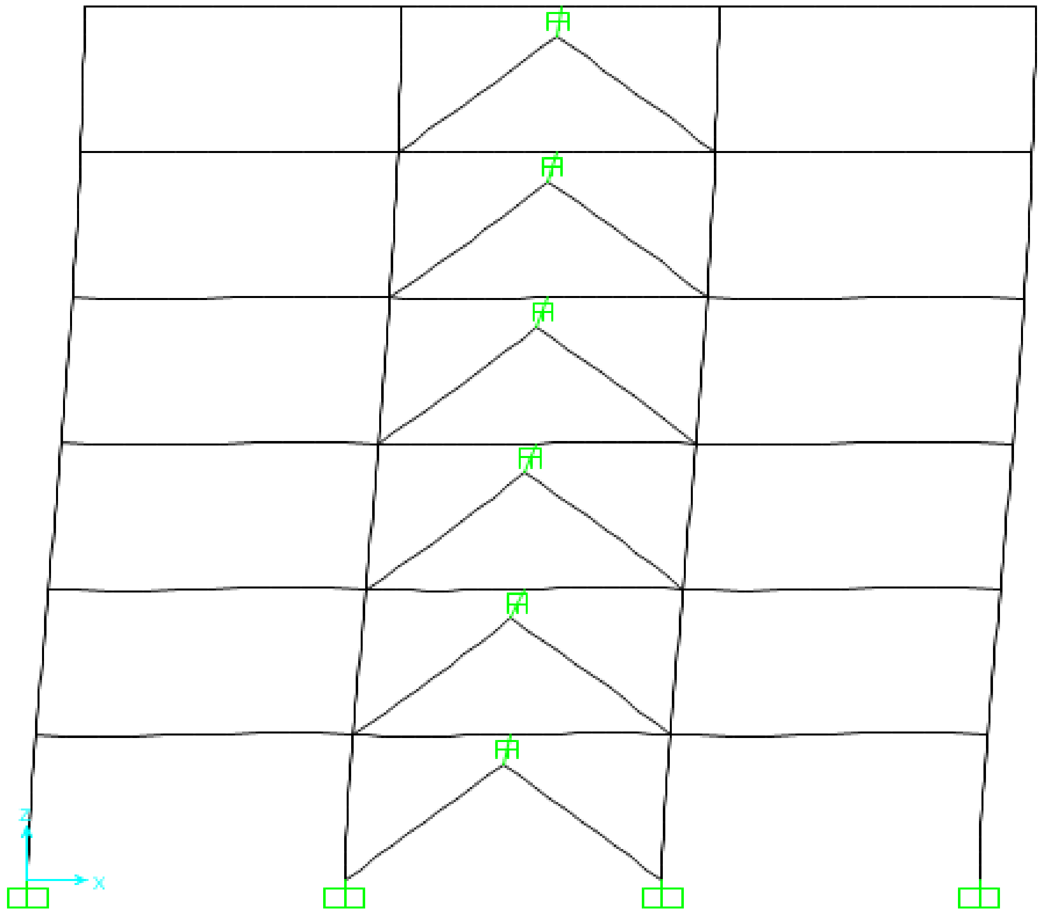

Figure A1 Mode shape of the 6 storey frame at T_1= 0.35 s

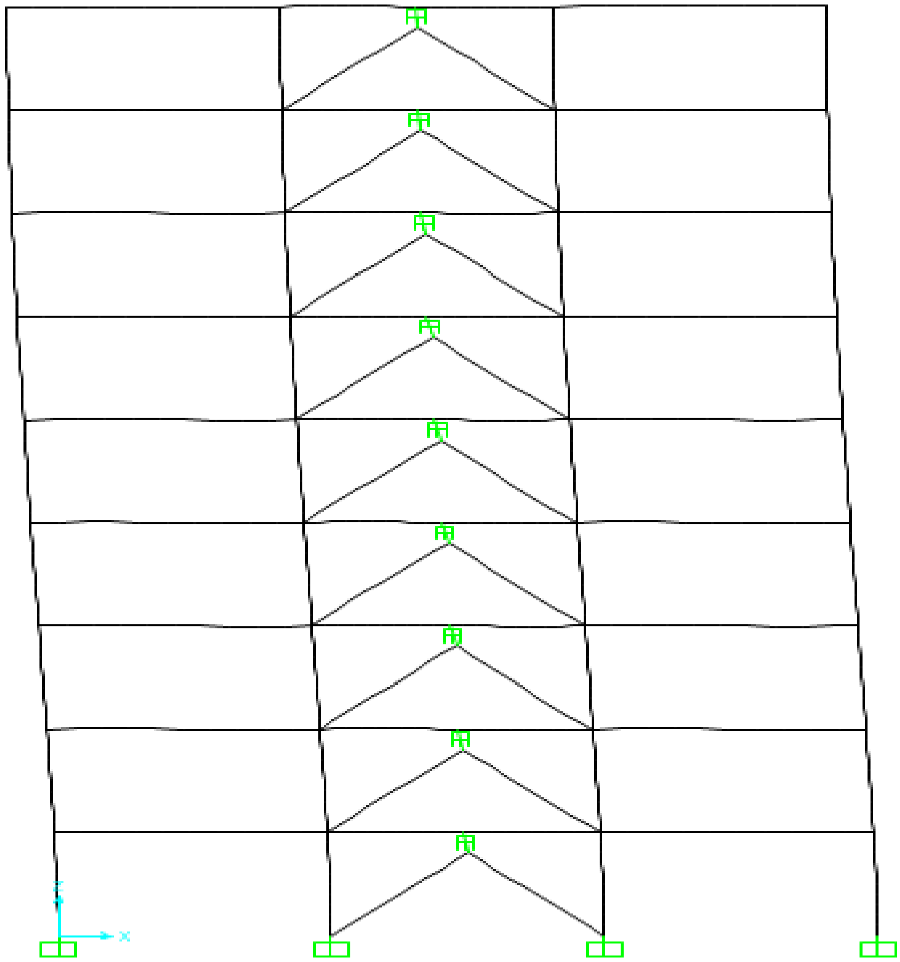

