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INELASTIC SEISMIC RESPONSE OF BUILDINGS WITH PASSIVE ENERGY DISSIPATION DEVICES

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Inelastic Seismic Response of Buildings With Passive Energy Dissipation Devices

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By
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ABSTRACT

INELASTIC SEISMIC RESPONSE OF BUILDINGS WITH PASSIVE ENERGY DISSIPATION DEVICES

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This thesis presents an analytical study aimed at evaluating the inelastic seismic performance of moment resisting frames (MRFs) retrofitted with different approaches. For this, a set of MRFs covering steel moment resisting frames (SMRFs) and reinforced concrete moment resisting frames (RC-MRFs) were studied. They were three regular four equal bay frame models having three different numbers of storeys (3, 6 and 12). The frames were considered to be representative of exterior moment resisting frames found in typical residential buildings. All frame models were designed with lateral stiffness insufficient to satisfy code drift and hinge limitations in zones with high seismic hazard. Three different retrofit strategies including traditional diagonal bracing system and passive energy dissipation devices such as buckling restrained brace and viscoelastic damper were utilized for seismic upgrading of the existing structures. In the nonlinear time history analysis, a group of ground motions indicating the design earthquake with 10% exceedance probability in fifty years was taken into account. Considering the local and global deformations, the analysis results in terms of plastic hinge formations, interstorey drift index, global damage index, roof drift time history, height-wise distribution of peak storey drift and base shear demand were compared. The results pointed out a considerable enhancement in the earthquake performance of retrofitted structures and it was shown that both buckling-restrained braces and viscoelastic dampers allowed for an efficient reduction in the seismic demands of the upgraded frames as compared to traditional braces.

Keywords: Buckling restrained brace; Earthquake; Energy dissipating device; Moment resisting frame; Structural response; Viscoelastic damper

ÖZET

PASİF ENERJİ SÖNÜMLEYİCİLİ BİNALARIN ELASTİK ÖTESİ SİSMİK TEPKİSİNİN DEĞERLENDİRİLMESİ

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Bu tez, farklı yaklaşımlarla güçlendirilmiş moment aktaran çerçevelerin elastik ötesi sismik performanslarını değerlendirmek amaçlı bir çalışma sunmaktadır. Bunun için, betonarme moment aktaran çerçeveler ve çelik moment aktaran çerçeveleri kapsayan bir dizi çerçeve çalışılmıştır. Bu çerçeveler, farklı katlara (3, 6 ve 12) ve dört eşit açıklığa sahiptir. Seçilen çerçeveler tipik konut yapılarında bulunan moment aktaran dış aks çerçevelerini temsil etmektedir. Bütün modeler yüksek derecede sismik tehlikeli bölgeler için tanımlanmış mafsal sınırlandırmalarını ve kod ötelenme değerlerini karşılamada yetersiz yanal rijitlik ile tasarlanmıştır. Mevcut yapıların sismik iyileştirilmesi için viskoelastik sönümleyici ve burkulması önlenmiş çapraz gibi pasif enerji sönümleyici cihazlar ile geleneksel diyagonal çapraz sistemleri içeren üç farklı güçlendirme stratejisi araştırmada kullanılmıştır. Doğrusal olmayan zaman tanım alanında yapılan analizlerde, 50 yıl içinde %10 aşılma olasılığına sahip tasarım depremi ile uyumlu bir grup yer hareketi dikkate alınmıştır. Yerel ve genel deformasyonlar dikkate alınarak, analiz sonuçları plastik mafsal oluşumları, katlar arası ötelenme indeksi, genel hasar indeksi, zaman tanımlı çatı ötelenmesi, yüksekliğe bağlı maksimum kat ötelenmesinin dağılımı ve taban kesme talebi açısından karşılaştırılmıştır. Sonuçlar, güçlendirilmiş yapıların deprem performanslarında önemli iyileşmeler olduğunu ve geleneksel çaprazlarla kıyaslandığında, iyileştirilmiş çerçevelerin sismik taleplerini azaltmada viskoelastik sönümleyiciler ve burkulması önlenmiş çaprazların daha etkili olduğunu göstermiştir.

Anahtar Kelimeler: Burkulması önlenmiş çapraz; Deprem; Enerji sönümleyici cihaz; Moment aktaran çerçeve; Yapısal tepki; Viskoelastik sönümleyici

This is a gift

To the soul of my father

God forgive him Insha-Allah

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LIST OF SYMBOLS

BRB	Buckling restrained farce
c	Damping coefficient
CBF	Conventional brace frame
CP	Collapse prevention level
E	Total earthquake energy
Ed	Energy dissipated by structural protective systems
Eh	Irrecoverable energy dissipated by the structural system through inelastic deformation or other inelastic forms of damping
Ek	Absolute kinetic energy
Es	Recoverable elastic strain energy
IO	Immediate occupancy level
LS	Life safety
MRF	Moment resisting frame

NC	Near collapse
PGA	Peak ground acceleration
PGD	Peak ground displacement
PGV	Peak ground velocity
RC	Reinforced concrete
rup	Restrict range of closest distance to rupture plane
RC-MRF	Reinforced concrete moment resisting frame
SMRF	Steel moment resisting frame
T_p	Pulse period (duration of a full velocity cycle)
T_s	Fundamental period of the frame
UBF	Unbraced frame
Vs30	Average shear wave velocity of top 30m of the site
VD	Viscoelastic damper
u_1	Peak displacements at specific time (t1)
u_2	Peak displacements at specific time after (t1)
ζ	Damping ratio

CHAPTER 1

INTRODUCTION

1.1 General

Poor performance of steel and reinforced concrete (RC) buildings was demonstrated dramatically in latest earthquakes in Turkey, Taiwan, Sumatra, and Pakistan (Saatcioglu et al., 2001; Sezen et al., 2003; Yakut et al., 2005).

These catastrophic results had seen due to the fact that the existing steel and RC buildings were either designed to gravity load only or designed to earlier codes that do not cover sufficiently lateral load carrying capacity and ductility. In order to minimize the earthquake risks and damages in these heavily populated regions, inclusive programs about seismic risk evaluation, urban planning of cities, reviewing and assessing previous designs and strengthening/demolition feasibility studies are required to be carried out. Basic element of such programs is to develop seismic upgrade methodologies that improve seismic performance of the buildings considering severe budgetary limitations (Gültekin, 2014).

Building seismic capacity can be achieved by many conventional ways such as installation of structural shear wall, structural steel braces (internal and external steel bracings), or by innovative ways such as structural protective systems. Among the above techniques, structural shear wall is the commonly used approach, which affect significantly on increasing lateral stiffness and strength of the buildings and helps in relieving the seismic loads on the deficient members under seismic attack. However, this method has disadvantages such as being time and money consuming (Özçelik, 2011).

Furthermore steel braces are known to be effective in improving lateral strength and stiffness of the structures. High lateral stiffness of the braces controls lateral deflection of the structures. Although using steel braces is very economical for low rise frame structures for seismic resistance, but steel brace members face severe strength degradation as a result of tension-compression cycles after buckling. This effect is reflected in the seismic codes through the use of relatively low response modification factors compared to those used for other structural systems (Marino and Nakashima, 2006; Tremblay and Robert, 2001).

Therefore, in addition to conventional approaches for controlling dynamic behavior and response of structures structural control systems which can be classified as passive, active and hybrid protection have been developed and investigated.

- Passive control systems are dependent on using 1- displacement (rate-independent) or 2- velocity (velocity dependent) between contacted points to create control forces or energy dissipation. The first type depends on yielding of metal core or friction between components, therefore energy do not dissipate until yielding or slip, although initial stiffness exist. The second type velocity-dependent devices which include viscous fluid and viscoelastic solid devices, can provide high levels of motion, but do not have the same energy dissipation capacity as rate-independent devices (Marshall and Charney, 2009).
- Active control systems are dependent on feeding a constant energy from outside. Because of that due to that cost of these types of dissipation systems

will be higher than the passive dissipation devices. This system controls acceleration and displacement of the buildings. Active control systems are a mix of electronic components such as computers and machines. The design of active control systems are independent from the intensity of the ground motion, because the system changes its performance according to amount of the excitation; Therefore, the unpredictable of future ground motions characteristics do not affect the designs. Active control systems have mainly three applications, such as a) active mass damper, b) active variable stiffness and c) active passive composite tuned mass dumper (Torunbalci, 2014).

- A hybrid protection system is a combination of rate dependent and rate independent devices (MagarPatil and Jangid, 2015).

Passive energy dissipation systems for seismic applications have been under development for a number of years with a rapid increase in implementations starting in the mid-1990s. The principal function of a passive energy dissipation system is to reduce the inelastic energy dissipation demand on the framing system of a structure (Whittaker et al., 1993).

Buckling restrained braces commonly found are made by encasing a core steel cross-shape or flat bar member into a steel tube and confined by infill concrete as shown in Figure 1.1 (Black et al., 2004; Sabelli et al., 2003). The two requirements to be met in the design of the BRBs, namely the axial strength to avoid material failure and flexural rigidity to avoid buckling, are provided by an axially loaded core and the sleeve surrounding it, respectively. The space between the core and the sleeve is filled with infill concrete or any other inert filler. The steel core member is designed to resist the axial forces with a full tension or compression yield capacity without the local or global flexural buckling failure. When the brace is subjected to compression forces, an unbonding material placed between the core member and the infill concrete is required to reduce the friction. Thus, a BRB basically consists of three components, including steel core member, buckling restraining part, and the unbonding material.

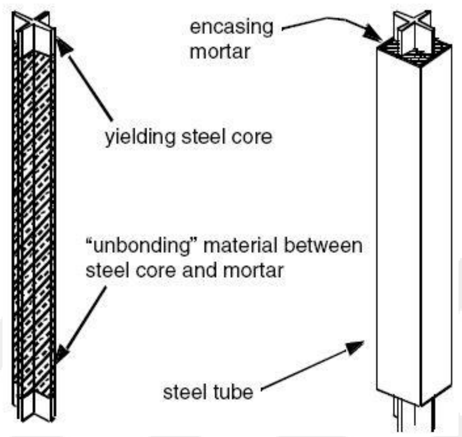


Figure 1.1 Schem of buckling restranied braces (Sabelli et al., 2003)

Viscoelastic dampers are energy dissipation devices of passive type, frequently used to mitigate excessive vibration of structures due to winds or earthquakes. The properties of viscoelastic dampers, such as the possibility of energy dissipation and stiffness, are frequency and temperature dependent and are commonly defined in terms of experimentally obtained storage and loss modules. Good understanding of the dynamic behaviour of dampers is required for the analysis of structures supplemented with viscoelastic dampers. The frequency dependence of the properties of viscoelastic dampers can be described by means of rheological models or using the complex modulus model (Lewandowski et al., 2012).

In accordance with the research need in this area, this dissertation describes an analytical investigation on the seismic behavior, evaluation, and retrofit of steel and reinforced concrete buildings. After summarizing the findings from an extensive literature review, analytical models of a series of steel (MRFs) and reinforced concrete moment resisting frames were developed to study the seismic response of buildings with and without the application of different retrofitting approaches. Based on nonlinear dynamic analyses, the behaviors of the buildings under lateral loads were investigated. Ultimately, the results can be used to develop retrofit recommendations for the seismic improvement of existing steel and reinforced concrete buildings.

1.2 Research Significance

The principle purpose of this thesis is to evaluate the seismic performance of steel moment resisting frames (MRFs) and reinforced concrete (RC) frames retrofitted with different methods. For this purpose steel and RC MRFs having three, six and twelve storeys with four equal spans of 5m were selected as models for the study. The models designed for gravity loads according to the codes (UBC, 1997 and ACI 318-8), with an inadequate capacity to satisfy code drift and hinge limitations in high seismic area. Three different retrofitting strategies, namely, conventional steel braces (CBs), buckling restrained braces (BRBs) and viscoelastic dampers (VDs), were used to improve the seismic performance of the case study frame models. The frames modeled using finite element method and evaluated by nonlinear time history analysis. Three ground motions representative of the design earthquake with 10% exceedance probability in fifty years was taken into consideration. Considering the local and global deformations, the results in terms of plastification of structure members, interstorey drift index, global damage index, roof drift history, height-wise distribution of peak storey drifts and base shear were used for comparison.

1.3 Outline of the Thesis

Chapter 1- Introduction: Objectives of the thesis were introduced.

Chapter 2- Literature review and Background: In this chapter a brief review of previous studies about rehabilitation methods that could be used in steel and reinforced concrete moment resisting frames were given.

Chapter 3- Methodology: In this chapter, the analytical models for unbraced frames (UBFs), and braced frames with CBs, BRBs and VDs were discussed. Analysis methods, materials' characteristics, ground motion records were presented in details.

Chapter 4- Results and Discussion: In this chapter, nonlinear dynamic analysis results were presented. The seismic performance of the frames with conventional braces, buckling restrained braces and viscoelastic dampers were investigated and the effectiveness of the retrofitting methods was evaluated comparatively.

Chapter 5- Conclusions: Conclusions according to the findings and results of the analysis were summarized.

Appendix A: Roof drifts time history: Roof drift time history for 3 and 12 storey steel frames were given.

Appendix B: Height wise distribution of peak interstorey drift: Height wise distributions of peak storey drifts of steel frames were demonstrated.

Appendix C: Roof drifts time history: Roof drift time history for 3 and 12 storey RC frames were demonstrated.

Appendix D: Height wise distribution of peak interstorey drift: Wise distributions of peak storey drifts of RC frames were demonstrated

CHAPTER 2

LITERATURE REVIEW AND BACKGROUND

The emphasis of previous efforts for minimizing seismic risks concentrated on retrofitting methodology for seismically deficient buildings. Different methods have been developed over the last three decades, ranging from more conventional techniques, construction of new shear walls or conventional steel bracing elements, to new and evolving technologies that involve energy dissipation systems such as dampers and control systems (Molaei, 2014). In this chapter a review of existing retrofitting methodologies prepared for more understanding these techniques and undertaking the current research effort.

2.1 Steel Moment Resisting Frames

Bussell (1997) and Ronald et al. (2009) summarized the use of iron and steel in structures from 1780 up to date. They had given very good samples of constructed steel structures in New Zealand and United States of America. The study of the cyclic behavior of steel structures started in the 1950's by Egor Popov at the University of California, Berkeley (Popov, 2001). Since that time a large numbers of studies about steel frames (both bolted and welded connections) have been conducted. In steel moment resisting frames columns are attached to the beams with no release the joints during structural analysis. Loads will be resisted by flexural in the beams and columns, which lead to producing shear forces and bending moments in the beams, columns and their connections. There are three types of moment frames; 1- Ordinary moment resisting frame, 2- Intermediate moment resisting frame, and 3- Special moment resisting frame, defined: (Varghese and Borkar, 2013).

2.1.1 Ordinary Moment Resisting Frames

Members and joints of this type of frame can resist small inelastic deformations under lateral loads. Therefore this type of frames used in low seismic areas.

2.1.2 Intermediate Moment Resisting Frames

In this system the members and joints can resist limited inelastic deformations during subjecting to lateral loads. This type of frames typically used in medium seismic areas.

2.1.3 Special Moment Resisting Frames

Special moment resisting frames' members and joints can resist significant inelastic deformation during subjecting to lateral loads. To achieve this, it is required to use pre-qualified connections per AISC or connections that have undergone and passed a qualifying cyclic test. This type of frames sustains an inter-story drift angle of up to 0.04 radians and typically used in high seismic areas.

Rodgers and Mahin (2004) examined the effects of connections' hysteretic behavior on the nonlinear dynamic performance of steel moment frames. This done by dynamic testing of a frame specimen and analytical simulations. Five types of hysteretic behavior were examined. Both the experimental and analytical portions of this study produced results which show that the effects of degradation in connection hysteretic behavior on system response are dependent on several factors. These factors include the region of the response spectrum where the system is located, type of degradation, the severity of that degradation, and the amplitude of the earthquake excitation.

Hjelmstad and Haikal (2006) presented an improved technique for modeling of joint flexibility in nonlinear analysis of MRFs. The proposed model took both panel zone and connection flexibility into consideration. For studying the effects of geometric non-linearity in joint area, they assumed finite deformations leading to a nonlinear model for the panel zone. The model has the advantage of representing the behavior of steel elements in a way that is more consistent with experimental observation.

2.2 Reinforced Concrete Moment Resisting Frames

The response of a reinforced concrete moment resisting frames (RC-MRF) under high seismic loads depends basically on strength, stiffness, ductility of the members, specifications of its individual elements and connections. Many RC-MRF designed and constructed according to low capacity for bearing seismic loads leading to severe damages or even collapse during earthquake actions. Strengthening their components or increasing their stiffness and ductility with little disturbance to the building occupants to meet requirements of modern codes always pose a great challenge to the engineers (Dung and Plumier, 2010).

Massumi et al. (2004) assessed structural over-strength of reinforced concrete structures through nonlinear static and dynamic analyses for 25 reinforced concrete frames of multi-bay, multi-storey. The variables were number of bays and storeys of the frames. The results showed that number of bays do not have any effect on over-strength of the frames. Buildings of low height showed higher over-strength proportions which decreased by about 5 to 2.5 for one to four storeys buildings, respectively. While for high-rise buildings the over-strength ratios remain approximately constant and equal to 2.5.

Bianco and Granati (2015) presented an approach to apply a non-linear static seismic analysis and evaluation of reinforced concrete frame structures previously studied. The main concept of this approach is selection of RC elements, and later on assessing the frames drift capacity related to their collapse. By applying this approach to RC frames from previous papers, a good match established.

2.3 Conventional Steel Bracing

Steel brace is one of the structural systems which used to bear lateral loads in frame structures and make the frames more stable and much stiff. Conventional steel bracing has many advantages like it's cheapness, flexibility and less space requirement which help to fit with architectural design and meeting stiffness and strength requirements. (Rai and Goel, 2003; Tafheem and Khusru, 2013). Braced frames are often used to resist lateral loads but braces can interfere with architectural features and usually placed in vertically aligned spans. This system allows getting a required sufficient stiffness with a minimum added weight, which is too important for the structure of low lateral stiffness.

Earthquake lateral forces are transferred through the diaphragms to the braced frames, after that to the diagonal members (concentric brace). The diagonal members would form tension ties and compression struts, which would help to resist lateral forces. Often these members are strong enough to withstand forces elastically or in near elastic state. However, in cases where the demand is significantly high, exceeding the elastic capacity of the bracing elements, tension ties may yield and compression struts may buckle, while maintaining their lateral force resisting capacities and dissipating seismic induced energy. Frames braced by this system called conventional braced frames (CBFs) (Tafheem and Khusru, 2013; IBC, 2000; Molaei, 2014).

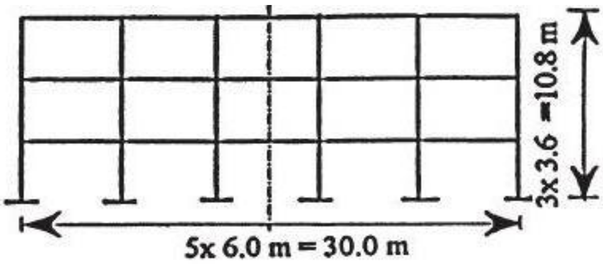
Eccentric bracing is used to improve lateral stiffness and energy dissipation performance of frame structures. This type of bracing is connecting to the beams eccentrically for that the lateral stiffness of the system will be governed by flexural stiffness of the beams. The load from earthquakes will dissolve into two components horizontal and vertical. The horizontal component will be transfer to the beams as a concentrated force through the connection of the bracings. (Tafheem and Khusru, 2013; IBC, 2000).

Behavior and performance of conventional braced frames for seismic loads were widely studied experimentally and analytically by many researchers. (Uang and Bertero, 1986; Whittaker et al., 1990; Jones et al., 2002; Ricles et al., 2002; Khatib et al., 1988; Roeder, 1989; Remennikov and Walpole, 1997; Tremblay, 2002; Kim et al., 2014).

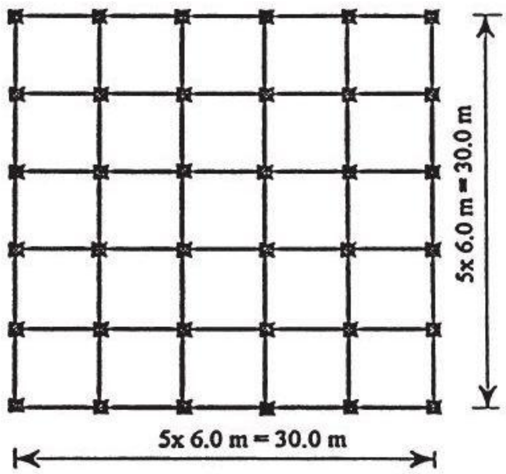
Maheri and Sahebi (1997) conducted a number of tests on reinforced concrete frames braced with different configurations of diagonal bracing. Behavior of the frames and braces in tension and compression has been studied and investigated. The results showed that adding diagonal braces (acting either in compression or tension) will increase shear capacity of reinforced concrete structures by about 2.5 times.

Ghobarah and Elfath (2011) investigated seismic characteristics of non-ductile RC structure retrofitted with eccentric steel bracing. For this purpose reinforced concrete building as shown in Figure 2.1 has been analyzed under 12 different earthquake records. The performance of the non-ductile RC frame members was described using a beam–column element able to modeling the strength softening and the effects of the axial force on the yield moment and the deformation capacities at peak strength of these members.

The results showed that an accurate selection of eccentric bracing configuration and strength for the building heights should be followed in order to get the best uniform storey drift for the building, because the distribution of the brace within height of the structure has a significant effect on plastic hinge specifications during seismic load application.



a)



b)

Figure 2.1 Typical RC Model a) front view and b) plan view (Ghobarah and Elfath, 2000)

El-Hifnawy et al. (2005) evaluated ductility characteristics of low-rise braced steel buildings using static pushover analysis. The building was a 6-storey braced steel frames which were designed according to the Egyptian code. Three different types of concentric brace configurations as well as one eccentric brace configuration were considered in this study as shown in Figure 2.2. The concentric brace cases include regular-X, split-X and inverted-V configurations. DRAIN-2DX computer program used for dynamic analysis of the buildings. The brace elements were modeled using a buckling element which is able to demonstrate the post-buckling behavior of brace elements. The results showed that the eccentric bracing shows the best deformability among all the bracing cases and concentric bracing cases designed according to a strength approach showed poor performance. The regular-X brace configuration exhibits the best performance among the concentric bracing cases.

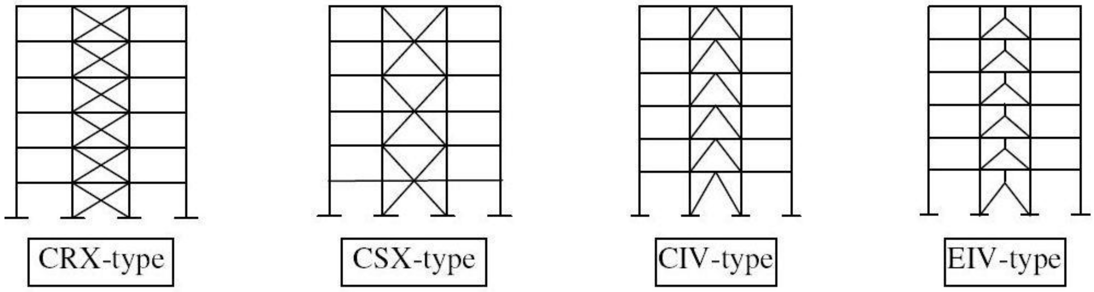


Figure 2.2 View of braced frames (El-Hifnawy et al., 2005)

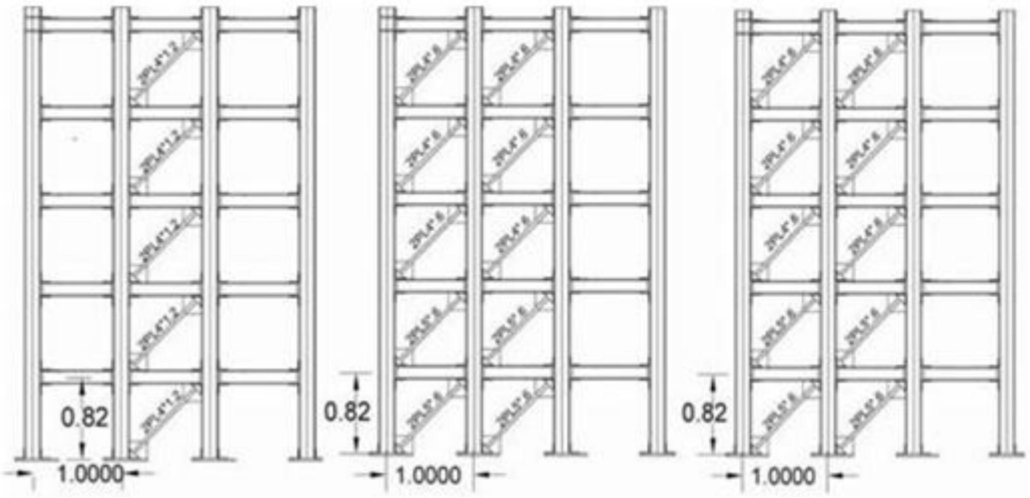
Kim and Choi (2005) evaluated the over-strength, ductility and the response modification factors (RMFs) of special concentric and ordinary concentric braced frames by conducting static nonlinear analysis for different frames taking number of storeys and span lengths as variables. A comparison done between the achieved results and those from nonlinear dynamic analyses. The results showed that RMFs of the models which calculated from the static push over analysis are less than the amount which specified in design codes. The results will not be accurate for low-rise special concentric braced frames.

Youssef (2007) evaluated the behavior of braced RC frames through an experimental study carried throughout testing two specimens under two cyclic loading. Available seismic codes used in design of the members (frames and braces). A rational design methodology was accepted for designing the braced frames and connections between the brace and concrete frame. The results showed that by applying bracing for the frames a significant increase in resisting lateral loads will observe, and the ductile behavior of the frame improved. Designing of braced frames by this method provide a very good seismic performance in accordance with other design methods.

Kim et al. (2011) analyzed eight steel bracing frames by conducting push over and time history analyses for investigation of progressive collapse potential of the frames. The analysis indicated that the frames remain stable after sudden removal of a column. Results of comparison conducted between braced frames, SMRF and MRF showed that the braced frames showed less drift and deformation.

Özel and Güneyisi (2011) studied seismic reliability of a six storey mid-rise reinforced concrete buildings retrofitted using different types of eccentric steel braces through fragility analysis. They constructed a fragility curve as a function of PGA utilizing nonlinear time history analysis to investigate the effect of type spatial distribution of eccentric steel braces. The researchers found out that the simulated fragility curves of reinforced concrete frames after retrofitting with steel braces showed improvement (less fragile) compared to those before retrofitting by as much as 1.8 times based on median PGA values simulated.

Heidari et al. (2011) conducted an experimental study on six braced frames as shown in Figure 2.3. Three brace configurations has been studied as shown in the same figure. The results indicated that the third pattern showed better performance and higher inelastic capacity and therefore greater response modification factor (R) value. First and second bracing schemes for most of the cases did not show same performance and the R value was not sufficient according to UBC-97.