Figure A2 Mode shape of the 9storey frame at T_1= 0.52 s

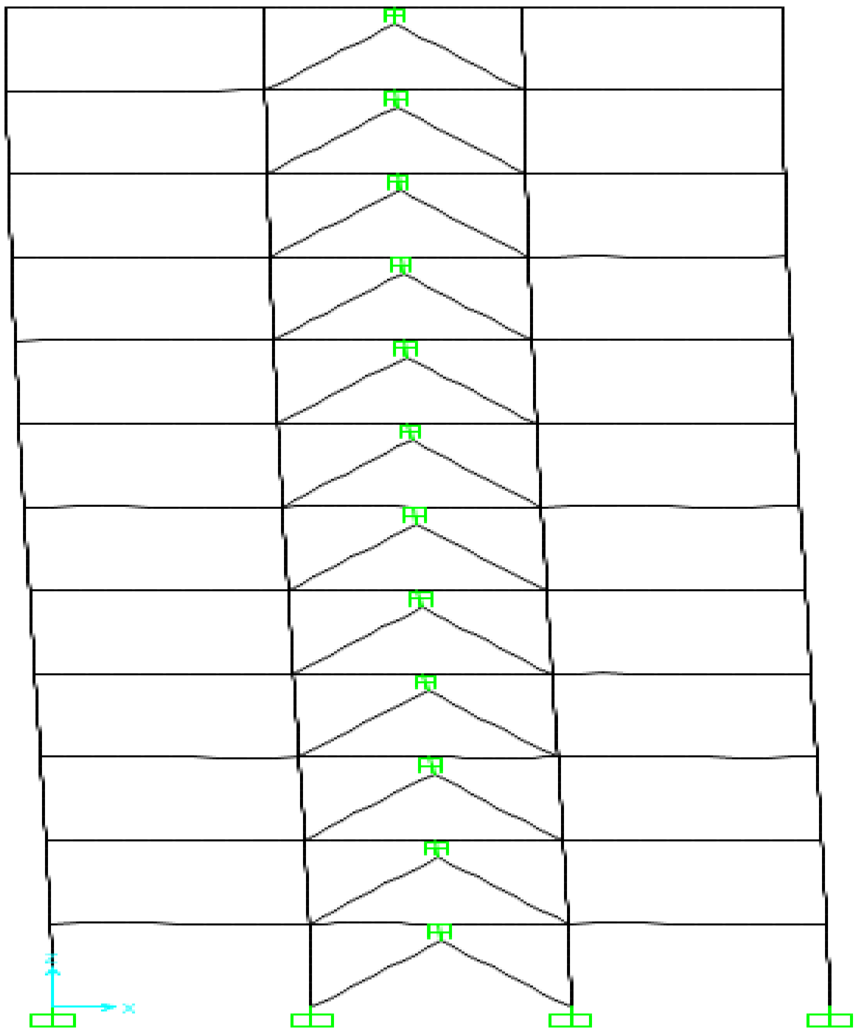

Figure A3 Mode shape of the 12storey frame at T_1= 0.72 s