(a) (b) (c)

Figure 2.3 Bracing configurations: a) 1st bracing schemes, b) 2nd bracing schemes and c) 3rd bracing schemes (Heidari et al., 2011)

Sangle et al. (2012) analyzed a series of tall steel buildings with different configuration of braces. A linear time history analysis for Northridge earthquake applied for the models. Seismic performance including natural frequency, basic time period, global damage index, interstorey drifts and base shear of the frames for all steel brace configurations were compared. The results showed that bracing elements have very significant effects on structural behavior under earthquake excitation. The results also indicated that by bracing frames in two directions the base shear will increase and the roof drift will decrease.

Gültekin (2014) investigated the seismic performance of existing reinforced concrete buildings retrofitted with inverted V and zipper braces. Nonlinear static and time history analysis by using a set of natural ground acceleration were performed. The analysis results lead to concluding that by adding inverted V and zipper braces the building load carrying capacity will increase. Zipper brace effect on performance of the building is more than inverted V brace and both braces decreased significantly global damage index and roof drift of the building comparing to the unbraced buildings.

Han et al. (2015) investigated effect of width/thickness ratio (w/t) of hollow square steel section braces on the hysteretic behavior, buckling mode, failure energy dissipation capacity of the bracing members. The results showed that decreasing of w/t ratio lead to delaying in initiation of the occurrence of local buckling and increasing energy dissipation capacities and ductility capacity. However, this effect becomes low below a certain w/t ratio because of the concentration of strain at ends containing a slot hole.

Maddala (2013) investigated a series of six storey steel moment resisting frames designed according to Indian Code (IS800-2007) with different configurations of eccentric steel bracing such as diagonal K and V bracings. Nonlinear static analysis results showed that bracing increases the base and top shear of the frames and decrease the roof displacement. Pushover curves and performance of the braced frames were compared to find the relative performances of various frames which considered.

Ferrario et al. (2016) carried out an analytical analysis and experimental study to obtain new criteria for using high strength of circular hollow steel columns under earthquakes excitation. A series of seven full scale models of five storey building were tested under effect of different loading histories. Using Open-Sees program, models for the bracing frame were developed and calibrated against the experimental responses. Static and dynamic nonlinear analysis results compared to the experimental results. Evaluating of the analytical and experimental results indicated that the performance base method can be extended to concentrically braced frames with high strength tubular steel columns, when their failure and collapse under earthquake is prevented.

However, CBFs had exhibited certain unfavorable modes in the severe earthquakes as a result of strength and stiffness degradation and excessive flexure of beams. These potential problems are due to buckling deformation of braces and limited capacity in compression, as shown in Figures 2.4 and 2.5 (Becker, 1995; Qiang, 2005).

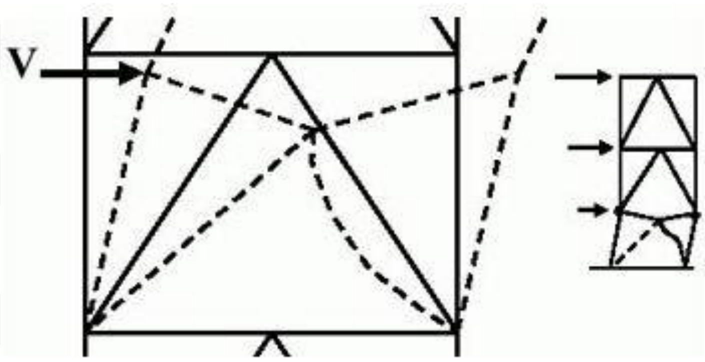


Figure 2.4 Buckling of chevron braces under lateral loads (Becker, 1995)

Figure 2.5 Behavior of conventional brace and buckling restrained brace (Qiang, 2005)

Many researchers investigated inelastic behavior of CBs when loaded cyclically and these studies showed that the hysteresis behavior of such axially loaded members depend on step by step losing of compressive capacity and weakening of stiffness in tension. As a consequence of these behaviors, in the design process the brace selected for some storeys are usually stronger than required while braces in the other storeys have a capacity near from the design. These discrepancies lead to concentration of the earthquake damages in few 'weak' storeys when some of the braces buckle prior to others. It was also observed that buckling of the conventional braces results in a substantial damage to the adjacent nonstructural members (Tremblay et al., 1995; Krawinkler et al., 1996; Sabelli et al., 2003; Mahmoudi and Zaree, 2010).

Behavior of concentrically braced frames is strongly depending on the cyclic inelastic behavior of the braces. However, CBs have a complex hysteresis behavior when loaded cyclically as shown in Figure 2.6. That is due to following physical phenomena a) Yielding in tension, b) Buckling in compression, c) Degradation of compressive capacity in post buckling range, d) Degradation of axial stiffness, and e) Low cycle fatigue fracture (Mahmoudi and Zaree, 2010).

The previous physical factors make the analytical modeling complicated. Nevertheless, in practical analysis, accurate prediction is required in modeling the inelastic range for each component as stated in FEMA 273 (FEMA 273, 1997).

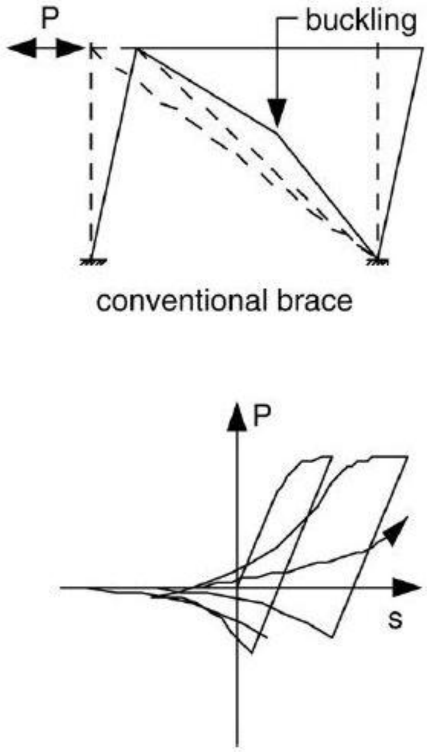


Figure 2.6 Hysteresis behavior of conventional brace (Qiang, 2005; Mahmoudi and Zaree, 2010)

2.4 Energy Dissipation Systems

During the lifetime of any structure, many severe events such as earthquakes and winds that might effect on structural system may impact the structural performance and cause potential damage (Bitaraf et al., 2012). To minimize effect of earthquakes and strong winds on the structures , new energy dissipation devices are proposed to be used as part of structural protection systems (Yang et al., 2010).

These retrofitting techniques were proposed to decrease the overall seismic drift demand of the structures, while also enhancing its lateral load resistance. They consist of passive and active control systems and base isolation systems. Most of these systems aim to improve properties of the original structure. The energy exchange between a structure and the ground during a seismic event can be summarized by the concept of the conservation of energy. According to Uang and Bertero (1988), the total energy is broken down to the following components:

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

Where;

E : Total earthquake energy;

E_k : Absolute kinetic energy;

E_s : Recoverable elastic strain energy;

E_h : Irrecoverable energy dissipated by the structural system through inelastic deformation or other inelastic forms of damping;

E_d : Energy dissipated by structural protective systems.

Commonly used and recently researched global retrofitting options are presented in the following sections. Each of these methods has contribution(s) to one or more of the structural response terms given in the equation above (Molaei, 2014).

During severe earthquakes, large amount of kinetic energy will fed to structures. Applying global modifications to structural and nonstructural components may generally increase the design demand of the elements as shown in Figure 2.7 (Bozorgnia and Bertero, 2004; Di Sarno and Elnashai, 2009).

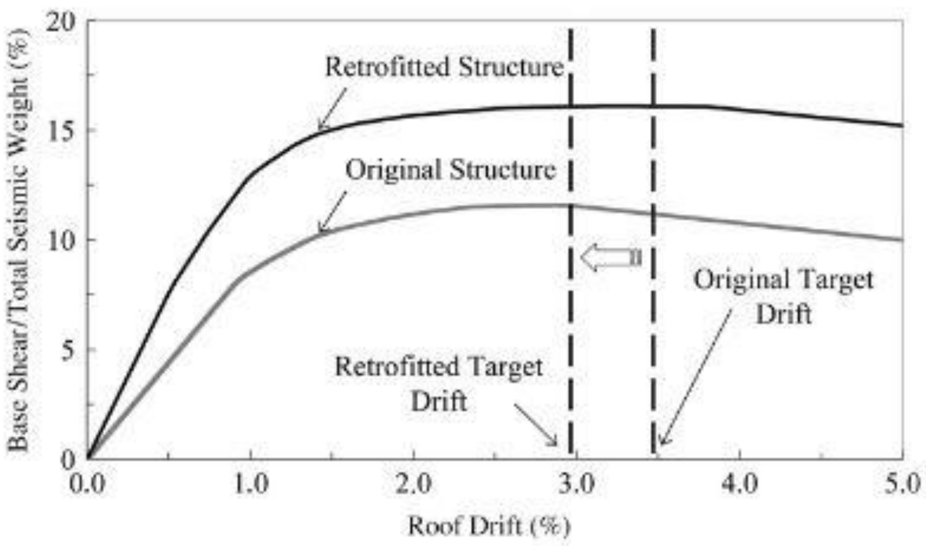


Figure 2.7 Effect of global intervention approaches on seismic retrofitting of structures (Di Sarno and Elnashai, 2009)

International building codes assured that resisting the earthquake energy by increasing elasticity and seismic capacity of the building only is not economic and practical. Thus, it is better to accept that yielding occurs in the structure but in such a way that plastic deformation would be concentrated at specific locations or structural fuses and major structural members remain in elastic manner (Deulkar et al., 2010).

2.4.1 Buckling Restrained Brace (BRB)

In order to overcome limited energy dissipation capacity and many other potential difficulties associated with utilizing conventional braces as a lateral load resisting system (Qiang, 2005; Asgarian and Amirhesari, 2008), in United States since 1994 Northridge and California earthquakes a new lateral force resisting technique established. This new system was called as “buckling-restrained brace (BRB) system” and it was accepted for practical applications in high seismic regions (Kiggins and Uang, 2006). The concept of the BRB is simple, providing lateral support to the brace so that buckling deformation is prevented, as shown in Figure 2.8 (Ameen, 2012).

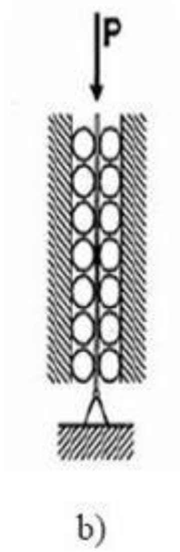


Figure 2.8 Idea of a) CBF and b) BRBF (Ameen, 2012)

The concept of BRB is to produce a structural member behaving in constant manner under compression and tension load and deformation. Figure 2.9 shows schematically the concept of a BRB and its components. BRBs' components can be summarized as:

a) Metal core: This part will dissipate energy by axial deformation (BRB core should yield). The core can be in different shapes as shown in Figure 2.10 (Zhao, 2014);

b) Mortar, concrete or grout fill that prevent buckling of the core; and

c) A steel jacket that cover the mortar.

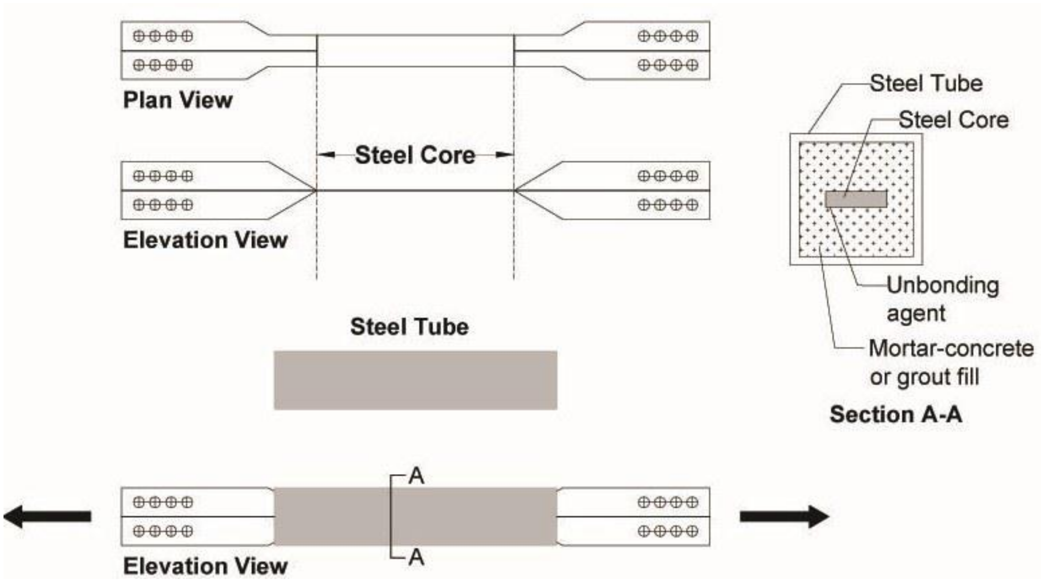


Figure 2.9- Schematic configuration of BRB (Tremblay et al., 2006)

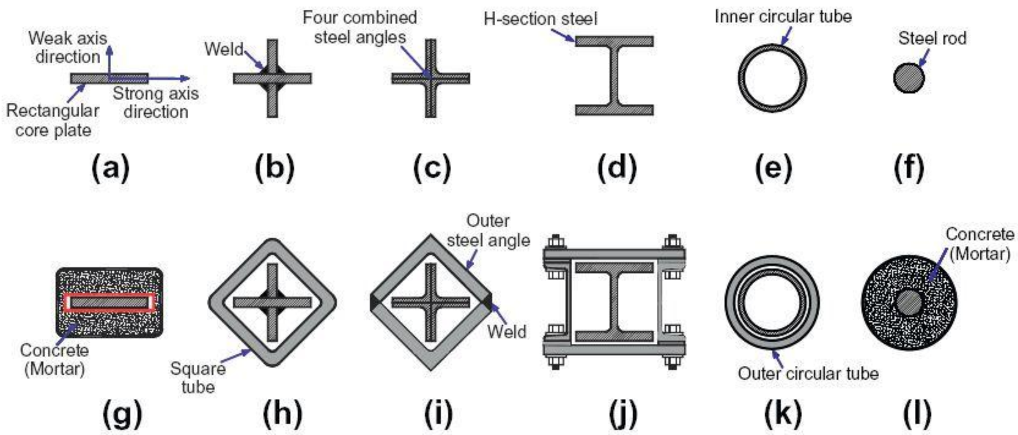


Figure 2.10 Typical BRB core sections (Zhao, 2014)

Typically the steel core is separate from the mortar grout fill in order to prevent transferring stresses between the core and the grout; in order to keep the compression strength of the brace same as tension strength (Teran-Gilmore and Ruiz-Garcia, 2011). In the literature, various studies are available to show the response of different BRBs under monotonic and cycling loadings. All the studies showed that BRBs exhibit symmetric hysteresis behavior with high energy dissipation capacity through stable tension-compression yield as shown in Figure 2.11. (Gu et al., 2014; Saeki et al., 1995; Iwata et al., 2000; Black et al., 2002; Kim and Seo, 2004; Sabelli et al., 2004; Aiken and Kimura, 2001).

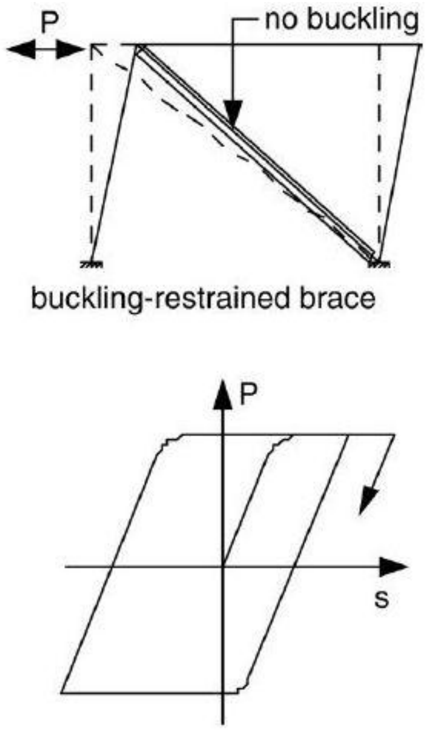


Figure 2.11 Hysteresis behavior of buckling restrained brace (Qiang, 2005)

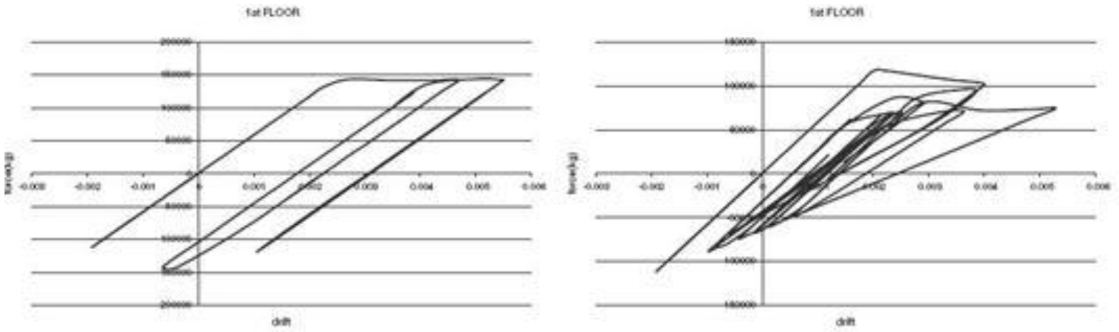
Kim and Choi (2004) studied nonlinear static and dynamic analysis performance and equivalent damping for five and ten storeys BRB braced structures using El Centro and Mexico earthquake records. A design approach compatible with a target drift in the framework of the capacity spectrum method was followed bilinear stress-strain behaviors with post-yield stiffness of 0.05, was assumed for the members. After performing nonlinear static and dynamic analysis by DRAIN2D+ structural analysis program, the result of the analysis showed that using BRB will improve the structural performance of the structure. Moreover, it was observed that increasing stiffness of BRBs lead to increase the equivalent damping ratios of single degree of freedom frame significantly, and decrease the maximum displacements of structures.

Shawn and Uang (2006) investigated the possible effect of using BRB on decreasing residual strain in a dual system. Nonlinear static and time-history analysis for three and six storeys frames with different BRB configurations were designed as either a building frame or dual system performed. The results demonstrated that by using the dual system, residual storey drifts decrease by about 10% to 12%.

Kumar et al. (2007) used SAP2000 to conduct nonlinear time history analysis for single-bay-ten-storey frame braced with BRBs, under El Centro earthquake effect. The analysis results indicated that using buckling restrained bracings decrease roof drift, interstorey drift and increase base and top shear of the structures.

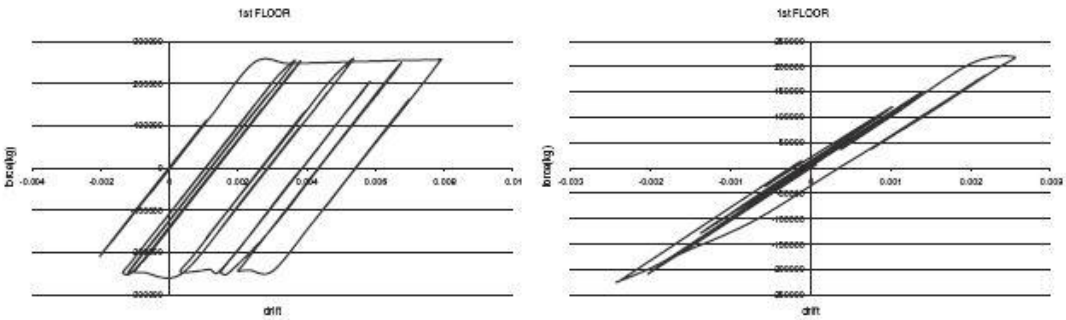
Asgharian and Amerhesari (2008) applied nonlinear time history analysis on buildings of three spans in each direction with span of 5m, having four, six, eight and ten storeys and different bracing configurations, such as: Split , V, inverted V and compression types were used. The buildings designed according to Iranian earthquake resistance design code (BHRC, 1999) for insufficient lateral load resistance. After applying a strong earthquake to the buildings and performing nonlinear time history. The results indicated that BRBs improve the seismic performance of the frames significantly and lead to a uniform response along the height of the frames. BRBs are showed more stable behavior compared to unbraced frames as shown in Figure 2.12.

Di Sarno and Elnashai (2009) conduct nonlinear time history for evaluation of nine storey frames retrofitted with three different types of bracing such as: concentric, BRB and mega braces. The frame previously designed with weak performance and not satisfies code drift limits in high seismic zones. Among the braced frames mega braced shows best performance and decrease the peak storey drift by 70% with respect to unbraced frames.



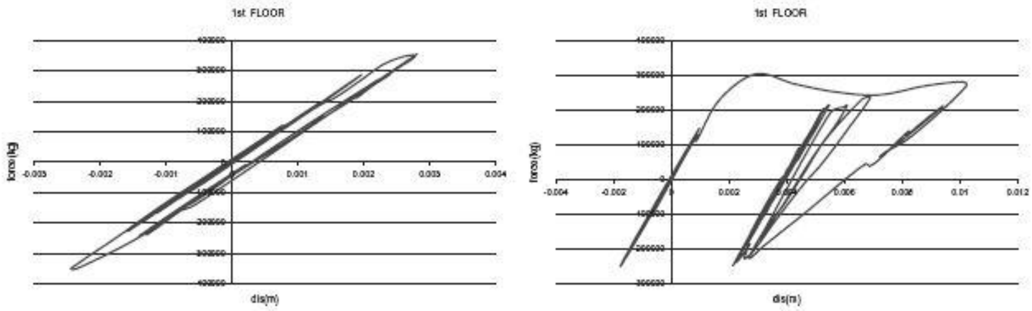
A B

(a)



A B

(b)



A B

(c)

Figure 2.12 Hysteresis behavior of a) 4, b) 8 and c) 10 storey frames as A-buckling restrained braced frames and B- ordinary frames

Asgarian and Shokrgozar (2009) studied twenty four moment resisting frames retrofitted with BRBs in three configurations as diagonal, cross, inverted V by using Open-Sees program. From the analysis results, they observed that over-strength and ductility factor of the frames affected by type of the bracing and height of the frames. The results showed that cross and chevron types of bracing showed best effect on increasing over-strength and ductility of the frames.

Deulkar et al. (2010) used SAP2000 program to model, analysis and design of two dimension five storey steel frames retrofitted with various types of BRB (different length and area of BRB core) as shown in Figure 2.13. The earthquake records used was El Centro with a suitable scale. It is found that length and area of metal core of BRBs effect on type of the braces. Also it is showed that adding BRBs improve significantly fundamental period, peak drift, interstorey drift and base shear of the frames as shown in Figure 2.14.

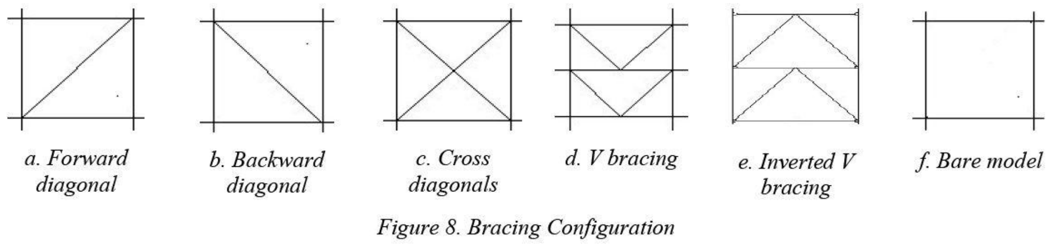
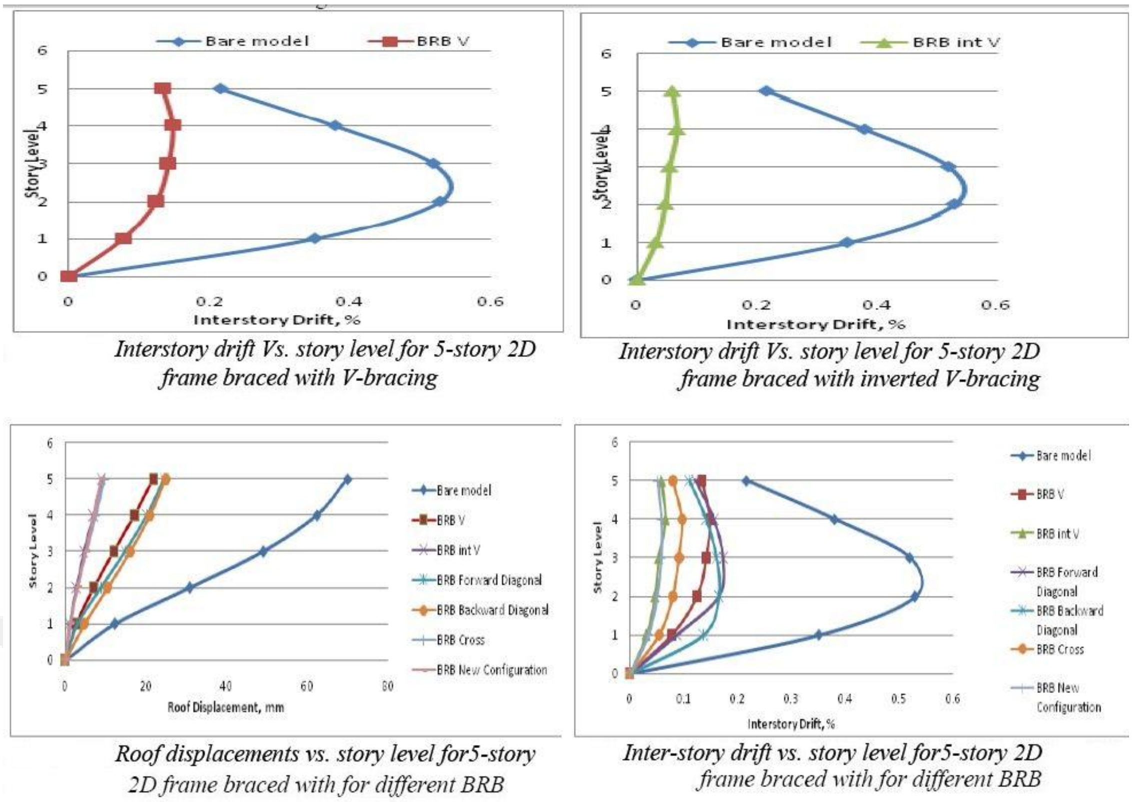


Figure 2.13 Types of BRB configurations (Deulkar et al., 2010)



Interstory drift Vs. story level for 5-story 2D frame braced with V-bracing

Interstory drift Vs. story level for 5-story 2D frame braced with inverted V-bracing

Roof displacements vs. story level for5-story 2D frame braced with for different BRB

Inter-story drift vs. story level for5-story 2D frame braced with for different BRB

Figure 2.14 Roof displacement and interstorey drift for frames braced with different BRB configurations (Deulkar et al., 2010)

Youssef et al. (2010) focused on assessing effect of BRBs and super elastic Shape Memory Alloy Braces (SMABs) on the seismic performance of three-storey RC building. Two RC frames are designed using both BRBs and SMABs. Pushover and nonlinear time history analyses were conducted using the Seismo Structural computer program (Seismo Soft , 2007) for evaluating SMABFs and BRBFs. The SMAB system is found to reduce seismic residual deformations considerably. However, this advantage is lost at peak ground accelerations close to the peak ground acceleration causing failure of the frame. The results of the static analysis also showed that at early stages of static pushover loading, the SMABFs exhibited better response than the BRBFs. At higher deformations, the response of the BRBFs was better as shown in Figure 2.15.

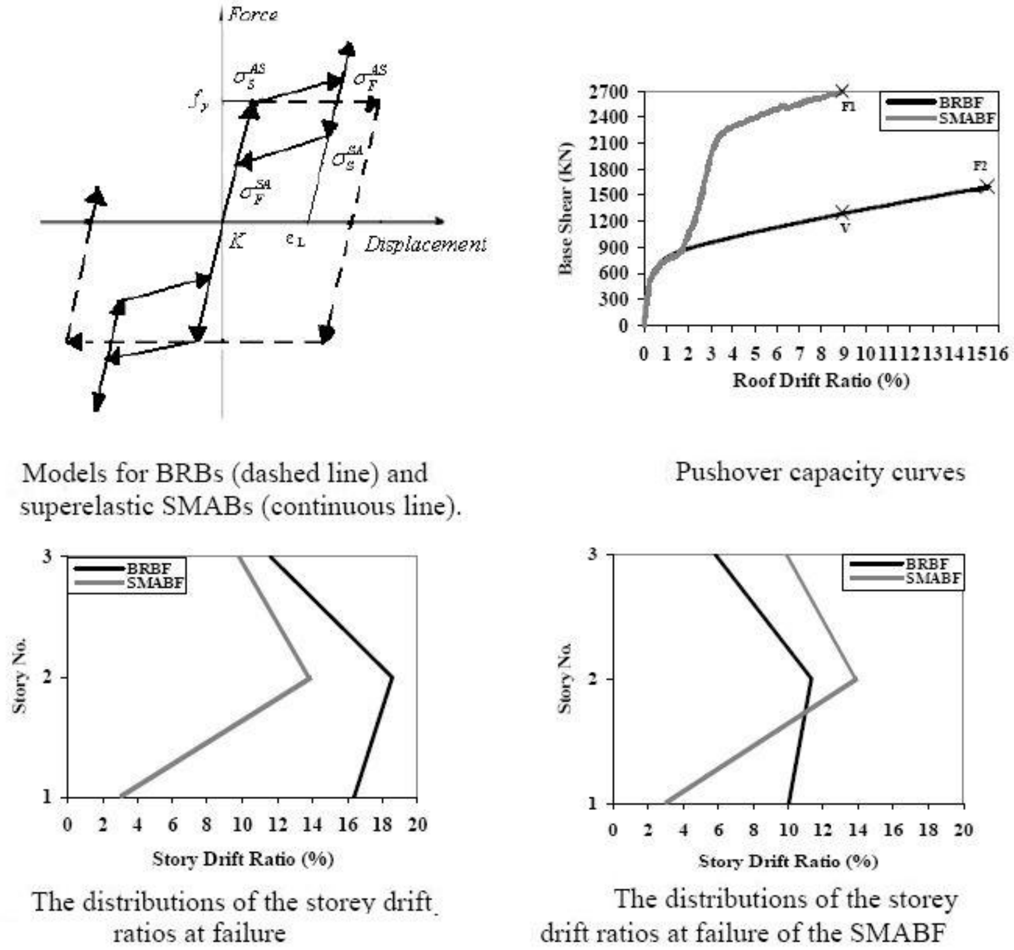


Figure 2.15 Results of static and nonlinear time history analyses (Youssef et al., 2010)

Di Sarno and Manfredi (2010) prepared a study about retrofitting of RC two storey buildings designed previously for gravity loads. The RC buildings were too weak to resist earthquake loads. Different configurations of BRBs were used to retrofit the frames. Seven natural earthquake records were used for the time history analysis. Analysis results of pushover and nonlinear time history indicated that seismic performance notably improved by retrofitting the frames, for the braced frames, under moderate-to-high magnitude earthquakes. The average period elongation is about 30% for the braced frame, while for the existing building the elongation is negligible (lower than5%). BRBs dissipate about 60% of input seismic at ultimate limit states of the loadings.

Güneyisi (2011) investigated retrofitting of steel MRFs by CBs and BRBs through fragility analysis. A large number of natural earthquake ground motion records were used in the fragility analysis. Nonlinear time history analysis applied for all the frames and by comparing fragility curves of the cases it was observed that inserting CB and BRB to the structures improve seismic performance of the frames, but BRBs have better affect.

Pin connected BRBs with collars has been studied by Zhao et al. (2012) to find effect of end rotation of the BRBs on stability of the braces and retrofitted frames. Different behaviors of the BRBs mechanism (end rotation, space between core and cover effect and collar-case gap effect) in addition to the specification of the cases had taken into consideration as shown in Figure 2.16. The results showed that due to rotation of ends of the braces a moment will induce in the case of the BRB. Finally a simple approach proposed to estimate the end moment of the braces. It was also seen that if this bending moment exceeded the yield moment of the casing, the specimens failed by global buckling, indicating a significant bad effects on global stability of BRBs.

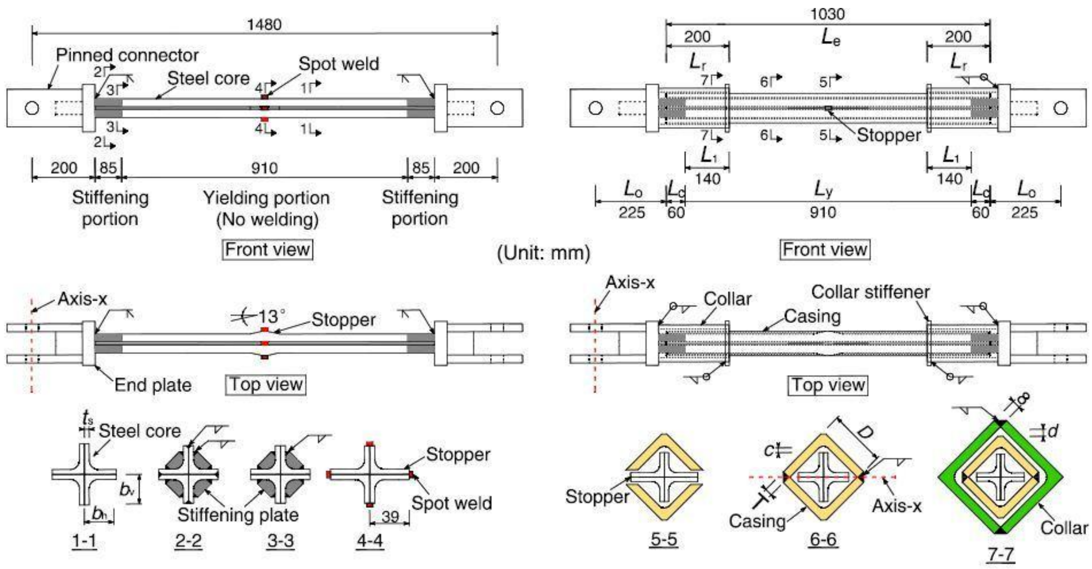


Figure 2.16 Configurations of BRB (Zhao et al., 2012)

Kumbhar and Shiyekar (2014) compared lateral load carrying capacity of frame models for bare frames, CBFs and BRBFs. The models consist of single bay two storey steel frame with storey height and bay width equal to 800 mm. The test results showed that by keeping the cross sectional area of brace equal to that of conventional brace and just putting it in a sleeve we can make it as buckling restrained brace. It is found that lateral deflection of framed braced by buckling restrained braces is less than CBF, BRB is more effective in resisting the lateral deflection and in carrying lateral load.

Analytical five models for connections of zigzag configured BRBs with RC frames were proposed by Qu et al. (2015) as shown Figure 2.17. Nonlinear time history analysis with different earthquake records applied for the cases. The results showed that the flexibility of RC corbels increased the interstorey drift of the whole building; the elastic strain of post-tensioned bolts has little effect on the global responses of the buildings.

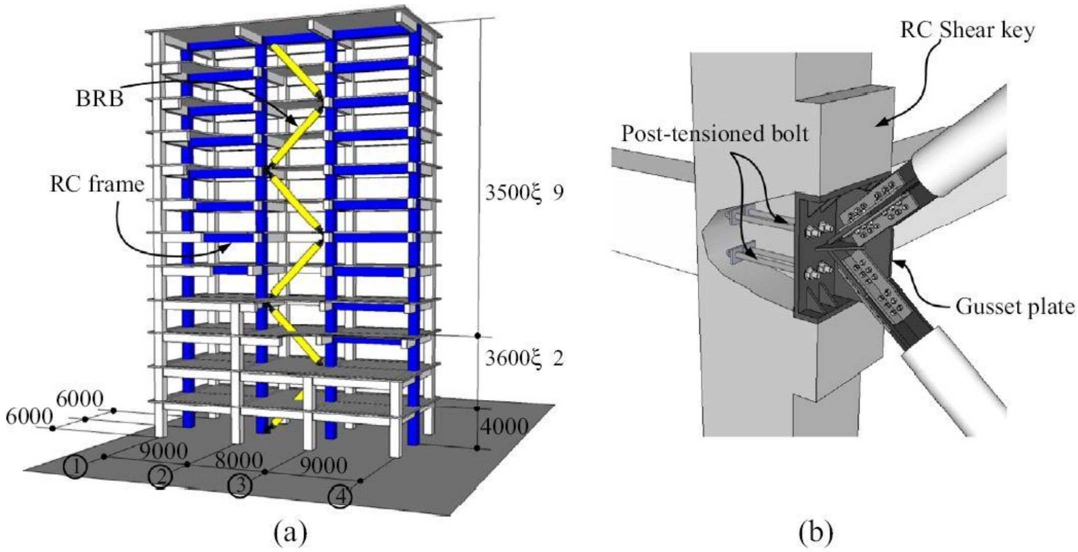


Figure 2.17 Zigzag buckling restrained braced RC frames: (a) BRB configuration and (b) details of the BRB connection (Qu et al., 2015)

2.4.2 Viscoelastic Damper (VD)

Another innovative technique used to improve energy dissipation capability of buildings is the use of viscoelastic damper which is generally manufactured from a highly dissipative polymeric material. This mechanism employed for the first time by Gerb (1978) for energy dissipation purpose under machine foundation (Tezcan and Uluca, 2003). VDs contain viscoelastic materials with certain viscosity specifications. The most commonly used materials are acrylic copolymers, were used as the energy-absorbing components in dampers, they are normally used in the form of shear layers and the exposed surface area is very small relative to the volume of material as shown in Figure 2.18 (Langenbach and Kelley, 1991). In this regard, viscoelastic dampers are known to be used in new buildings to alleviate the harmful effect of the earthquakes (Nagarajaiah and Narasimhan, 2007), in bridges (Madhekar and Jangid 2009), and also in retrofitting of existing structures (Malhorta et al., 2004, Potty and Nambissan, 2008). Dampers have proven to be effective systems for reducing earthquake forces in structures (Chandra et al., 2000). Moreover, the performance of dampers near collapse during large seismic events has proven feasible by analytical and experimental investigation (Armouti, 2013).

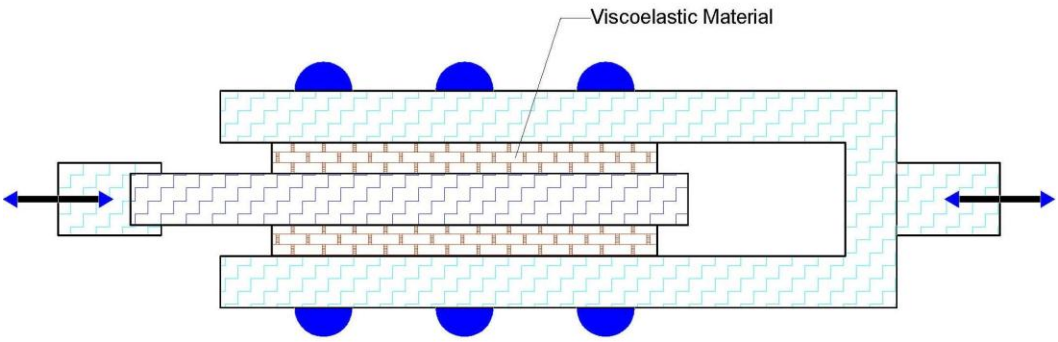
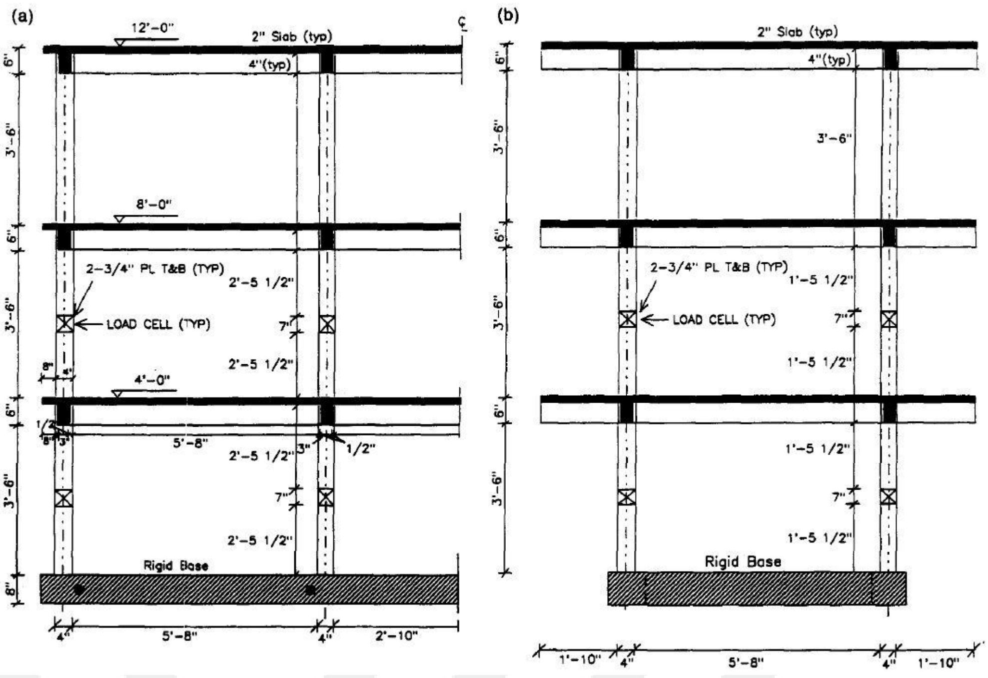


Figure 2.18 Schematic configuration of viscoelastic damper (Langenbach and Kelley, 1991)

Shen et al. (1995) presented that viscoelastic dampers are very good devices for improving seismic response and inelastic deformation of RC structures. A three-bay by one-bay RC frame retrofitted by two types of viscoelastic dampers as shown in Figure 2.19 were used as a case study. The frames has been tested in the laboratory under simulated strong earthquake ground motions.



(a) (b)

Figure 2.19 Studied bare frame: (a) front view; (b) side view (Shen et al., 1995)

Munshi and Kasai (1995) discussed seismic retrofitting of 10 storey weak steel MRFs by viscoelastic dampers as shown in Figure 2.20. The Performance of the frame with and without dampers is investigated under four different earthquake excitations. The frames designed by Anderson and Bertero (1969), satisfying the code minimum strength requirement but intentionally disregarding the code drift limit. The viscoelastic damper behavior is modeled by a nonlinear element which takes into account the frequency and temperature dependency of the damper's material. The results indicated that viscoelastic dampers significantly reduce stress and deformation demand on the members and connections and enhance the frame performance under strong earthquakes. The amount of stiffness and damping needed can be reasonably estimated based on drift control strategy proposed herein. A simplified linear approach is proposed as a practical analytical method for VE-damped frames.

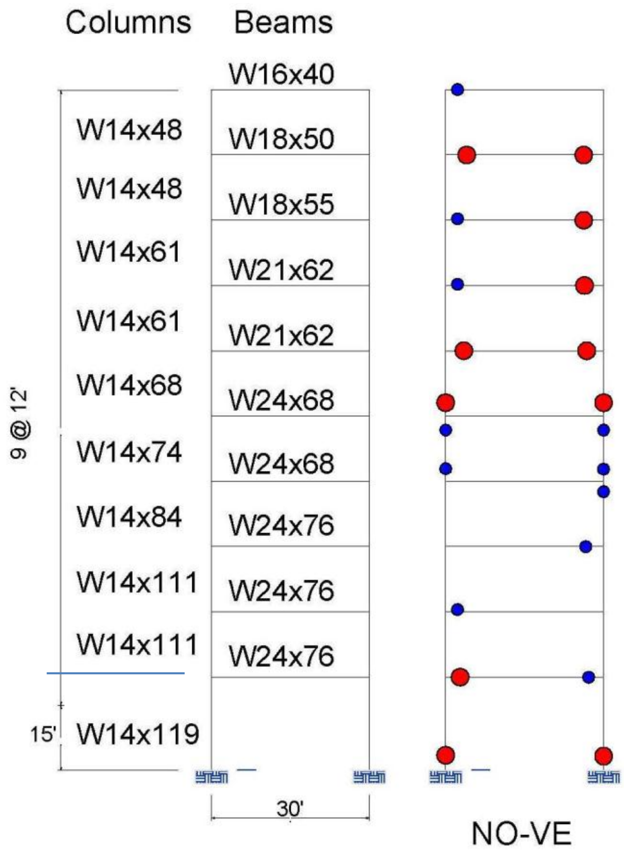


Figure 2.20 Unbraced MRF model (Munshi and Kasai, 1995)

Munshi (1997) carried out a parametric study about effect of viscoelastic dampers on hysteretic response, ductility and energy dissipation of weak RC elements as shown in Figure 2.21. DRAIN-2DX program used to simulate behavior, degradation and pinching characteristics of RC frames and VD characteristics. After applying nonlinear dynamic analysis for the bare and retrofitted elements, it was observed that inserting VDs in the bare frame will reduce the curvature at the plastic hinges, stiffness degradation, strength decay and finally pinching on the RC elements' hysteresis as shown in Figure 2.22.

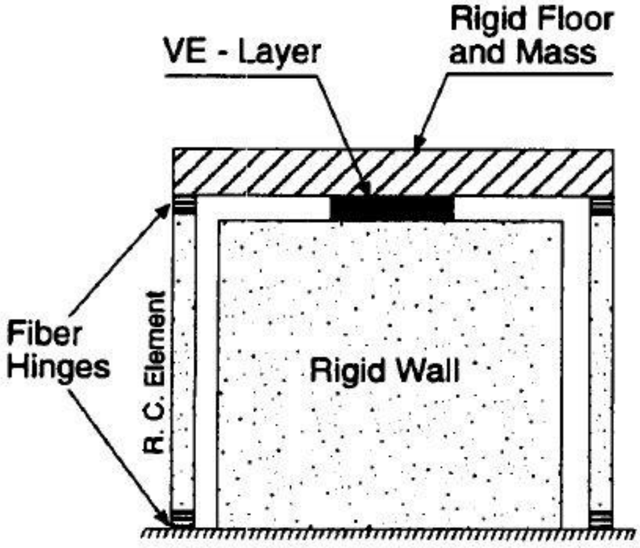


Figure 2.21 RC frame without brace (Munshi , 1997)

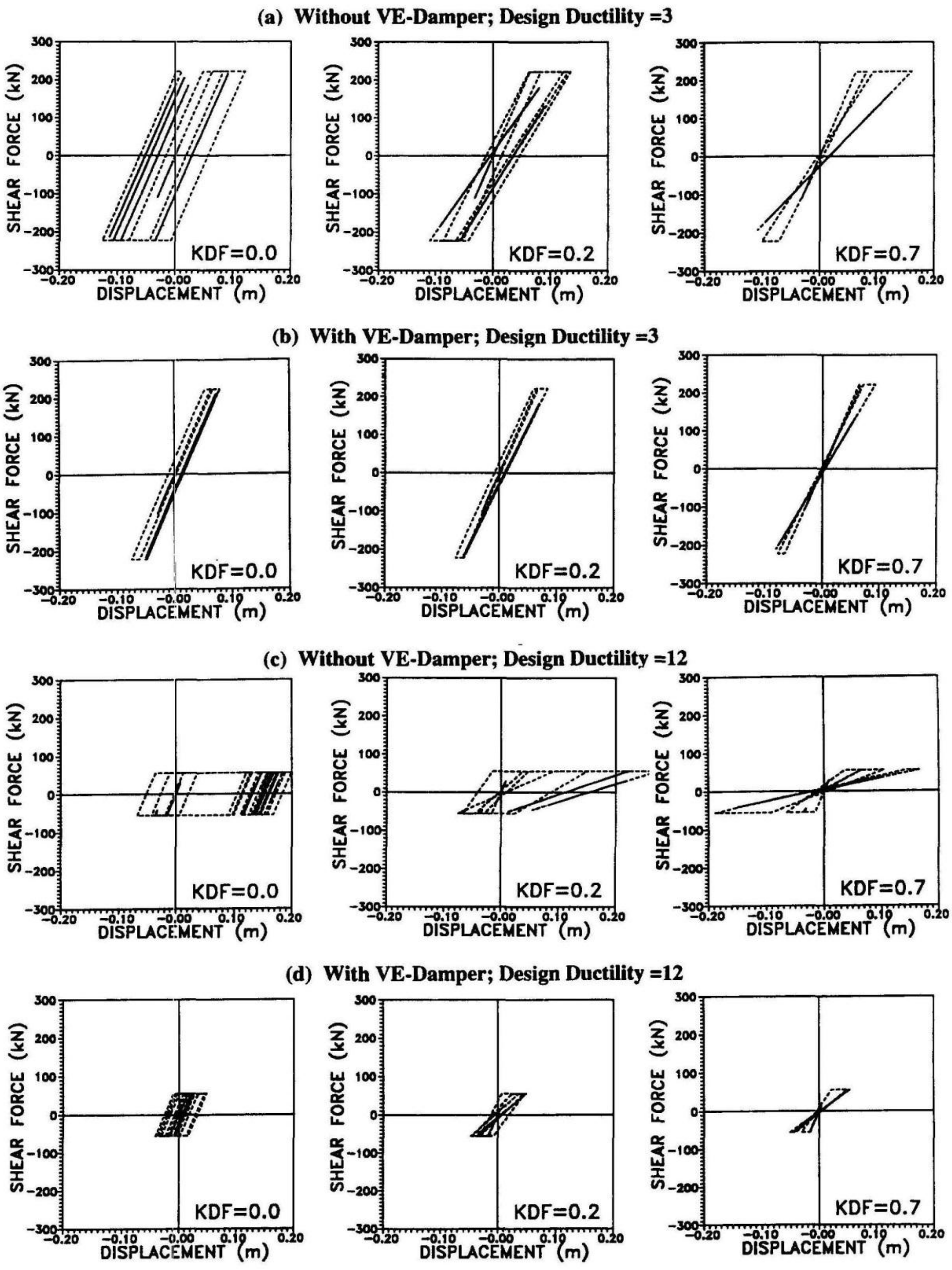
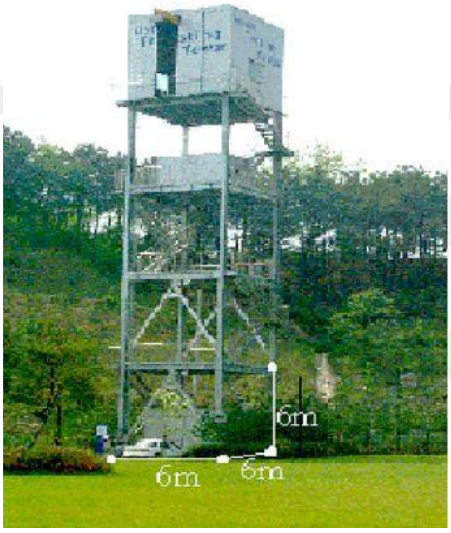


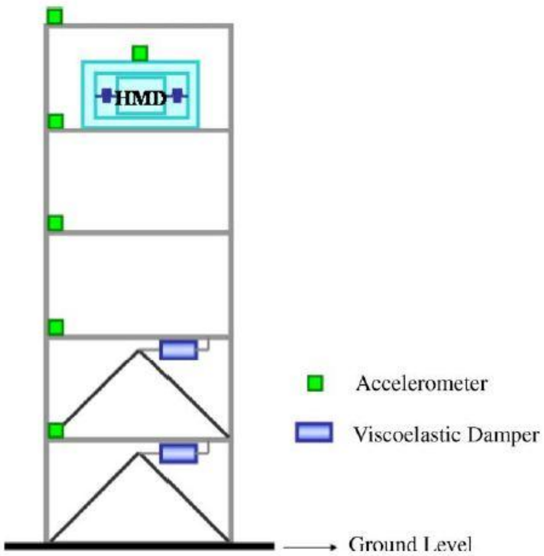
Figure 2.22 Effect of stiffness degradation on energy dissipation of the system

(Munshi , 1997)

A full scale steel structure of one bay and 5 storey was studied as a full-scale model structure vibrated using the hybrid mass driver located on the top floor as shown in Figure 2.23 (Min et al., 2004). Using modal strain energy approach suitable size of dampers configured. The test results indicated that number of VD and damping ration improve the performance of the structure and reduce the acceleration response. The dampers effect was the best at the temperature from 24 C°.



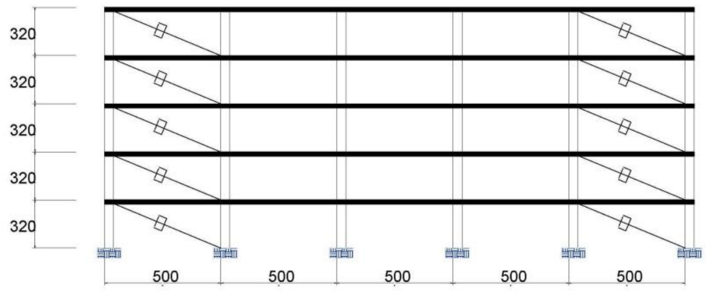
(a)



(b)

Figure 2.23 a) Full-scale model of the structure and b) schematic view of locations of VED and accelerometers (Min et al., 2004)

Mortezaei and Zahrai (2009) investigated viscoelastic dampers as energy-absorbing devices in buildings under near-field earthquakes using Sap2000 computer program. Three cases 5-storey, 10-storey and 15-storey RC buildings as shown in Figure 2.24. Roof drift, interstorey drift and base shear results of the analysis showed that applying VD in the bare frame will improve considerably the earthquake response of the structure as shown in Figure 2.25.



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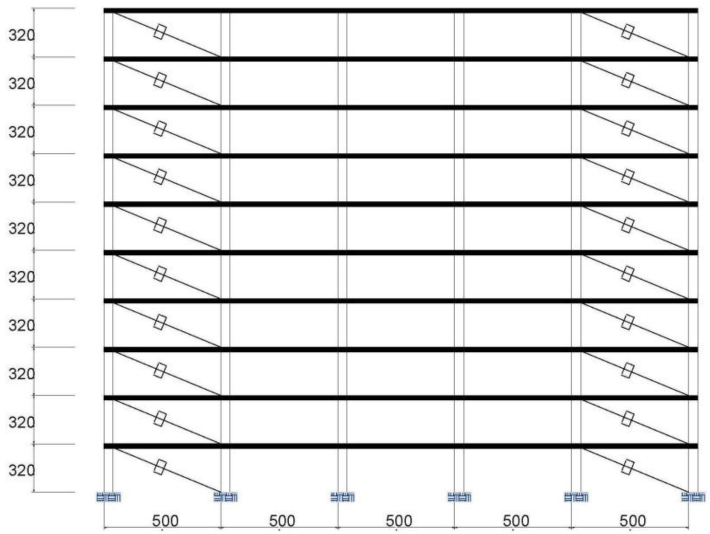

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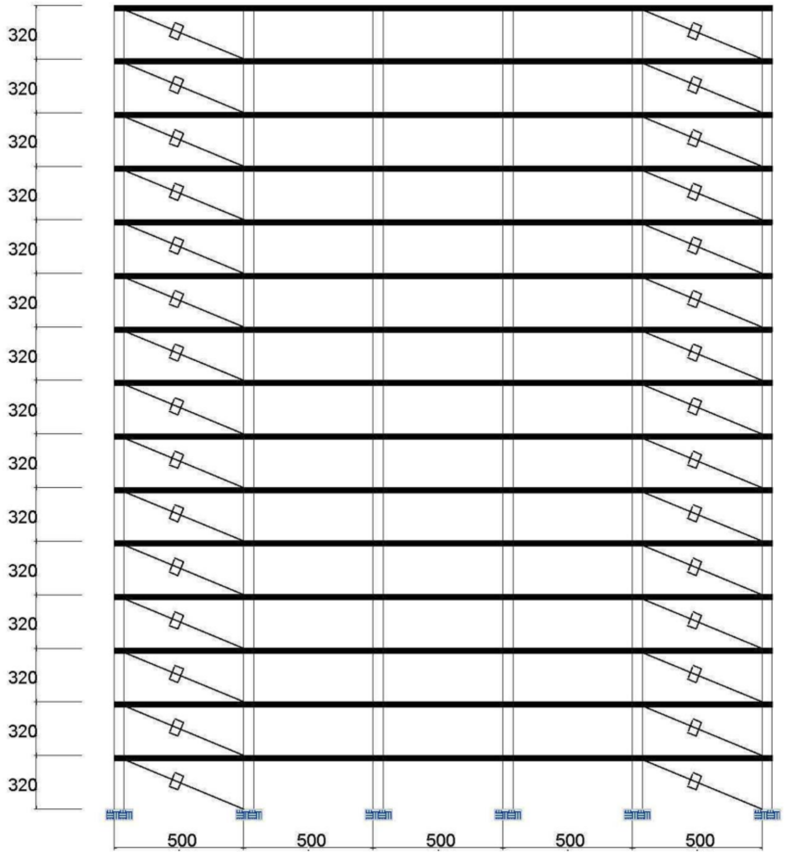
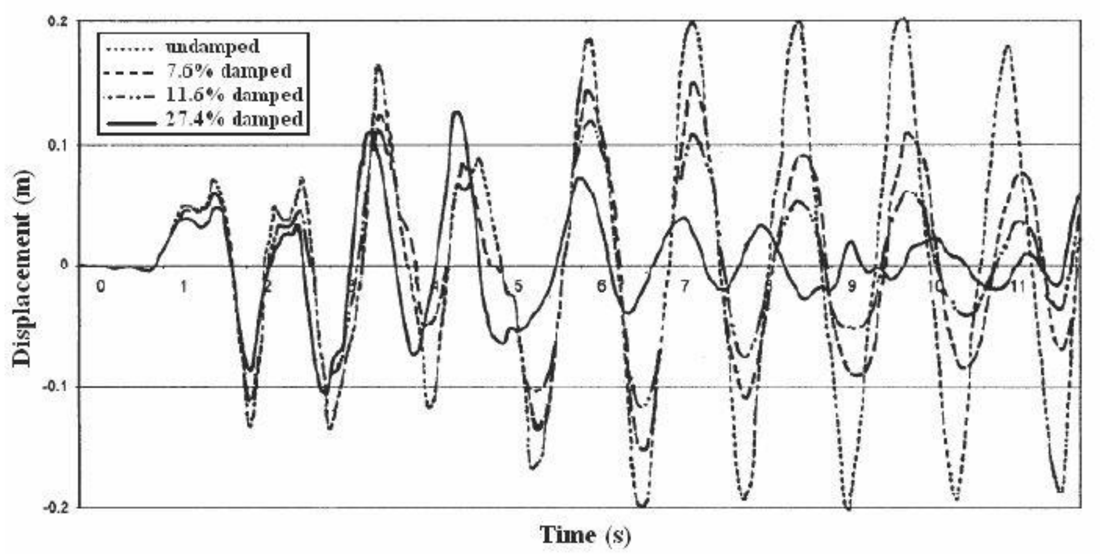
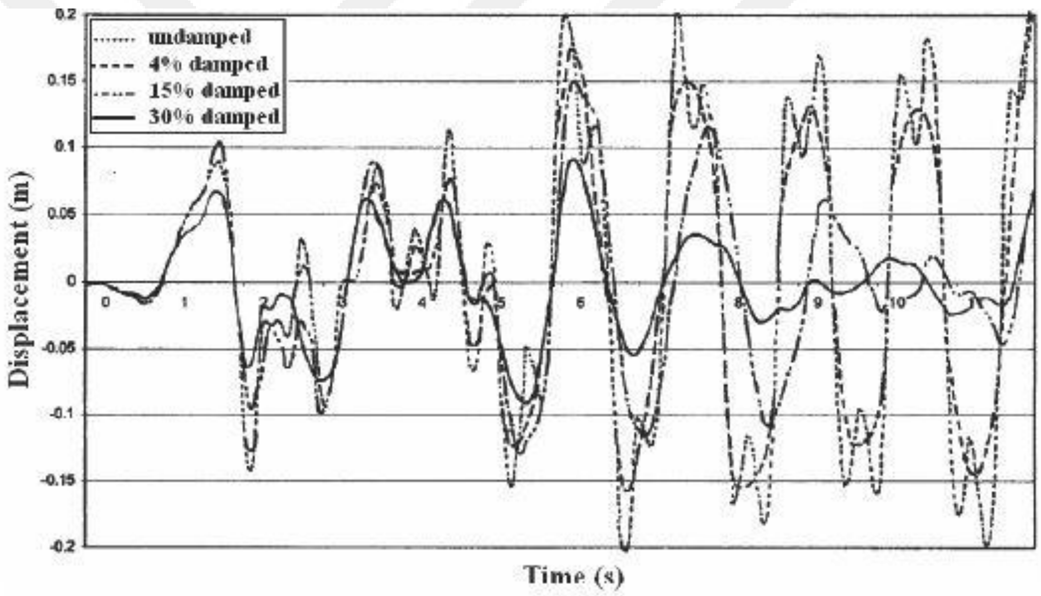


(c)

Figure 2.24 Elevation and braces of the cases: a) 5 storey, b) 10 storey and c)15 Storey RC-MRFs (Mortezaei and Zahrai, 2009)



(a)



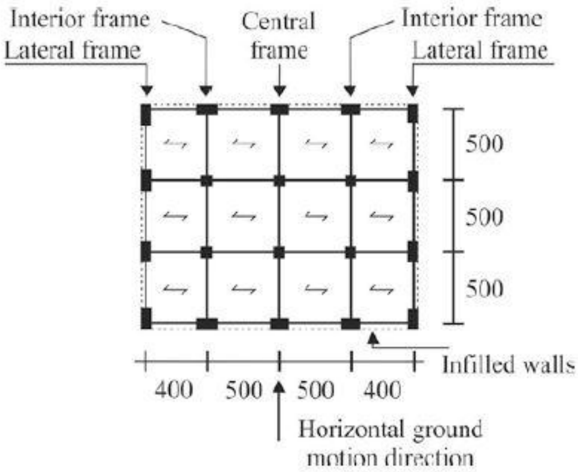
(b)

Figure 2.25 Top story relative horizontal displacements a) 5 storey and b) 10 storey buildings (Mortezaei and Zahrai, 2009)

Nakamura et al. (2011) proposed a practical nonlinear response analysis method for viscoelastic dampers that takes into consideration the temperature, frequency and amplitude dependencies. Analyses of random wave response experiments were made, and it was confirmed that the level of accuracy of the proposed model was roughly equivalent to that achievable with high-accuracy models. The proposed model is an effective model that provides a sufficient level of accuracy to qualify as a practical nonlinear response analysis method.

Zandi et al. (2012) studied effect of passive viscoelastic dampers on reducing seismic vibration of structures. Three models of RC moment resisting frame of 9, 14, 17 storey frames with a variable number of openings were prepared by Open-Sees software. Nonlinear time-history analysis were conducted to evaluate seismic characteristics of the frames using eight different accelerations records and in type C soils. The analysis results indicated that increasing damping ratio by 25% will reduce the base shear by about 25% and leads to a constant reduce in the maximum storey displacement of the roof.

Friction, metallic yielding and viscoelastic and viscos dampers has been studied by Mazza and Vulcano (2013). For this purpose a residential RC building of 6-storey designed previously for low seismic zone has been studied. The view of the unbraced and braced buildings and also the configurations for damper bracings were given in Figures 2.26 and 2.27. An empirical design procedure was proposed for distributing dampers in order to reach required level of seismic intensity according to stiffness and strength of the dampers. The proposed design procedure showed a very good reliability.



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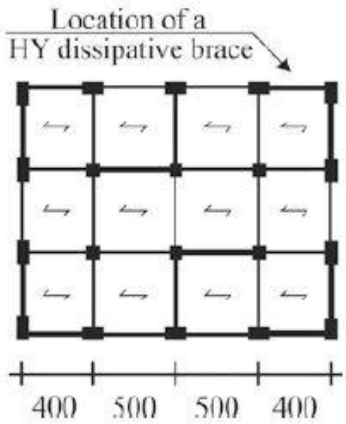


(b)

Figure 2.26 Floor plan of six storey a) unbraced framed structure (UF) and b) braced structure (DBF) (Mazza and Vulcano, 2013)

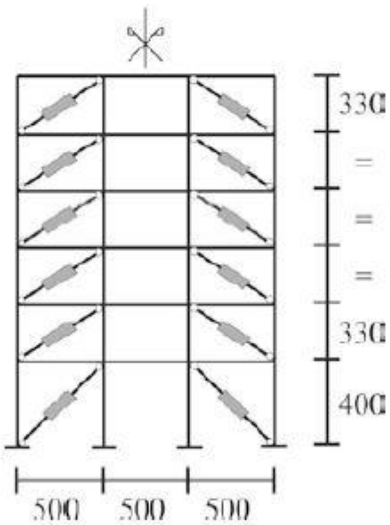


Figure 2.27 Bracing configuration of six storey building (Mazza and Vulcano, 2013)

CHAPTER 3

METHODOLOGY

3.1 Description of the Analytical Models

3.1.1 Steel Moment Resisting Frame

The steel structural models selected for this part of the thesis were 3, 6 and 12 storey steel moment resisting frames (SMRFs). The SMRFs were designed according to UBC (1997) with lateral stiffness insufficient to satisfy code drift limitations in zones with high seismic hazard. They have the same floor plan (4x4 bays) with 5 m bay spacing, whereas the height is 4.2 m at the ground storey and 3.2 m at the remaining floors. The buildings' height are 10.6m, 20.2m, and 39.4m for 3, 6, 12 storey structures, respectively. Typical plan and elevation of the SMRFs are given in Figures 3.1 and 3.2, respectively.

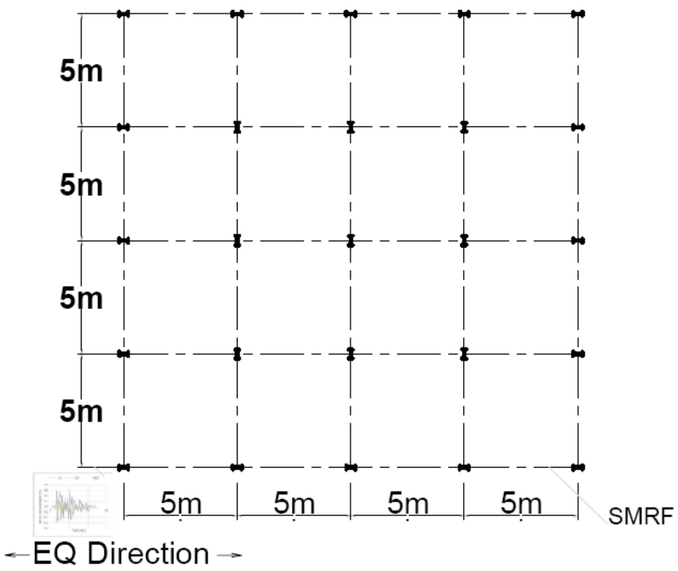


Figure 3.1 Plan view of SMRFs

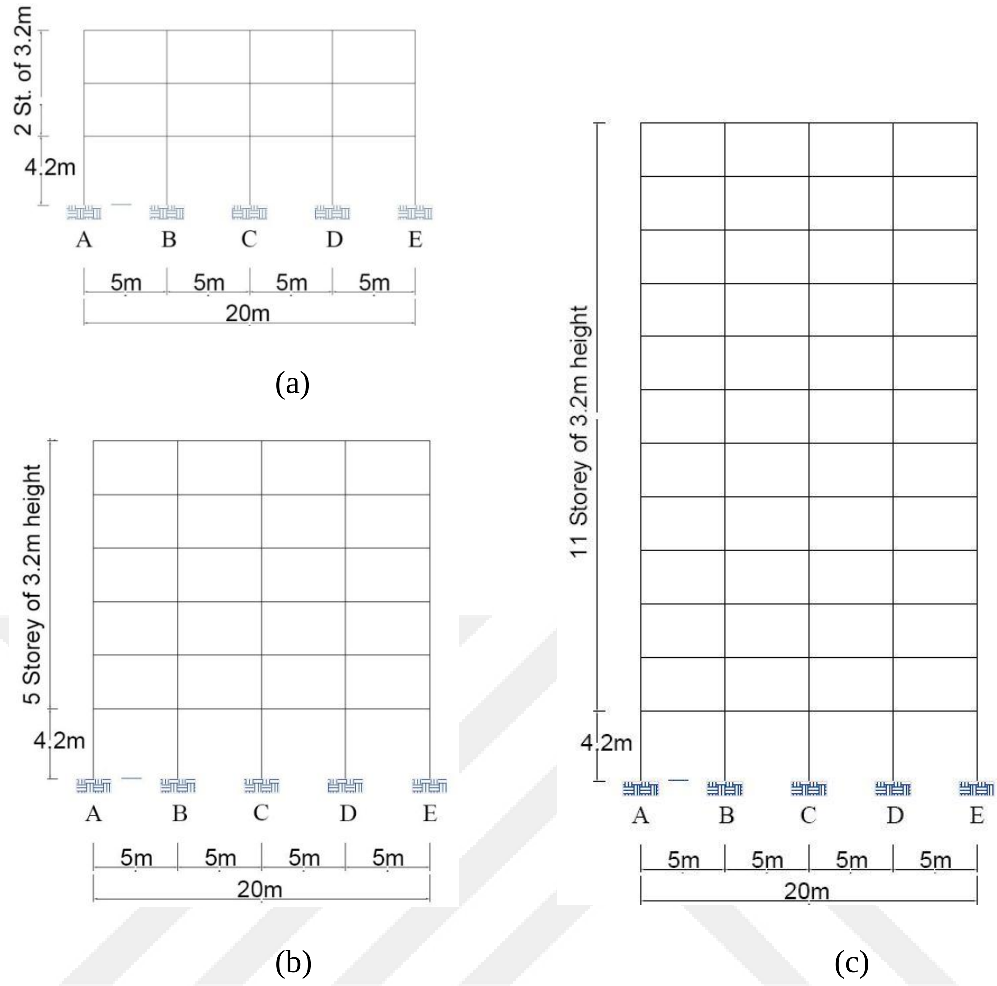


Figure 3.2 Elevation views of a) 3-storey, b) 6-storey and c) 12-storey SMRFs

Section profiles for the unbraced frames (UBFs) are given in Tables 3.1, 3.2 and 3.3. The design dead load and the live load for the storey slabs were taken as 4.2 kN/m^2 and 2.0 kN/m^2; whereas for the roof slab they were taken as 3.6 kN/m^2 and 0.5 kN/m^2, respectively. The frames were assumed to have uniform mass distributions. Steel having nominal yield strength equal to 345 MPa and modulus of elasticity 200 GPa were used for the structural steel members.

For upgrading the seismic behavior, three types of the retrofitting approaches including conventional braces (CBs), buckling-restrained braces (BRBs), and viscoelastic dampers (VDs) were considered by inserting such devices diagonally in two middle bays of unbraced frame (UBF) as seen in Figures 3.2 and 3.3.

Table 3.1 Characteristics of the 3-storey SMRF

Storey	Columns				
	Axis				
	A	B	C	D	E
1st	IPE500V	IPE500V	IPE500V	IPE500V	IPE500V
2nd	IPE450	IPE450	IPE450	IPE450	IPE450
3th	IPE400	IPE400	IPE400	IPE400	IPE400

Table 3.2 Characteristics of the 6-storey SMRF

Storey	Columns				
	Axis				
	A	B	C	D	E
1st	IPE750x185	IPE750x210	IPE750x185	IPE750x210	IPE750x185
2nd	IPE750x185	IPE750x185	IPE750x185	IPE750x185	IPE750x185
3th	IPE750x185	IPE750x185	IPE750x185	IPE750x185	IPE750x185
4th	IPE500	IPE500	IPE500	IPE500	IPE500
5th	IPE500	IPE500	IPE500	IPE500	IPE500
6th	IPE500	IPE500	IPE500	IPE500	IPE500

Table 3.3 Characteristics of the 12-storey SMRF

Storey	Columns				
	Axis				
	A	B	C	D	E
1st	IPE750x196	IPE750x222	IPE750x222	IPE750x222	IPE750x196
2nd	IPE750x196	IPE750x222	IPE750x222	IPE750x222	IPE750x196
3th	500V	600V	550V	600V	500V
4th	500R	600V	550V	600V	500R
5th	500R	600R	550V	600R	500R
6th	400V	600R	400V	600R	400V
7th	400V	400V	400V	400V	400V
8th	400V	400V	400V	400V	400V
9th	330O	330O	330O	330O	330O
10th	330O	330O	330O	330O	330O
11th	330O	330O	330O	330O	330O
12th	330O	330O	330O	330O	330O

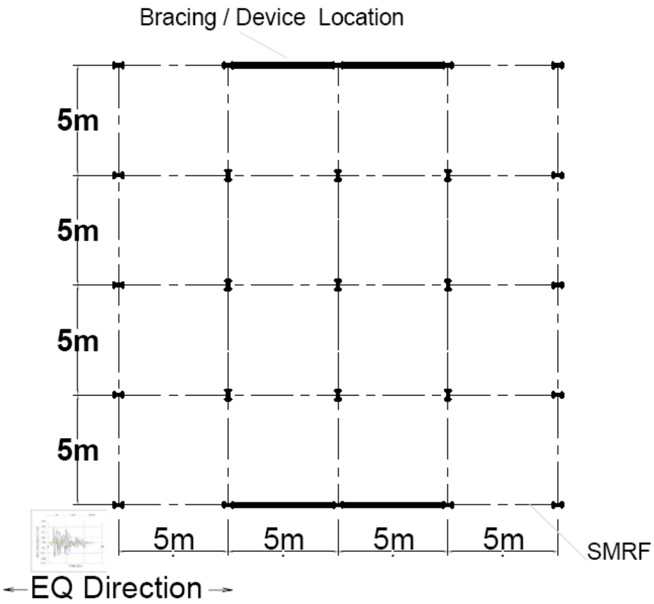


Figure 3.3 Location of bracing and energy dissipation devices in SMRFs

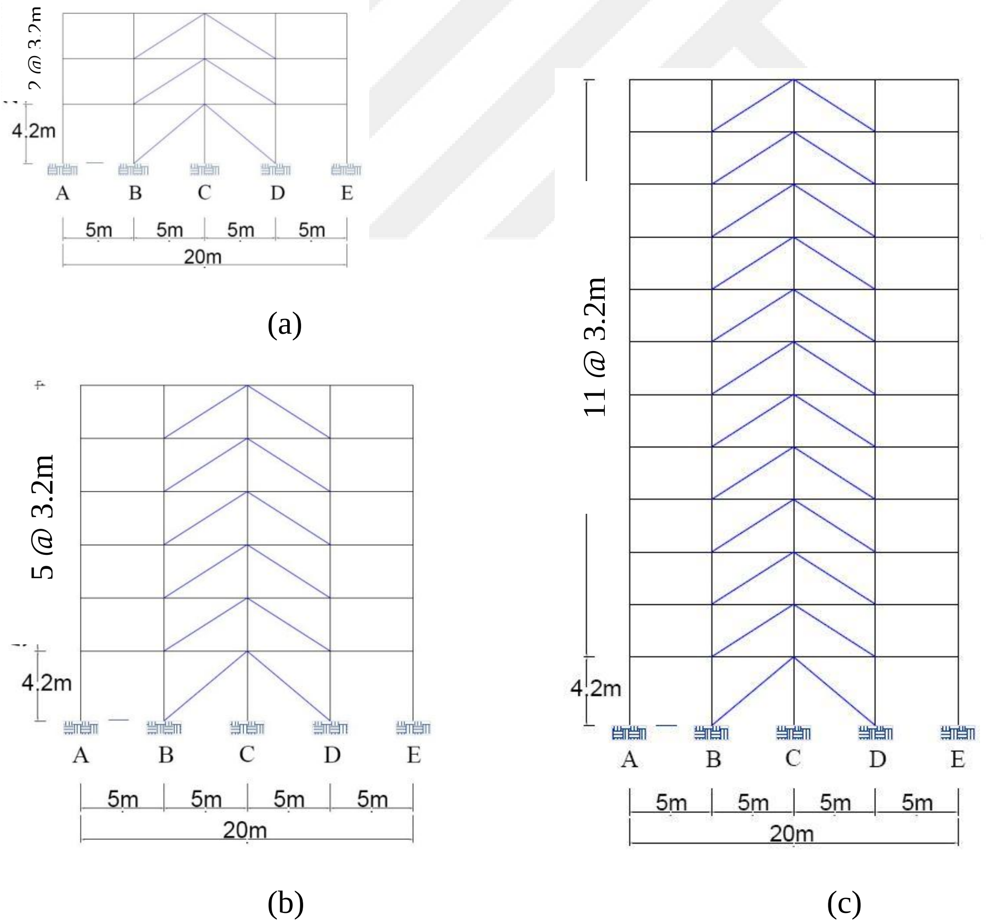


Figure 3.4 Elevation views for a) 3-storey, b) 6-storey and c) 12-storey retrofitted SMRF

For conventional braces, hollow square steel sections (HSS) of 6”x6”x3/16”, 8”x8”x3/16” and 10”x10”x3/16” were used in 3, 6 and 12 storey frames, respectively.

In order to compare the efficiency of conventional and buckling restrained braces, the BRBs were designed for the same capacity with the conventional braces in tension.

3.1.2 Reinforced Concrete Moment Resisting Frame

The structural models selected for this part of the thesis were 3, 6 and 12 storey reinforced concrete (RC-MRFs) frames. The frames have floor plan (4 x 4 bays) with 5 m bay spacing, whereas the height is 4.2 m at the ground storey and 3.2 m at the remaining floors as shown in Figure 3.5. The buildings’ height are 10.6m, 20.2m, and 39.4m for 3, 6, 12 storey structures, respectively as shown in Figure 3.6. The design dead load and the live load for the storey slabs were taken as 4.2 kN/m^2 and 2.0 kN/m^2; whereas for the roof slab they were taken as 3.6 kN/m^2 and 0.5 kN/m^2, respectively. Crushed compressive strength for concrete is 21 MPa and yield strength of the steel reinforcement is 415 MPa.

The reinforced concrete frames were designed according to ACI 318-08 for gravity with lateral stiffness insufficient to satisfy code drift and hinge limitations in zones with high seismic hazard (ACI 318-08; CSI 2009). All details of beams and columns are presented in Table 3.4.

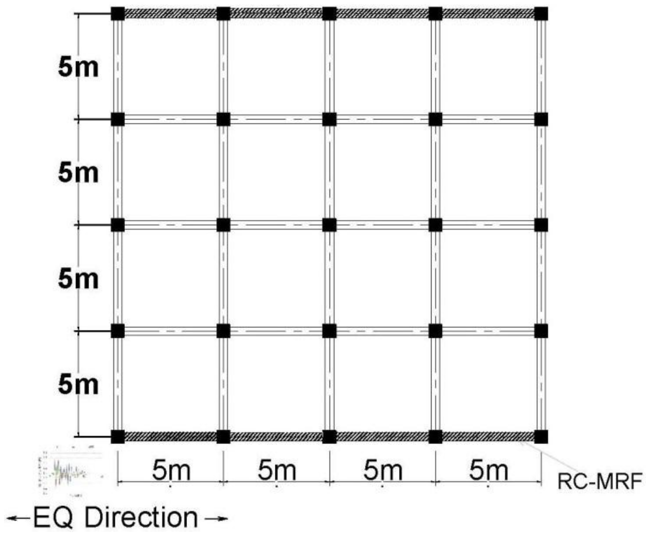


Figure 3.5 Plan view of RC-MRFs

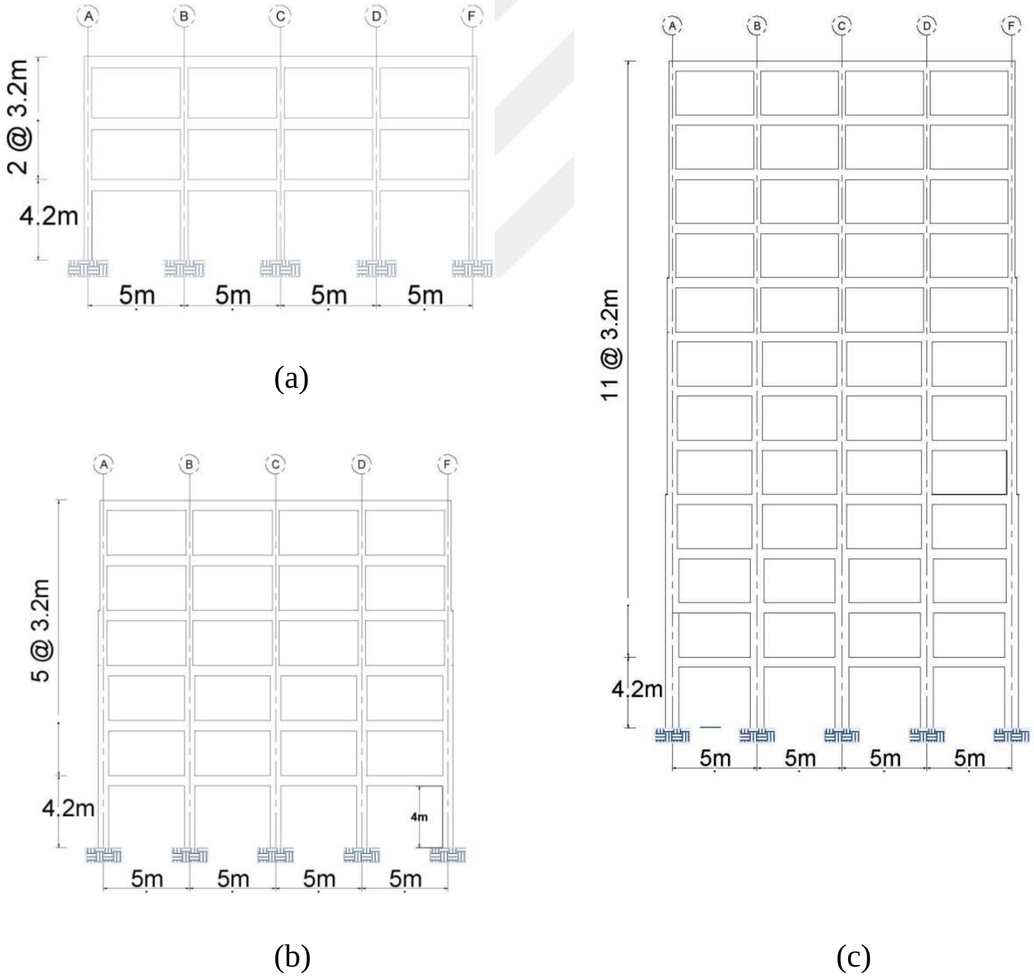


Figure 3.6 Elevation views of a) 3-storey , b) 6-storey and c) 12-storey RC-MRFs

Table 3.4 Details of RC-MRFs

RC Frames	Storeys	Beams' size (m)	Columns' size (m)
3 Storey	1 to 3	0.60x0.45	0.45x0.45
6 Storey	1 to 3	0.60x0.45	0.55x0.55
	4 to 6	0.60x0.45	0.45x0.45
12 Storey	1 to 4	0.60x0.45	0.80x0.80
	5 to 8	0.60x0.45	0.60x0.60
	9 to 12	0.60x0.45	0.45x0.45

Three retrofitting approaches including conventional braces (CBs), buckling-restrained braces (BRBs), and viscoelastic dampers (VDs) were considered by inserting such devices diagonally in two middle bays of unbraced frame (UBF) for improving seismic capacity of the frames as seen in Figures 3.7 and 3.8.

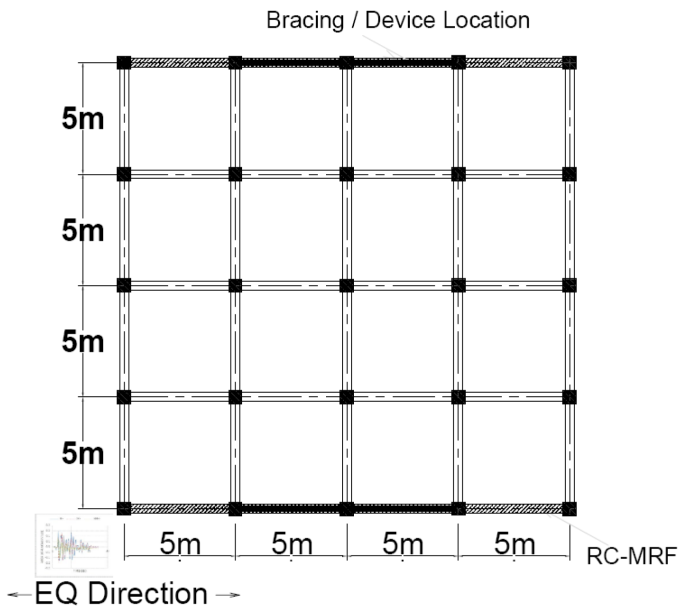


Figure 3.7 Location of bracing and energy dissipation devices in RC-MRFs

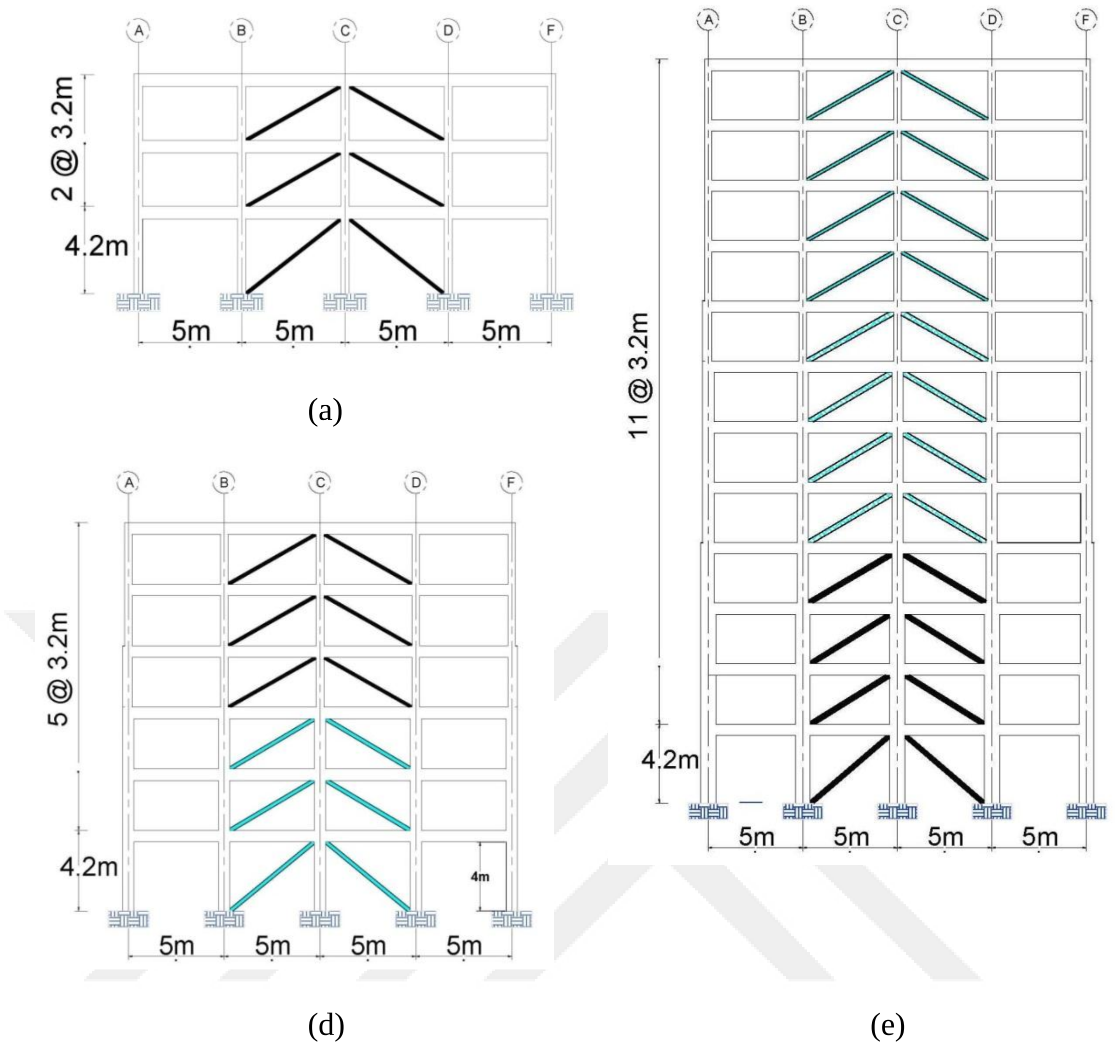


Figure 3.8 Elevation views for a) 3-storey, b) 6-storey and c) 12-storey retrofitted RC-MRFs

For conventional braces, hollow square steel sections (HSS) were used. Details of braces for all models are presented in Table 3.5.

Table 3.5 Characteristics of HSS Braces

RC Frames	Storeys	Steel Braces	Cross section area mm^2	Inertia (mm^4x10^6)
3-Storey	1 to 3	HSS-8”X8”X4/16”	4897	31.26
6-Storey	1 to 3	HSS-10”X10”X3/16”	4690	48.28
	4 to 6	HSS-8”X8”X3/16”	3723	24.22
12-Storey	1 to 4	HSS-10”X10”X5/16”	7677	76.17
	5 to 8	HSS-9”X9”X3/16”	4206	34.88
	9 to 12	HSS-8”X8”X3/16”	3723	24.22

In order to compare the efficiency of conventional and buckling restrained braces, the BRBs were designed for the same capacity with the conventional braces in tension.

3.2 Analytical Modeling

In this study two types of moment resisting frames were considered. The first one was steel moment resisting frame and the second type was reinforced concrete moment resisting frame. Steel MRFs and RC MRFs were designed for gravity loads according to the codes UBC 1997 and ACI 318-8, respectively, with an inadequate capacity to satisfy code drift and hinge limitations in high seismic area. For each type 3 storey, 6 storey and 12 storey frames studied under three different earthquake records (Hector Mine, Morgan Hill and Erzincan) as shown in Figure 3.9.

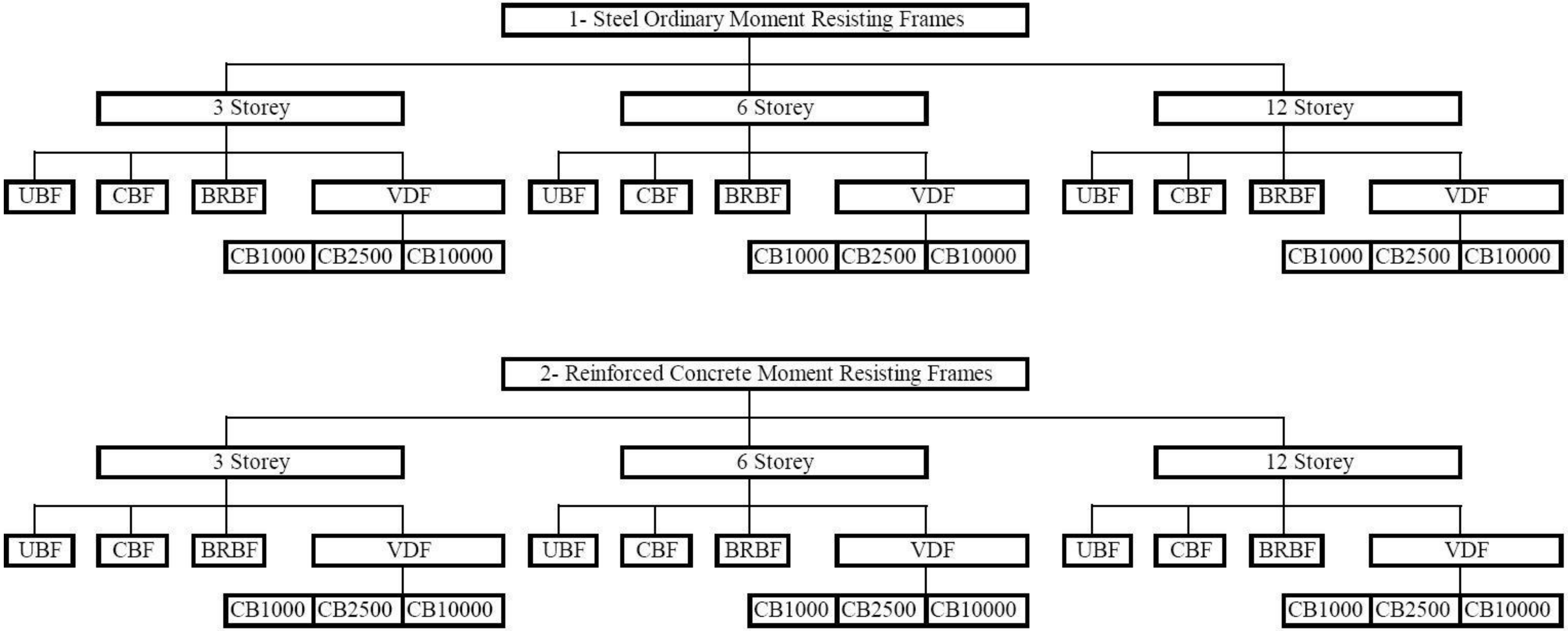


Figure 3.9 Tree diagram of the cases in this thesis

Nonlinear behavior of the structure is dependent on nonlinear response of the elements that are used in the lateral force resisting system. Therefore, before applying any nonlinear analysis method on a building structure, the nonlinear behavior of such elements must be clearly defined.

Load deformation behavior of structural members according to FEMA-356 (FEMA 356, 2000) is as shown in Figure 3.10. After the member yields (when applied load/yield load proportion (Q/Qy) is equal to 1, the subsequent strain hardening accommodates, as the member continues to deform toward the expected strength. The horizontal axis of this load deformation behavior may either express as curvature or strain (Θ or Δ).

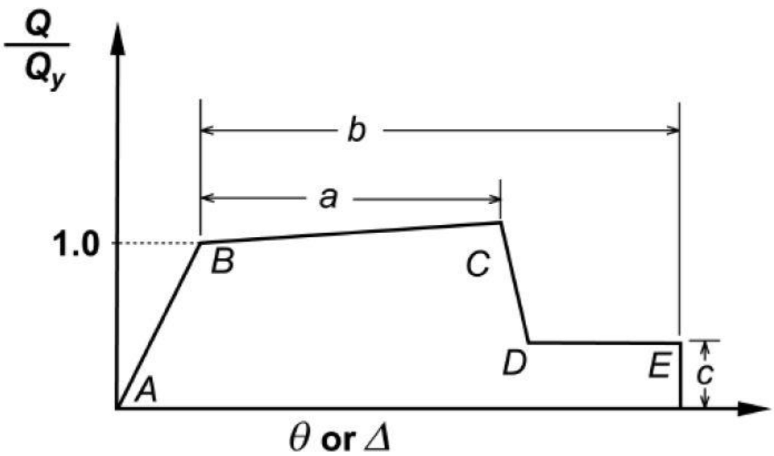


Figure 3.10 The generalized load deformation relation for a structural member (FEMA 356, 2000)

In this load deformation curve:

- Point-A represnts unloaded condition,
- Point-B represents yielding point situation,
- Point-C represents strain flow due to significant strength degradation,

- The drop from C to D represents the initial failure of the element,
- D to E Shows the residual resistance and allows the frame elements to bear gravity loads. Beyond point E, the maximum deformation capacity is reached and gravity load can no longer be sustained.

FEMA-356 code defines the acceptance criteria depending on the plastic hinge rotations by considering various performance levels. In Figure 3.11, the acceptance criteria on a force versus deformation diagram are given. In this diagram, the points marked as IO, LS and CP represent immediate occupancy, life safety and collapse prevention performance levels, respectively.

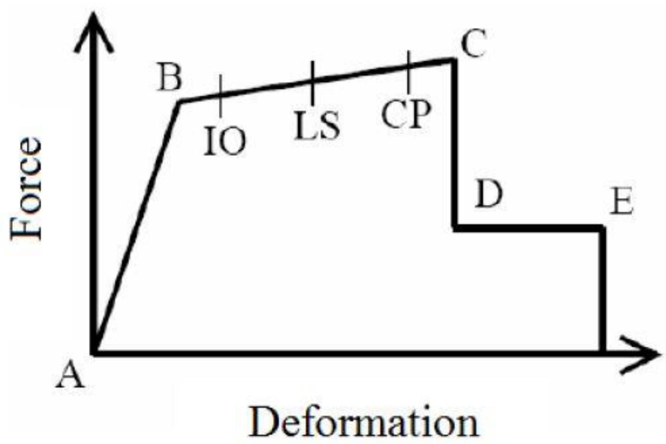


Figure 3.11 Acceptance criteria on a load deformation curve (FEMA 356, 2000)

In this study, axial force-moment interaction (PMM) according to FEMA356 (FEMA356, 2000) was defined for determining the nonlinear hinge properties of a columns, such hinges required that the axial force vs. Moment interaction diagram to be calculated. Moment hinge (M3) according to FEMA356 (FEMA356, 2000) was introduced for plastic hinges of the beam elements.

For nonlinear behavior of the braces, two types of elements were used: One with an elasto-plastic load deformation curve (Figure 3.12-a) for modeling of BRBs (Kumar et al., 2007) and the other with the phenomenological model (Figure 3.12-b) for modeling of CBs (Jain et al., 1980; Rai and Goel, 2003; Kim and Chio, 2005). As in the study of Kumar et al. (2007), for damage evaluation of BRBs, being on the conservative side, hinge specifications in FEMA 273 (FEMA 273, 1997), for modeling braces in tension were used. The limit strains of the BRB for the calculation of the performance level, was taken as $12\varepsilon_y$ for ultimate failure, $10\varepsilon_y$ for collapse prevention, $8\varepsilon_y$ for life safety and ε_y for immediate occupancy performance levels.

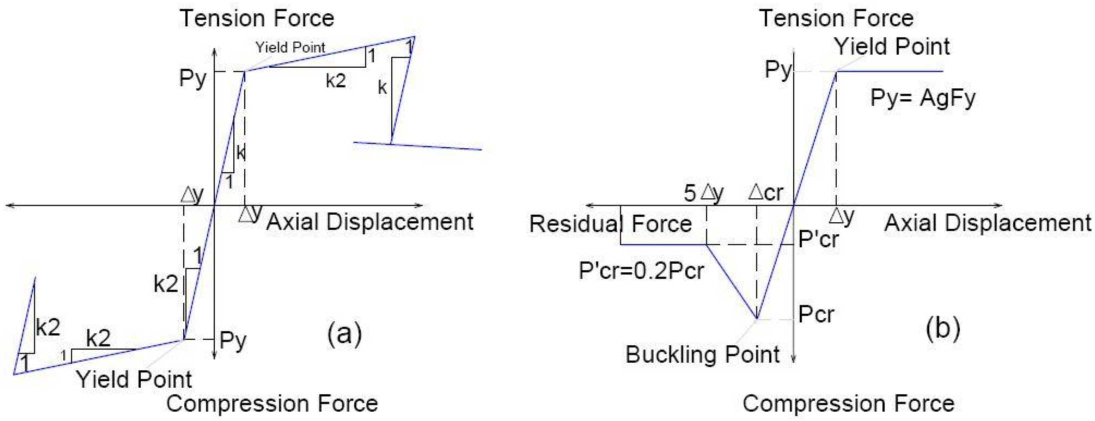


Figure 3.12 Load deformation behavior for a) BRBs (Kumar et al., 2007) and b) CBs (Kim and Chio, 2005)

The viscoelastic dampers (VDs) were modeled by nonlinear Link members. In these members, it is possible to identify independent damping properties for each deformational degree of freedom. Viscoelasticity is defined with the Maxwell model that consists of linear or nonlinear damper in series with a spring. The nonlinear behavior of the VDs is defined according to the following equation:

$$f = kd_k = cv^{cexp} \tag{3.1}$$

In this equation, f is the nonlinear force, k is the spring constant, d_k is the deformation across the spring, c is the damping coefficient, v is the velocity across the damper, and *cexp* is the damping exponent. The sum of the spring deformation (d_k) and the damper deformation (d_c) gives the deformation of the link member. The rational selection of the damping exponent is in between 0.2 and 2.0. (CSI, 2009; Teran-Gilmore and Ruiz-Garcia, 2011; Tezcan and Uluca, 2003). In this study, the damping exponent was assumed as 1.0. For seismic upgrading, three different VDs with damping coefficients of 1000, 2500 and 10000 kNs/m were utilized. Equal structural bracings having the same stiffness coefficient with the CBs were used for the viscoelastically damped frames with the intention of focusing on the effect of VDs. The damping ratio provided by the VDs is achieved by following an analogy of the logarithmic decrement method for single degree of freedom system. In the computation of the damping ratio, a decreasing sine curve of SDOF system which defined as displacement versus time was utilized as given in the following equation (Tezcan and Uluca, 2003).

$$\mathrm{Ln}\left(\frac{u_1}{u_2}\right) = 2\pi\zeta/\sqrt{(1-\zeta^2)} \qquad (3.2)$$

In this equation, u_1 and u_2 are the peak displacements at two consecutive time periods, ζ is the effective damping ratio. In the computer program, by altering the damping coefficient c of VDs, the effective damping ratio ζ is altered likewise. The damping coefficients and the corresponding effective damping ratios obtained for each frame are summarized in Tables 3.6 and 3.7 for steel ordinary moment resisting frame and reinforced concrete moment resisting frames, respectively.

Table 3.6 Damping coefficient (c) and corresponding effective damping ratio (ζ) for steel MRFs

Frame No	Three-storey		Six-storey		Twelve-storey	
	c	ζ (%)	c	ζ (%)	c	ζ (%)
VDF1000	1000	15.77	1000	10.9	1000	11.9
VDF2500	2500	25.53	2500	17.5	2500	17.8
VDF10000	10000	35.43	10000	33.6	10000	37.1

Table 3.7 Damping coefficient (c) and corresponding effective damping ratio (ζ) for RC MRFs

Frame No	Three-storey		Six-storey		Twelve-storey	
	c	ζ (%)	c	ζ (%)	c	ζ (%)
VDF1000	1000	10	1000	7.79	1000	9.19
VDF2500	2500	13.86	2500	9.98	2500	15.63
VDF10000	10000	25.43	10000	15.19	10000	17.73

3.3 Nonlinear Time History Analysis

In order to get accurate nonlinear performance of buildings, nonlinear dynamic analysis were carried out on two dimensional analytical models for the frames developed by SAP 2000 Nonlinear version 14.0, which is a general purpose structural analysis program (CSI, 2009). In this method the structure will subject to real natural ground motion records. This makes the analysis method quite different from all of the other approximate analysis methods as the inertial forces were directly determined from these ground motions and the responses of the building either in deformations or in forces are calculated as a function of time, considering the dynamic properties of the building structure. In the nonlinear time history analysis, Hilber-Hughes-Taylor direct integration method has been adopted and Rayleigh damping was used.

3.4 Ground Motions

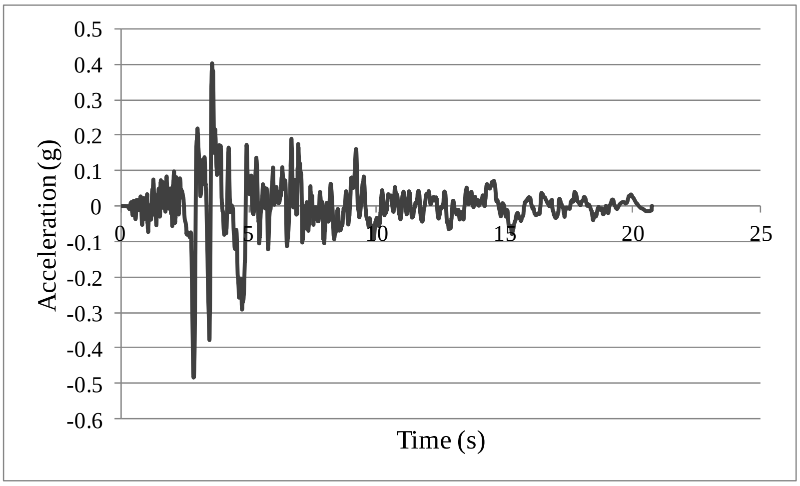
For the nonlinear dynamic analysis of the frames, a set of ground motions namely 1984 Morgan Hill, 1992 Erzincan, and 1999 Hector Mine with different intensities and characteristics were considered. As seismic hazard level, the acceleration spectrum defined in Turkish Earthquake Code (TEC 2007) for 10% probability of exceedance in 50 yrs was taken into account to assess the earthquake performance of the existing and retrofitted structures. Scaled earthquake records in order to match this target spectrum mainly in a range of periods from 0.2T to 1.5T (where T is the first natural period of the building) were obtained from PEER ground motion database (PEER, 2011). Figure 3.13 shows the earthquake acceleration time histories used for the analysis. The 5% damped average acceleration spectrum of the ground motions used, and the elastic code spectrum is given in Figure 3.14. Furthermore, Table 3.8 lists the characteristics of the earthquake ground motion records considered in this study for steel and RC frames.

Table 3.8 Properties of the earthquake ground motion records

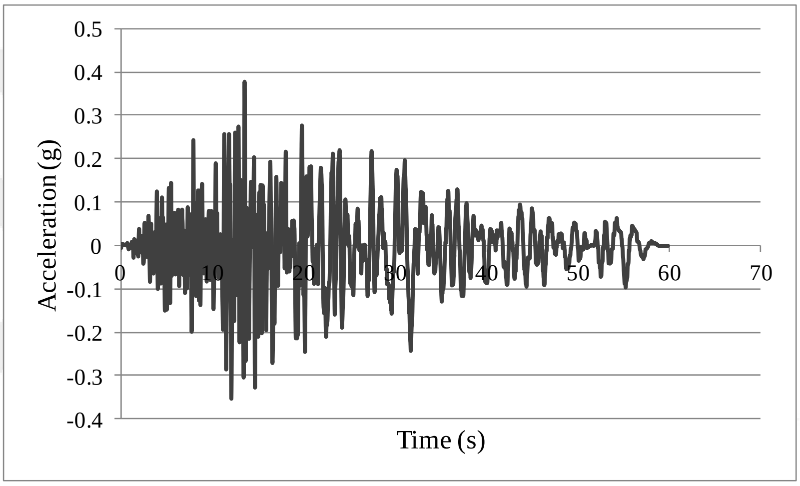
No	Earthquake Record	Station	Magnitude (Mw)	rup* (km)	Vs30** (m/s)	PGA (g)	PGV (m/s)	PGD (m)
1	Erzincan	Erzincan	6.7	4.4	274.5	0.48	0.52	0.19
2	Morgan Hill	Agnews State Hospital	6.2	24.5	239.7	0.38	0.58	0.26
3	Hector Mine	Beverly Hills Pac Bell Bsmt	7.1	191.4	301	0.51	0.92	0.78

*Restrict range of closest distance to rupture plane

**Average shear wave velocity of top 30 m of the site



(a)



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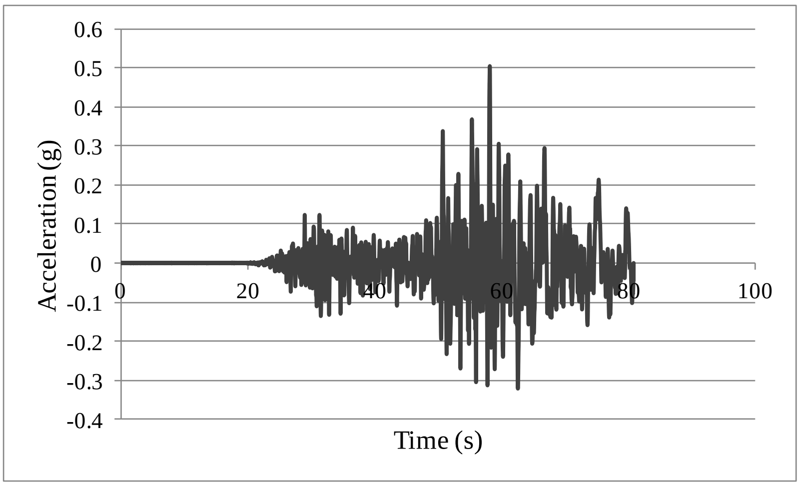


(c)

Figure 3.13 Earthquake ground motion records used for the analysis: a) Erzincan, b) Morgan Hill, and c) Hector Mine

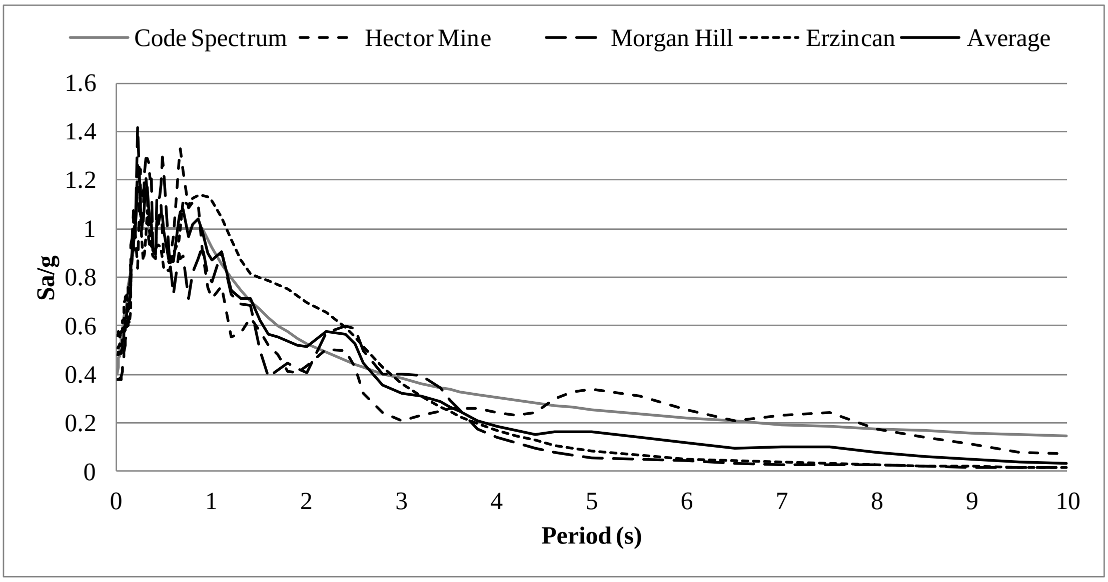


Figure 3.14 Code spectrum and average acceleration spectrum of the scaled earthquake ground motions

CHAPTER 4

RESULTS AND DISCUSSION

In this chapter, results of nonlinear dynamic analysis of unbraced frames (UFs), conventionally braced frames (CBFs), buckling restrained braced frames (BRBFs) and viscoelastic damped frames (VDFs) has been discussed, comparatively. A total of 36 different cases were taken into account and structural performance of three cases under the effect of three different earthquake ground motion were evaluated. Performance of the frames studied through plastification of structure members, interstorey drift index, global damage index, roof drift history, height-wise distribution of peak storey drifts and base shear.

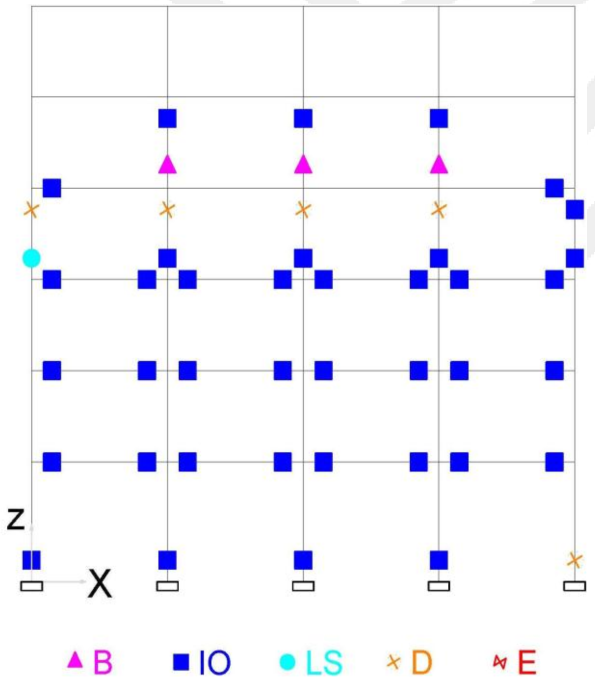
4.1 Steel Moment Resisting Frames

Considering the structural models and excitations described in this study, a comparison between different retrofitting strategies was made; a series of nonlinear dynamic analysis was performed using three different ground motion records for 18 different cases. These cases cover three storey, six storey, and twelve storey unbraced frames (UBFs), conventional braced frames (CBFs), buckling-restrained braced frames (BRBFs), and viscoelastically damped frames (VDFs) with three different damping coefficients of 1000, 2500, and 10000 kNs/m. The results were summarized below:-

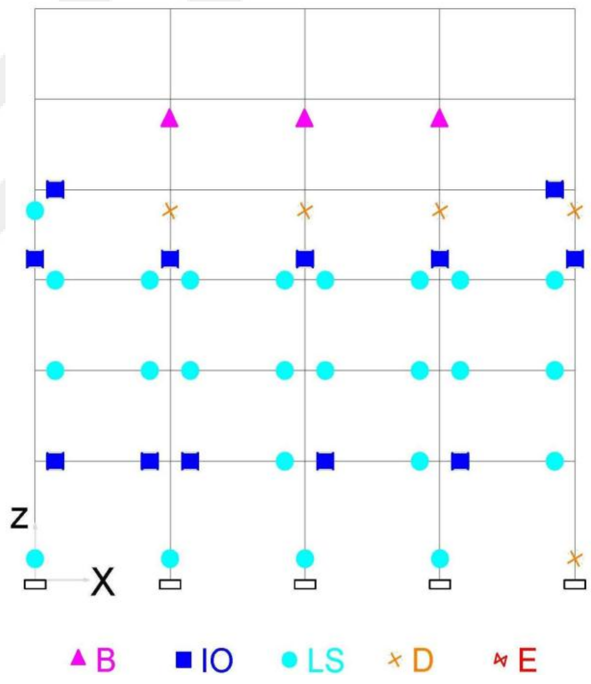
4.1.1 Plastification of Structural Members

In order to address the local deformations of the structural members, the plastifications of the structural members were investigated. Plastic hinge formations and their performance levels for the 6 storey frames considered as a sample and shown in Figures. 4.1 to 4.6. The notations IO, LS, and CP stand for immediate occupancy, life safety, and collapse prevention performance levels, respectively. The results revealed that especially the buckling restrained braces and the viscoelastic dampers with high damping coefficients are effective in reducing plastification observed in the UBFs. It was also observed that, depending on the earthquake acceleration, the plastification demand of the UBFs changed. For instance, for the 6 storey UBF, Morgan Hill and Hector Mine earthquake accelerations impose more plastic hinge formation compared to Erzincan earthquake acceleration. As seen in Figure 4.1, under the effect of Morgan Hill and Hector Mine earthquake accelerations, four columns at the fourth storey reached the collapse stage whereas under the effect of Erzincan earthquake acceleration, the frame was in the elastic range of deformation.

Comparing the performance of different seismic resisting systems considered in this study, it was observed that viscoelastic dampers (VDs) with damping coefficient 10000 kNs/m (c=10000) provides the least nonlinear behavior in beams and columns for 3, 6 and 12 storey structures. Furthermore, the BRBs and VDs with damping coefficient of 2500 kNs/m provided considerable improvement in seismic response of the UBFs by decreasing the plastification demand in critical structural members. Generally, this fact holds true independent of the frame height and the exposed earthquake acceleration. For example, the frames upgraded with BRBs remained mainly in the elastic range of deformation except the BRBs. In the VDF with c=2500 kNs/m , under Hector Mine and Morgan Hill earthquakes some plastifications mainly in the beams; under Erzincan earthquake, complete elastic behavior was observed as seen in Figure 4.4. On the other hand, the VDF with c=1000 kNs/m was not effective for upgrading the seismic behavior of the UBFs as shown in Figure 4.5. Among the rehabilitation approaches considered, addition of CBs to the unbraced frames was found to be the least effective in reducing the plastification, due to the poor behavior of conventional braced frames (CBFs). Conventional brace systems that have different load carrying capacity in compression and tension, exhibit buckling deformation in compression. The buckling of the braces resulted in a sudden transferring of the input energy to the beams and columns and caused formation of plastic hinges in the other structural members that might also cause the failure of the building. When CBs are compared with BRBs, it was seen that the significantly improved behavior of the BRBF comes from the enhanced nonlinear hysteretic performance and energy absorption capacity of the BRBs under compression.



a)



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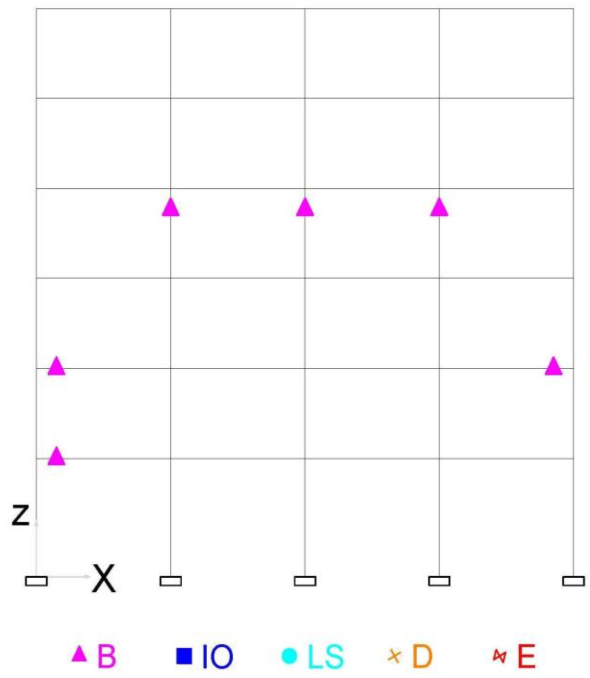


c)

Figure 4.1 Plastic hinge formations in 6 storey steel UBFs subjected to a) Hector Mine, b) Morgan Hill and c) Erzincan earthquakes

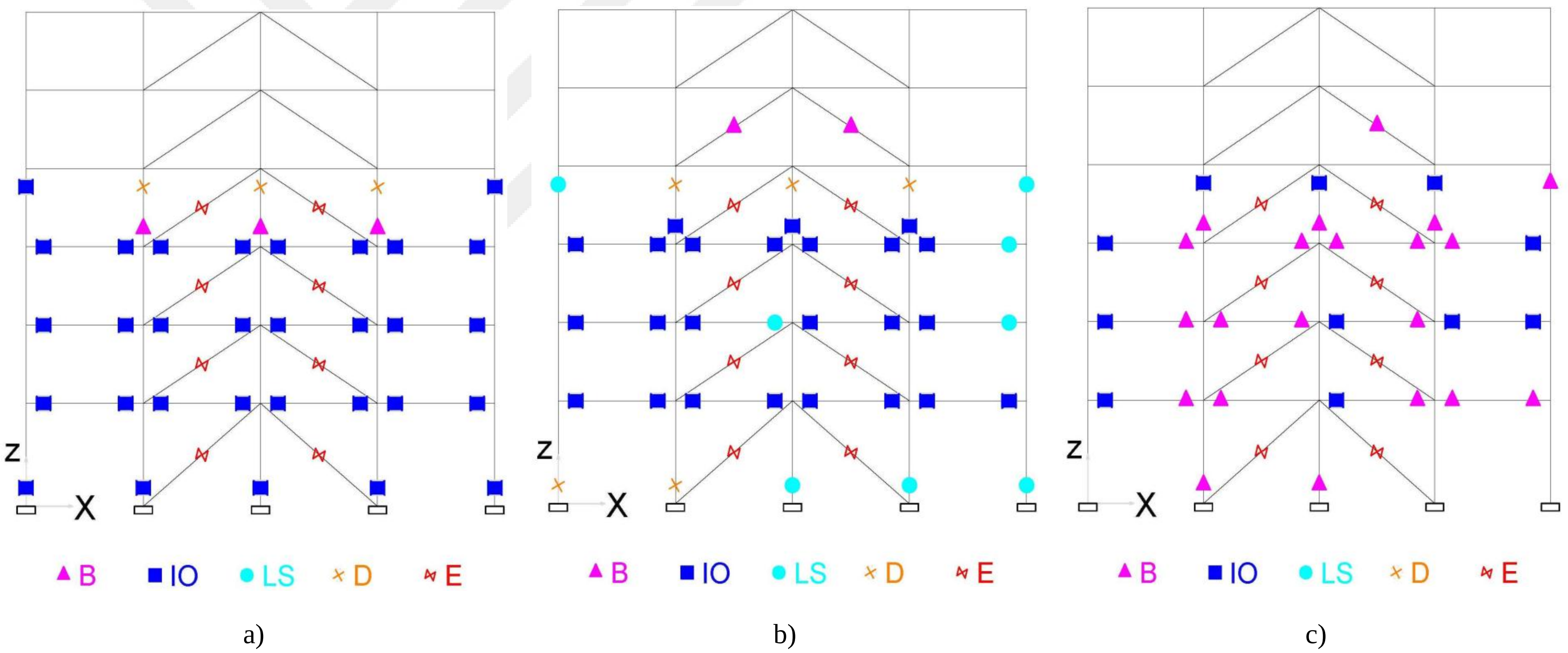


Figure 4.2 Plastic hinge formations in 6 storey steel CBFs subjected to a) Hector Mine, b) Morgan Hill and c) Erzincan earthquakes

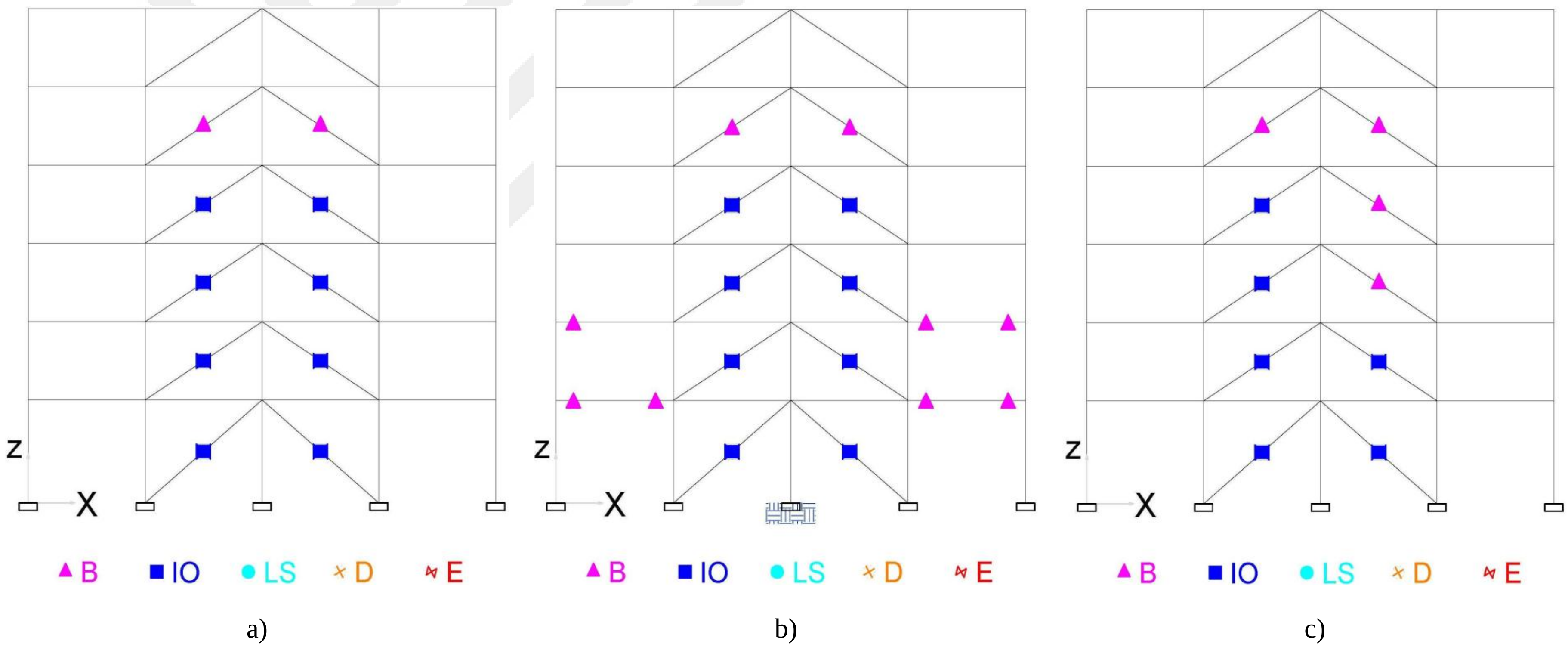
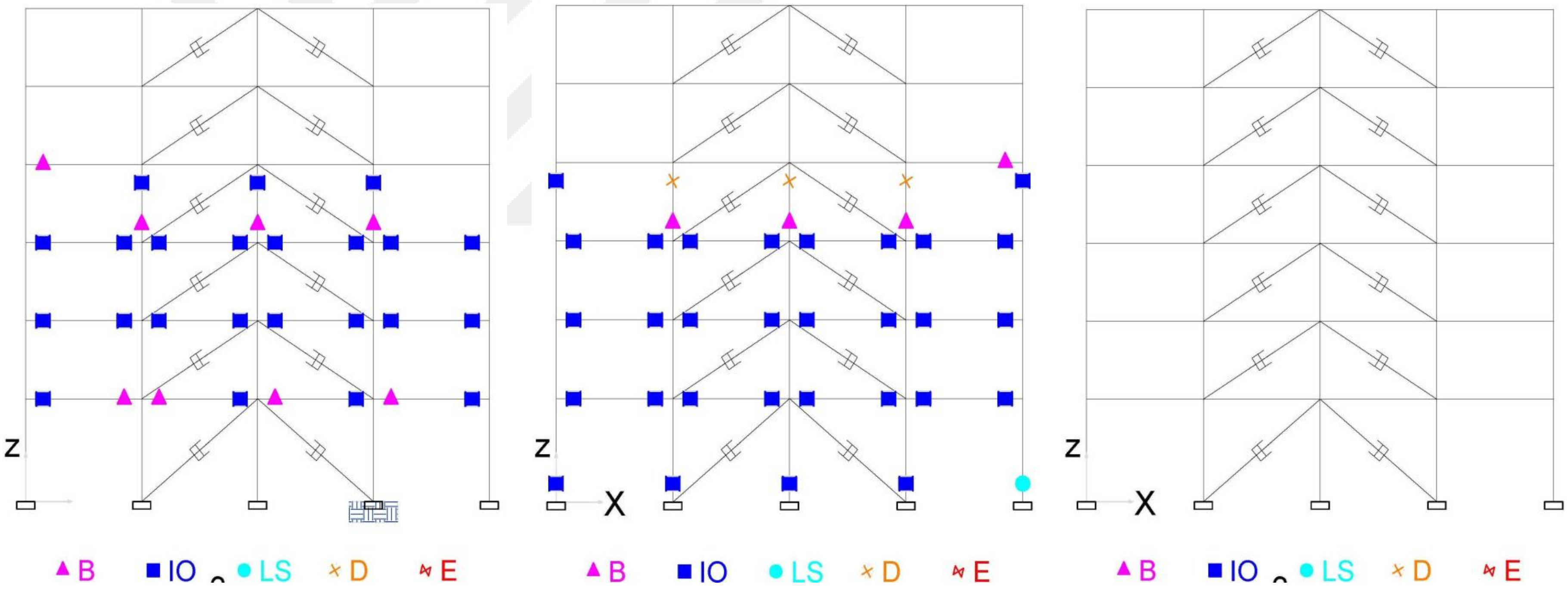


Figure 4.3 Plastic hinge formations in 6 storey steel BRBFs subjected to a) Hector Mine, b) Morgan Hill and c) Erzincan earthquakes



a) b) c)

Figure 4.4 Plastic hinge formations in 6 storey steel VDF with c=1000 kNs/m subjected to a) Hector Mine b) Morgan Hill, and c) Erzincan earthquakes

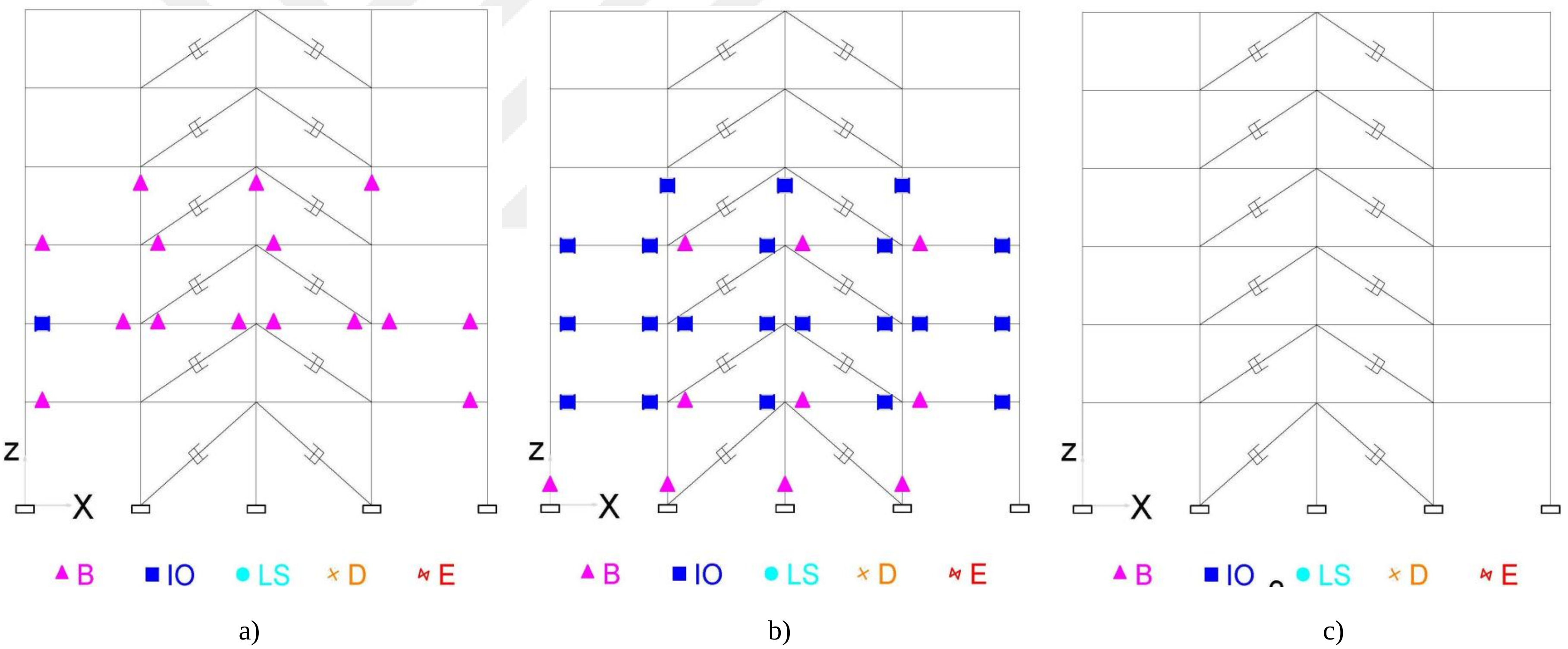
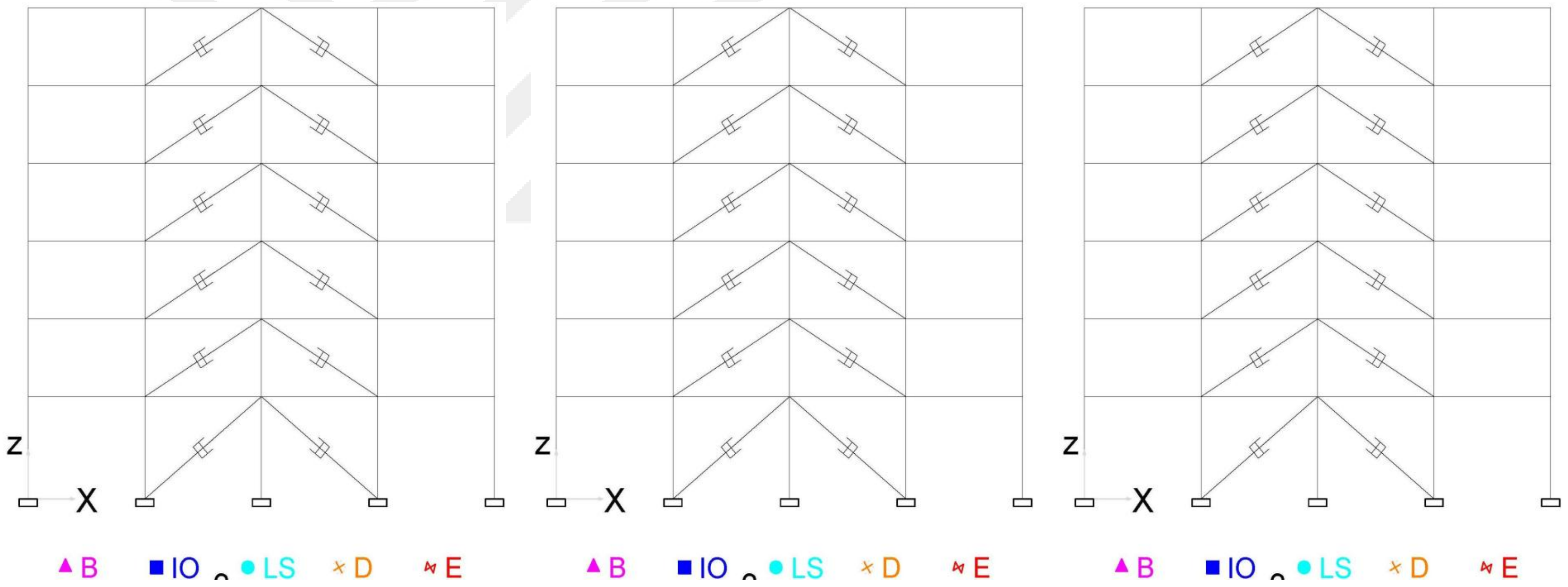


Figure 4.5 Plastic hinge formations in 6 storey steel VDF with c=2500 kNs/m subjected to a) Hector Mine, b) Morgan Hill and c) Erzincan earthquakes



a) b) c)

Figure 4.6 Plastic hinge formations in 6 storey steel VDF with c=10000 kNs/m subjected to a) Hector Mine, b) Morgan Hill and c) Erzincan earthquakes

4.1.2 Interstorey Drift Index

The maximum interstorey drift (δ_{max}) divided by the storey height (h) is defined as "Maximum Interstorey Index". This index is a good indication of the damages experienced by the structural members. Drift limit requirements depend on the building code and analysis procedure that has been utilized. In this study, 0.4g was considered as the acceleration for life safety (LS) performance state, this index was takes as 1.5% for LS according to (SEAOC, 1999) requirements. Dynamic responses of the frames can vary depending on the earthquake acceleration used. In this study, generally under Morgan Hill earthquake acceleration, the greater interstorey drift index was observed for the frames with and without passive energy dissipation systems. The inter-storey index obtained for 3, 6 and 12 storey frames are presented in Figures 4.7, 4.8 and 4.9. It is clear from the figures that the frames with passive energy control devices have interstorey index values less than those of the unbraced frames. However, the decrease in the displacement demand in terms of interstorey drift index could not be seen for 3 and 6 storey CBFs. Such increment in the displacement demand of these CBFs particularly refers to the poor nonlinear behavior of conventional braces under large compressive stresses. Among the passive energy dissipation systems utilized, the BRBs and VDs with damping coefficient of c=10000 kNs/m appear to be very effective in reducing the displacement demand of the UBFs. For example, with the addition of BRBs, a significant reduction in interstorey drift index values was obtained especially when subjected to Hector Mine and Morgan Hill earthquakes.

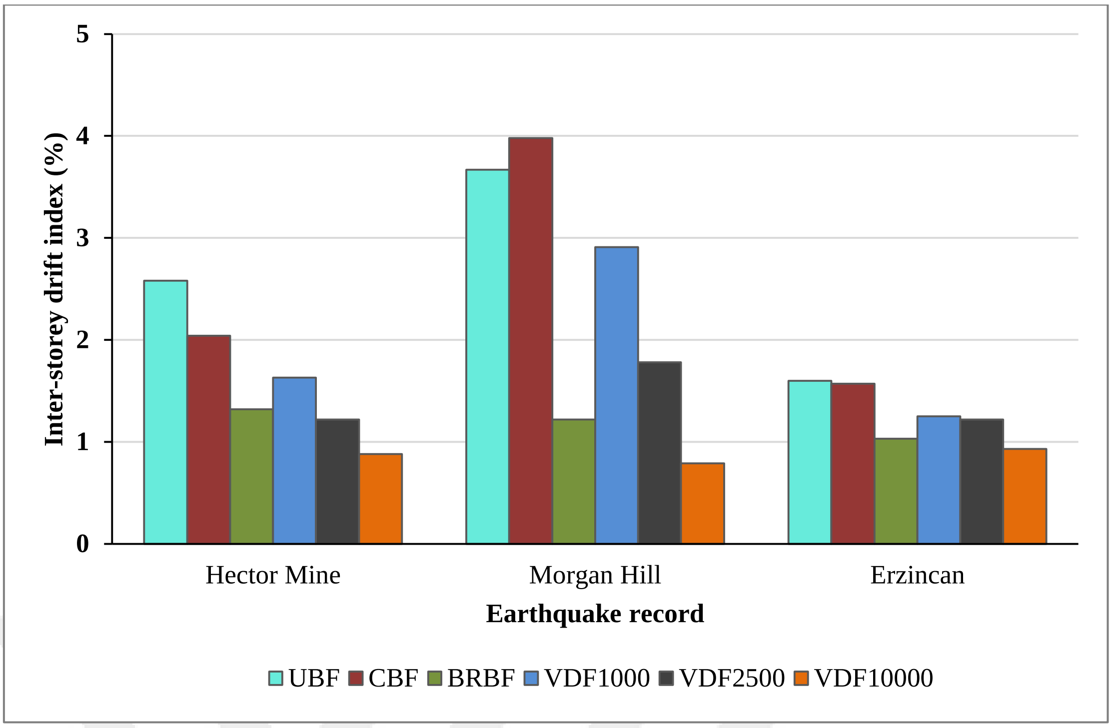


Figure 4.7 Interstorey drift index for 3 storey steel frames

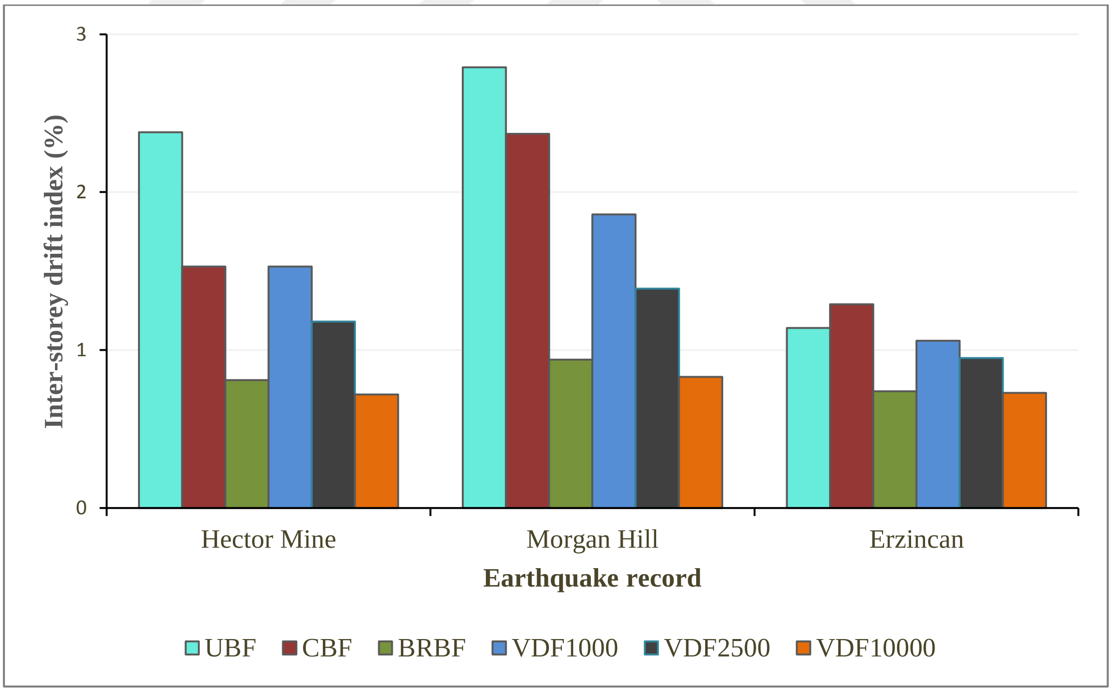


Figure 4.8 Interstorey drift index for 6 storey steel frames

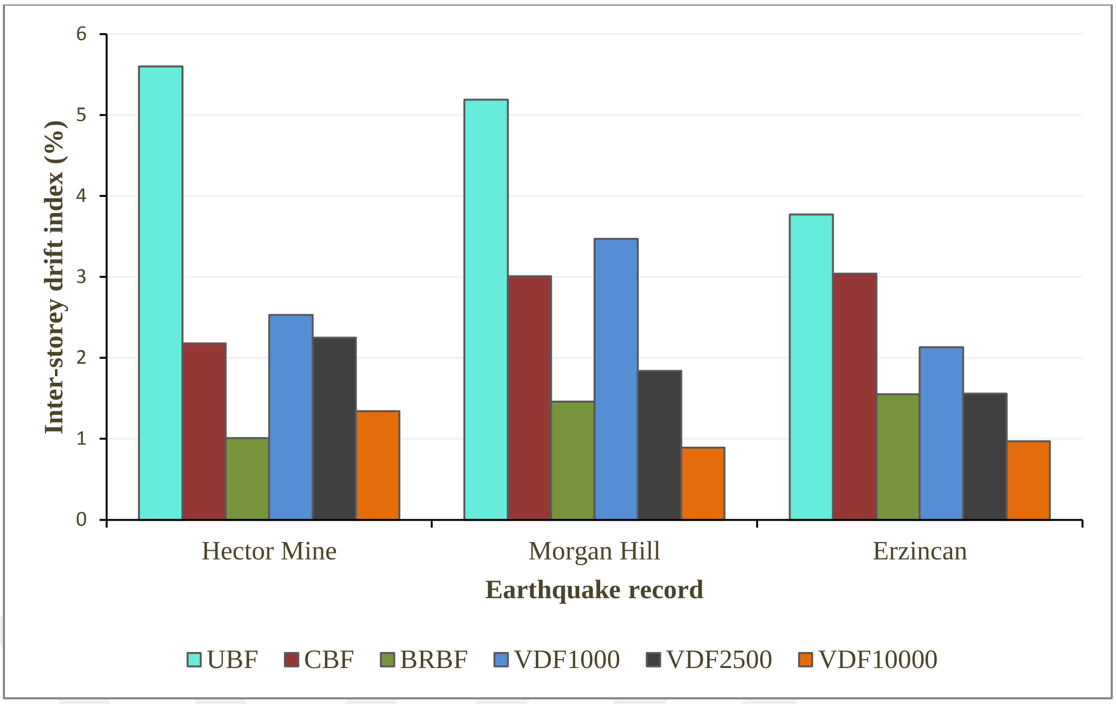


Figure 4.9 Interstorey drift index for 12 storey steel frames

4.1.3 Global Damage Index

The ratio of the roof displacement to the building height (D/H) called as "Global Gamage Index". Even though interstorey drift index and the global damage index have been used as global damage parameters for evaluating dynamic performance of structures under seismic actions, the interstorey drift index has been much more preferred than the global damage index. This is due to the fact that in an individual storey the latter may exceed by a factor of 2 or more (Krawinkler and Gupta, 1988; Barroso et al., 2002).

The global damage index obtained for 3, 6 and 12 storey frames are presented in Figures 4.10, 4.11 and 4.12. The results showed clearly that using passive energy control devices improved nonlinear behavior of the frames and reduced the global damage index. Similar to the results of interstorey drift index, the results showed that

BRBs and VDs with damping coefficient of c=10000 kNs/m appear to be the most effective approaches in reducing the displacement demand of the UBFs.

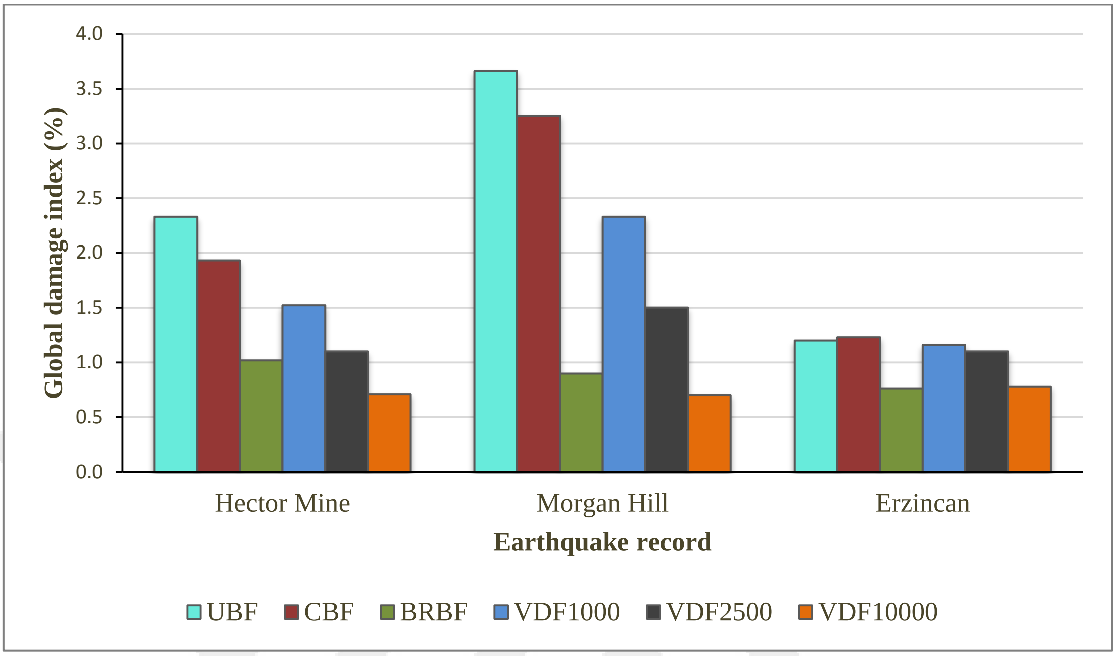


Figure 4.10 Global damage index for 3 storey steel frames

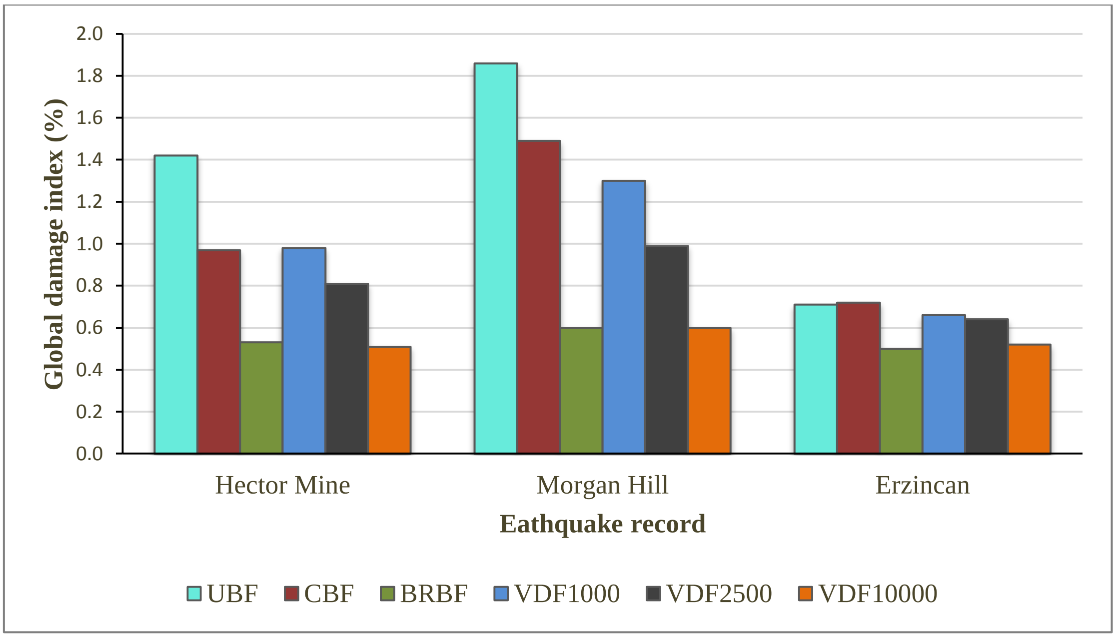


Figure 4.11 Global damage index for 6 storey steel frames

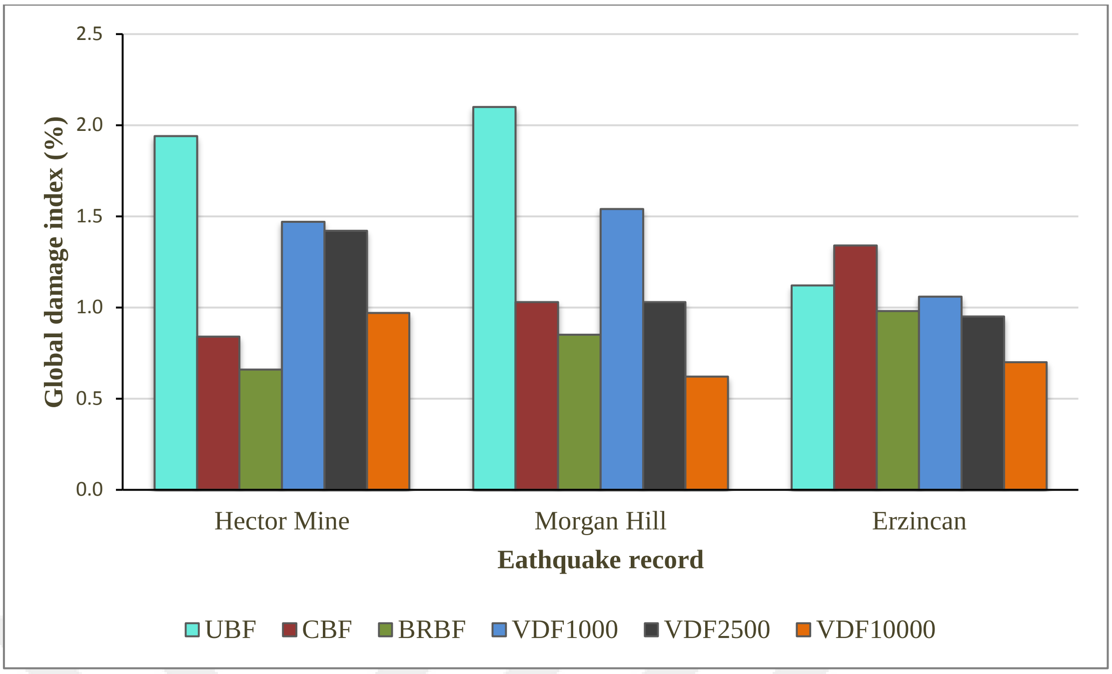
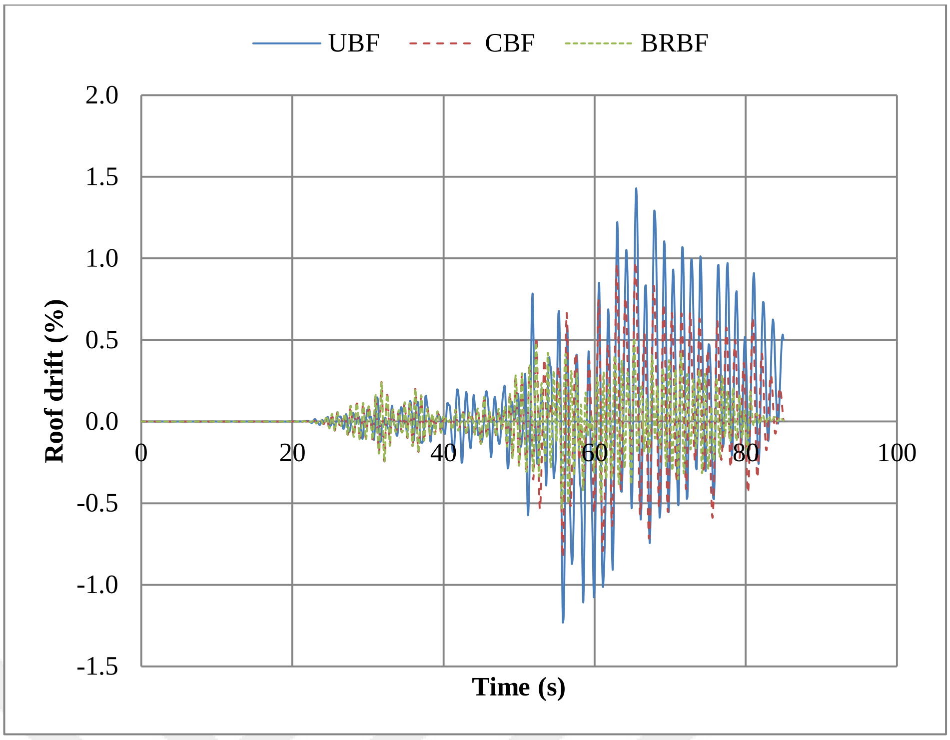


Figure 4.12 Global damage index for 12 storey steel frames

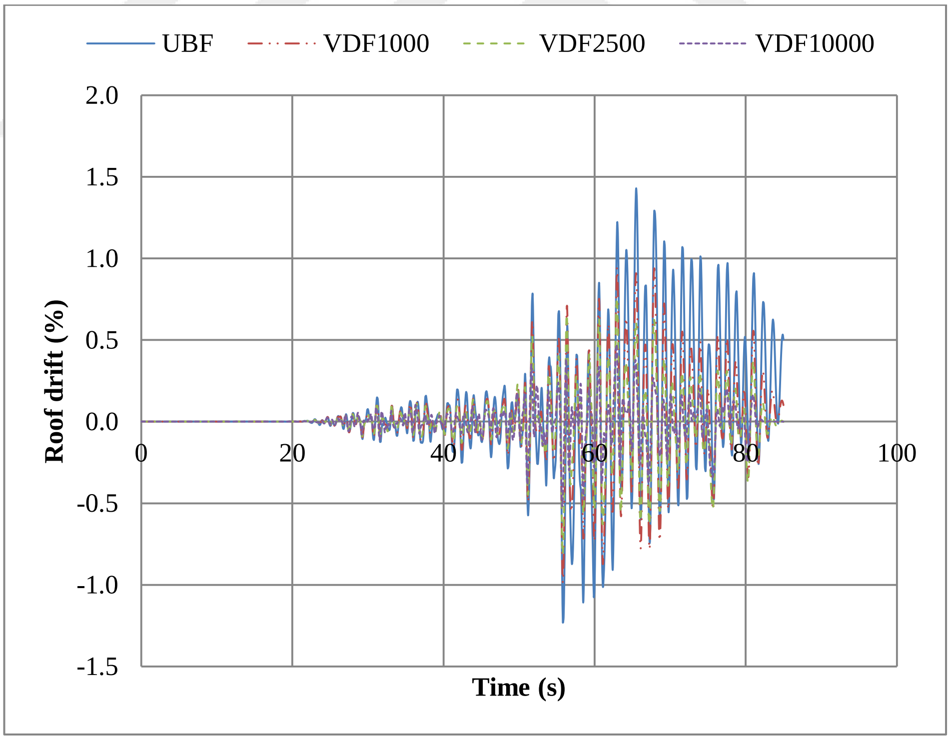
4.1.4 Roof Drift Time History

Figures 4.13, 4.14 and 4.15 provide roof drift time history response of the 6 storey frame before and after addition of traditional and innovative systems when subjected to Hector Mine, Morgan Hill, and Erzincan earthquake accelerations. Moreover, roof drift time history for 3 and 12 storey frames were presented in Appendix A.

The results of the drift response histories confirm the beneficial effects of using innovative energy dissipation systems especially BRBs and VDs with high damping coefficient, in reducing the lateral displacements. On contrary, the traditional bracing systems and the VDs with low damping coefficient (c=1000 kNs/m) were not as efficient as the other systems. For instance, under Morgan Hill earthquake acceleration, the residual roof drift obtained for 6 storey CBF and VDF with damping coefficient of c=1000 kNs/m is 0.693% and 0.391%, respectively. While the corresponding residual drifts for BRBFs and VDFs with damping coefficient of c=10000 kNs/m are 0.01% and 0%, respectively. On the other hand, when the 6 storey VDFs were compared with each other, the residual drifts are 0.386%, 0.129%, and 0% for VDFs with damping coefficient of 1000, 2500 and 10000 kNs/m, respectively. Consequently, it can be said that as the damping coefficient of the VDs and the total effective damping ratio of the frame increases, the roof residual roof drift decreases. It should be noted that each record was fictitiously lengthened by 5 seconds at zero acceleration in order to consider 5 seconds of free vibrations after the final post-quake configuration, which are essential to determine the residual drifts from the nonlinear time history analyses. Furthermore, when the times of the maximum response of the frames were compared, it was observed that the maximum response of the frames did not occur at the same time. The maximum roof drift for 6 storey frames under Morgan Hill record occurred at 37.75 s and 24.1 s for CBF and BRBF, respectively; while the maximum roof drift occurred at 32.1 s, 32.05 s, and 31.95 s for VDFs with damping coefficients of 1000 kNs/m, 2500 kNs/m, 10000 kNs/m, respectively. The performed comparative analysis of the roof drift demand exhibited that both BRBs and VD with damping coefficient of c=10000 kNs/m are the most effective controlling systems. They result in average reduction in the maximum roof drift by 51% and 52%, respectively.

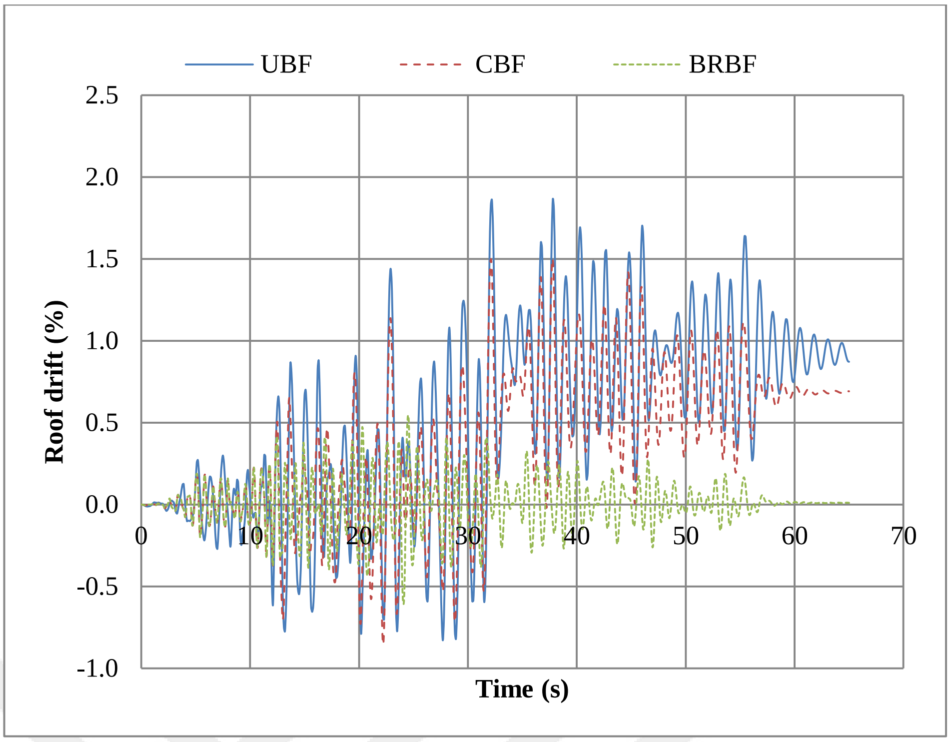


(a)

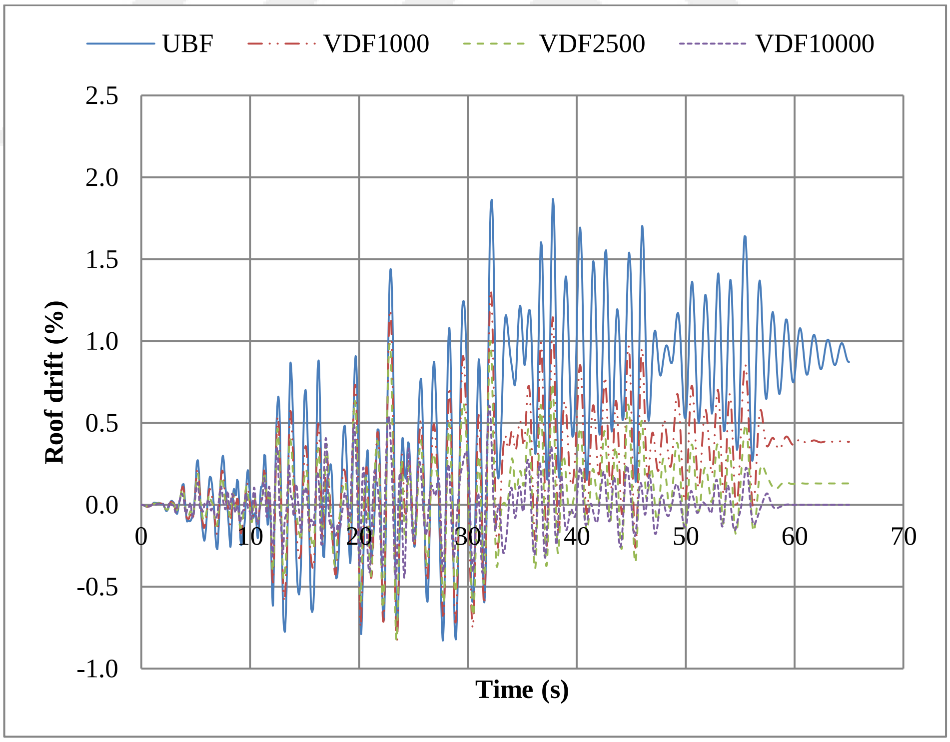


(b)

Figure 4.13 Roof drift time history response of the 6 storey steel a) UBF, CBF and BRBF, b) UBF and VDFs under Hector Mine earthquake.

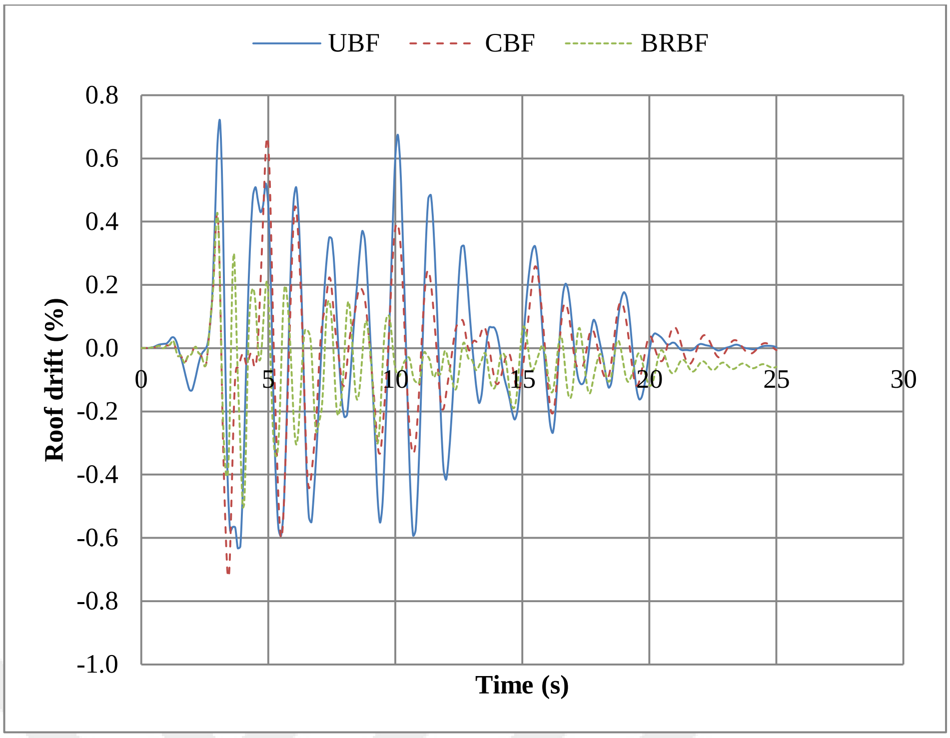


(a)

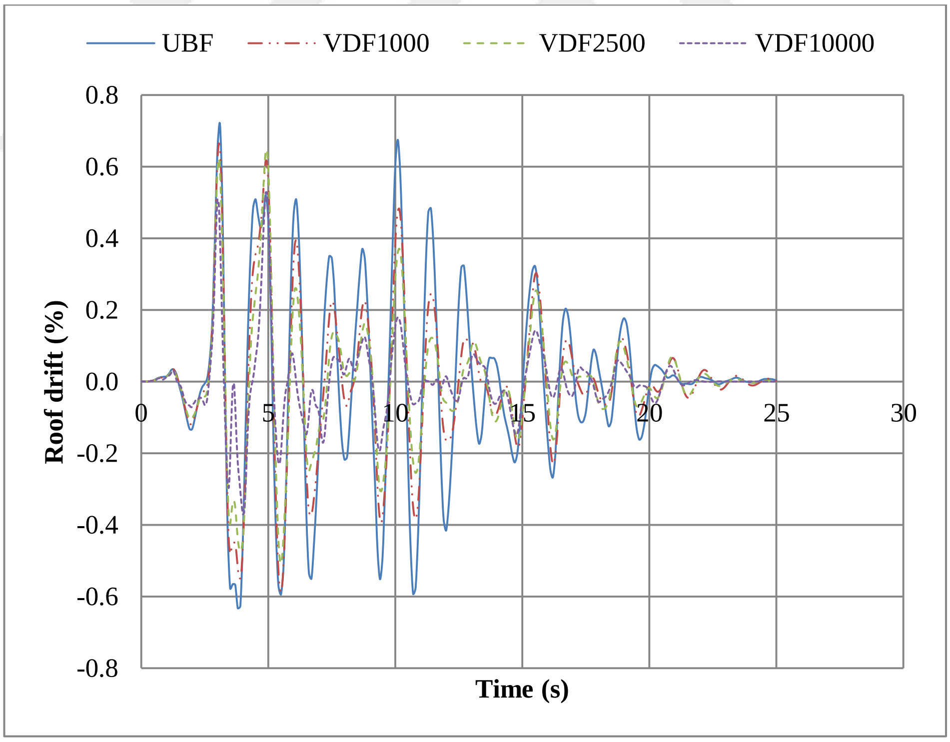


(b)

Figure 4.14 Roof drift time history response of the 6 storey steel a) UBF, CBF and BRBF, b) UBF and VDFs under Morgan Hill earthquake.



(a)



(b)

Figure 4.15 Roof drift time history response of the 6 storey steel a) UBF, CBF and BRBF, b) UBF and VDFs under Erzincan earthquake.

4.1.5 Height – Wise Distribution of Peak Storey Drift

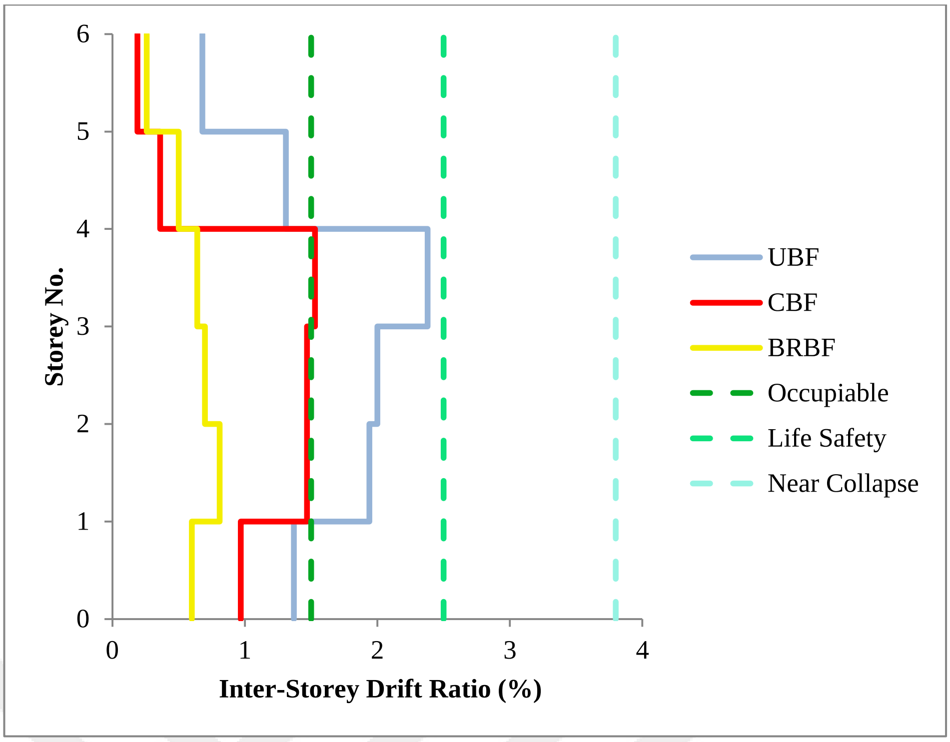
Only the interstorey drift index is not sufficient to fully understand the demands on the structural systems. Distribution of the drift demands over the height of the structure should be evaluated (Barroso et al., 2002). Figures 4.16, 4.17 and 4.18 show the height-wise distribution of the maximum interstorey drift demands of the original and upgraded 6 storey frames when subjected to ground excitations of Hector Mine, Morgan Hill, and Erzincan. Furthermore, height-wise distribution of the maximum interstorey drift for 3 and 12 storey steel frames were presented in Appendix B. In the present study, four structural performance levels, namely, operational (OP), occupiable (OC), life safety (LS) and near collapse (NC) limit states were considered for the structural assessment carried out. These limit states were determined in compliance with the SEAOC provision (SEAOC, 1999); and the relationship between seismic performance limit states and maximum transient drift ratios according to this provision is summarized in Table 4.1. The limit interstorey drift ratios for the OC, LS and CP performance levels have been also included in these plots as benchmarks.

Table 4.1 Structural performance levels (SEAOC, 1999)

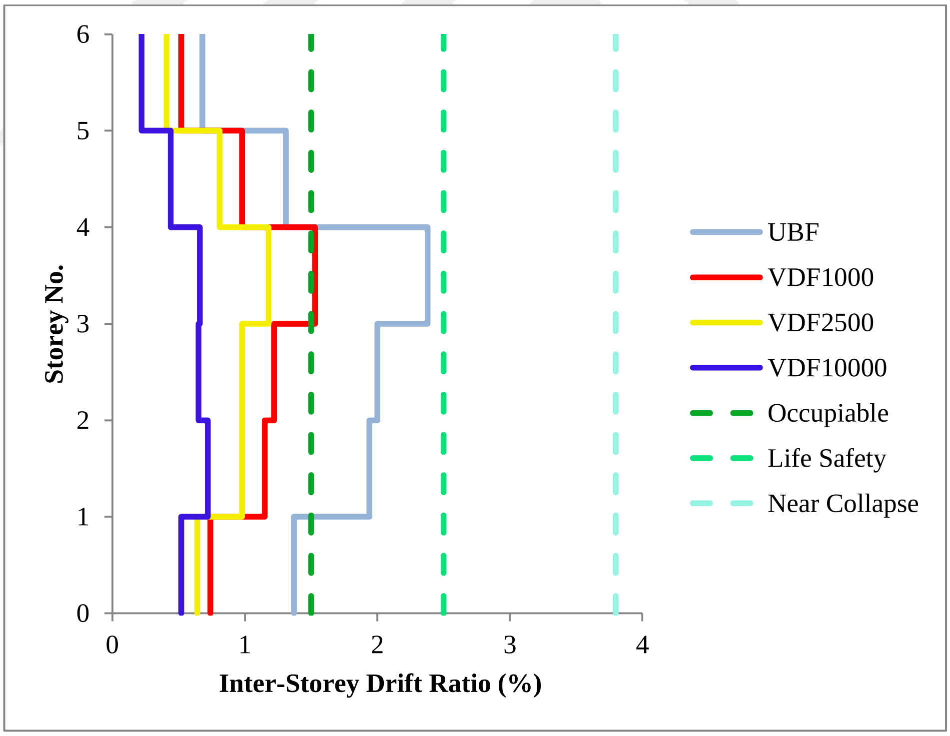
Performance level	Qualitative	Damage type	Recommended Interstorey drift ratio
SP-1	Operational	Negligible	0.5%
SP-2	Occupiable	Light	1.5%
SP-3	Life safety	Moderate	2.5%
SP-4	Near collapse	Severe	3.8%

In particular, BRBs provided a uniform distribution of the interstorey drift demand throughout the height of the structure and led to interstorey drift demands that were below OC limit state except the 12 storey frame of which interstorey drift ratio reached to 1.55%. In the CBFs, there was a concentration of large deformation in one or more storeys of the frame such that there were abrupt changes in the first and fourth floors of the 6 storey CBF under Morgan Hill ground motion record as shown in Figure 4.17-a.

Examination of the maximum interstorey drift throughout the height of the viscoelastic damped frames showed that the drift demand in all storeys was controlled effectively for the case with a damping coefficient of c=10000 kNs/m. In which, a uniform distribution of interstorey drift over the height were provided and interstorey drift demands were below OC limit state. However, it was found that there was a concentration of large deformation for the VDFs with damping coefficients of c=1000 kNs/m and c=2500 kNs/m. As it was the case for 6-storey VDF with damping coefficient of c=1000 kNs/m under Morgan Hill record, the interstorey drift demand in the second, third, and fourth floors is appreciably higher (by a factor of about 2) than first, fifth and sixth floors. Similarly, for 6 storey VDF with a damping coefficient of c=2500 kNs/m subjected to Morgan Hill ground motion, the peak inter-storey drift in the mid floors is 1.3 and 2.1 times higher than the lower and upper floors, respectively. Thus, with the increase in the damping coefficient of VDs, more uniform distribution of the drift was obtained.

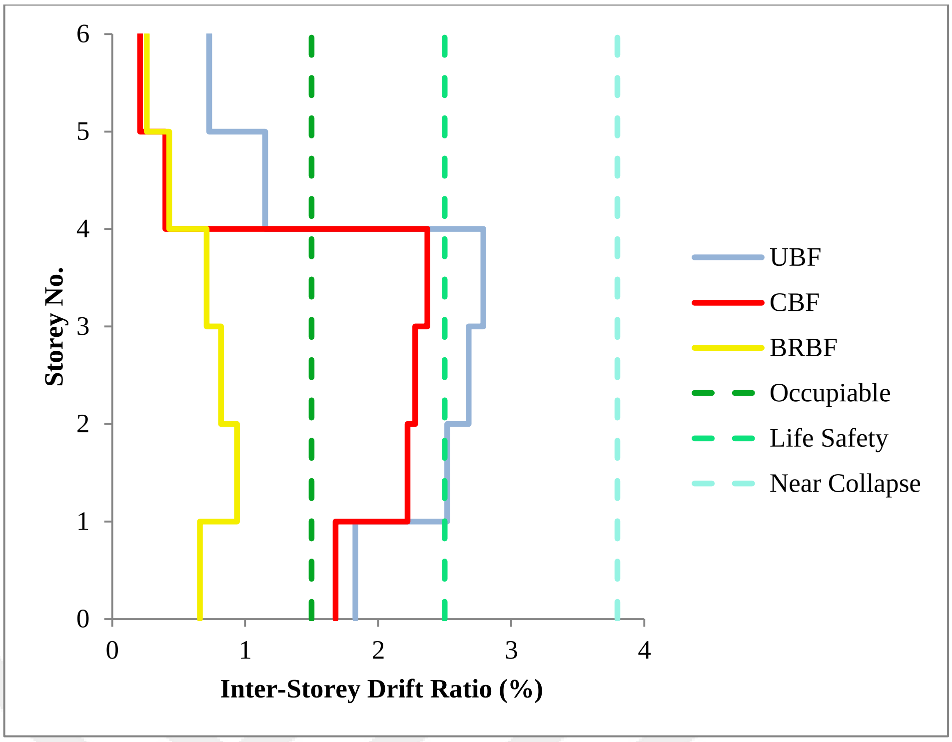


(a)

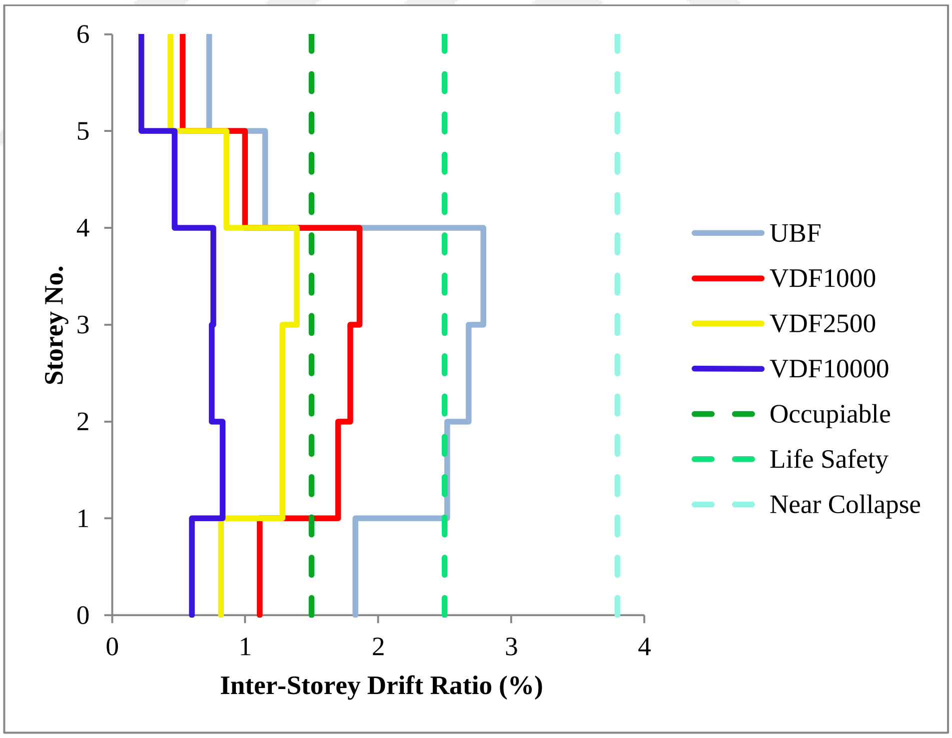


(b)

Figure 4.16 Height – wise distribution of peak storey interstorey drift of the 6 storey steel a) UBF, CBF and BRBF, b) UBF and VDFs under Hector Mine earthquake.

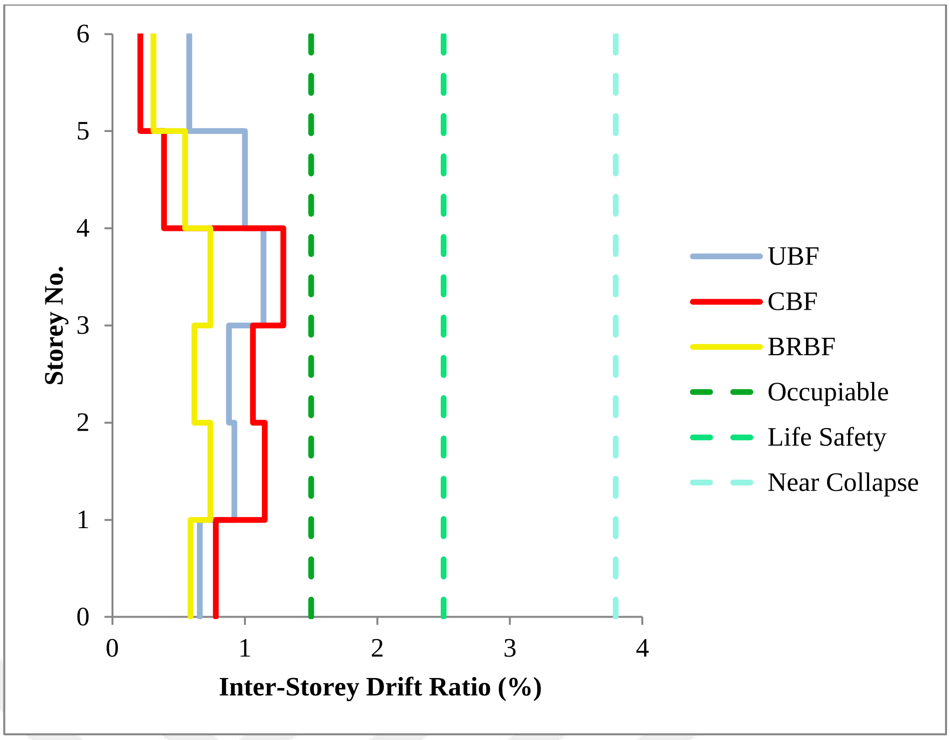


(a)

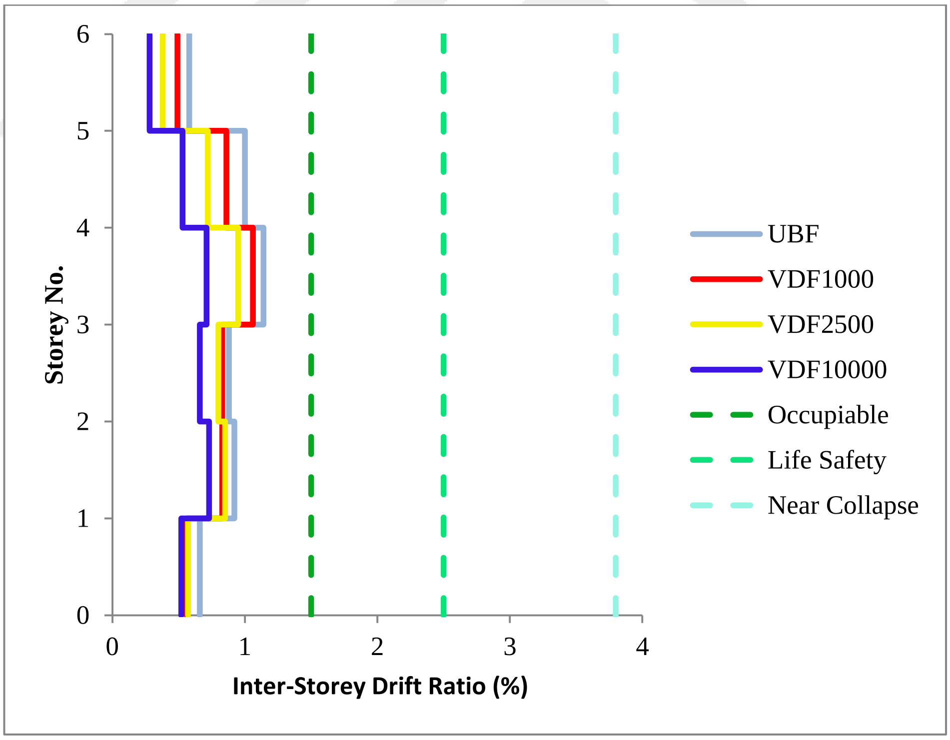


(b)

Figure 4.17 Height – wise distribution of peak storey interstorey drift of the 6 storey steel a) UBF, CBF and BRBF, b) UBF and VDFs under Morgan Hill earthquake.



(a)



(b)

Figure 4.18 Height – wise distribution of peak storey interstorey drift of the 6 storey steel a) UBF, CBF and BRBF, b) UBF and VDFs under Erzincan earthquake.

4.1.6 Base Shear

The base shear demand of the UBFs, CBFs, BRBFs and VDFs subjected to three ground motions (Hector Mine, Morgan Hill and Erzincan) are given in Figures 4.19, 4.20 and 4.21. The outcome of the analysis demonstrates that the use of viscoelastic dampers does not have considerable effect on the base shear demand. On contrary, the addition of the conventional and buckling restrained braces resulted in a significant increase in the base shear demand of the original frame. Strength of frame, peak ground acceleration, and earthquake type affected the variation of base shear forces. Position of the fundamental period of the frame with respect to earthquake excitation acceleration spectrum defined this variation in the elastic stage. Nevertheless, the base shear distribution was affected significantly by T_s/T_p ratio.

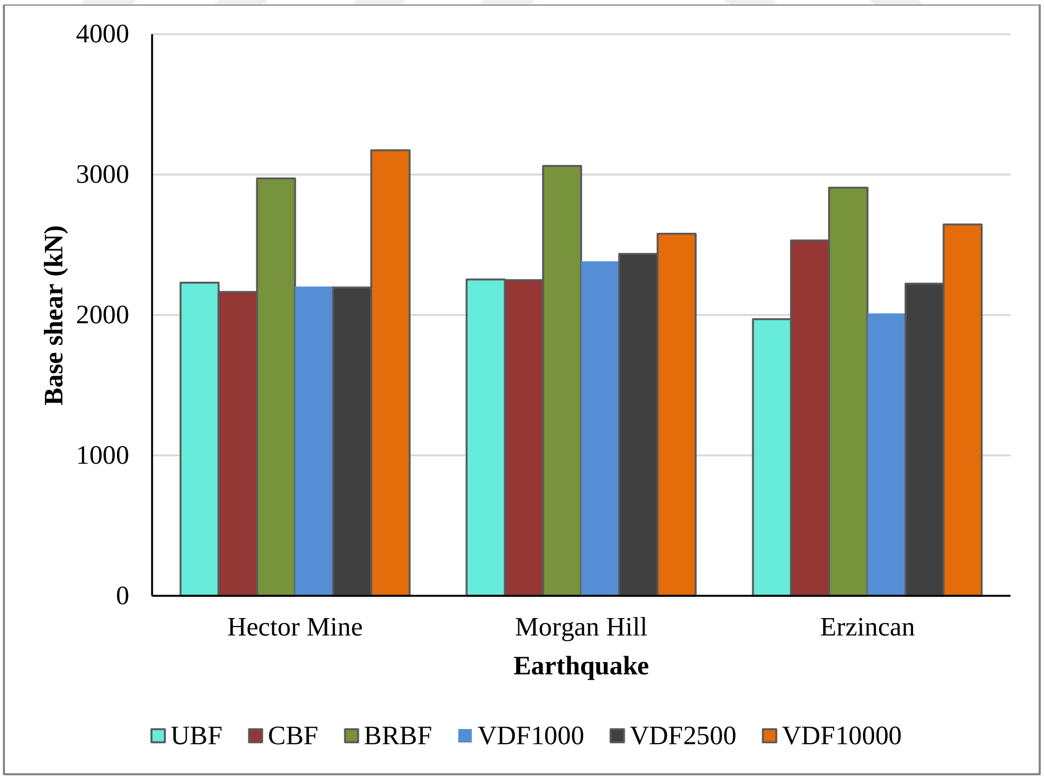


Figure 4.19 Comparison of the maximum base shear demand of 3 storey steel frames

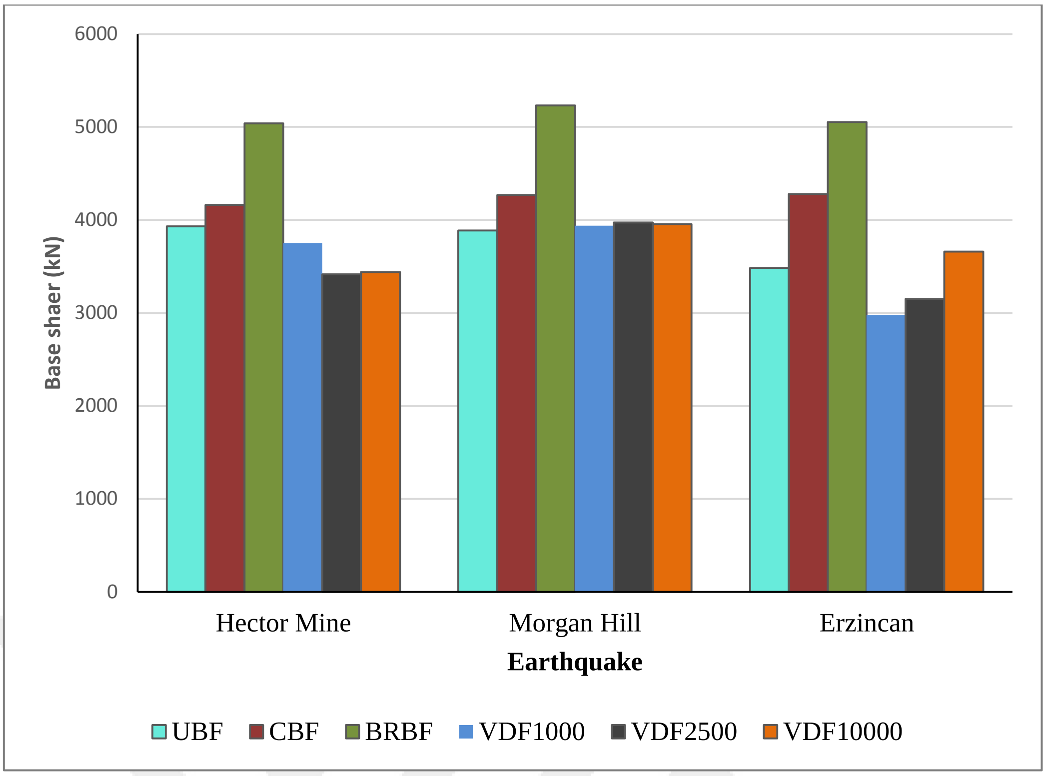


Figure 4.20 Comparison of the maximum base shear demand of 6 storey steel frames

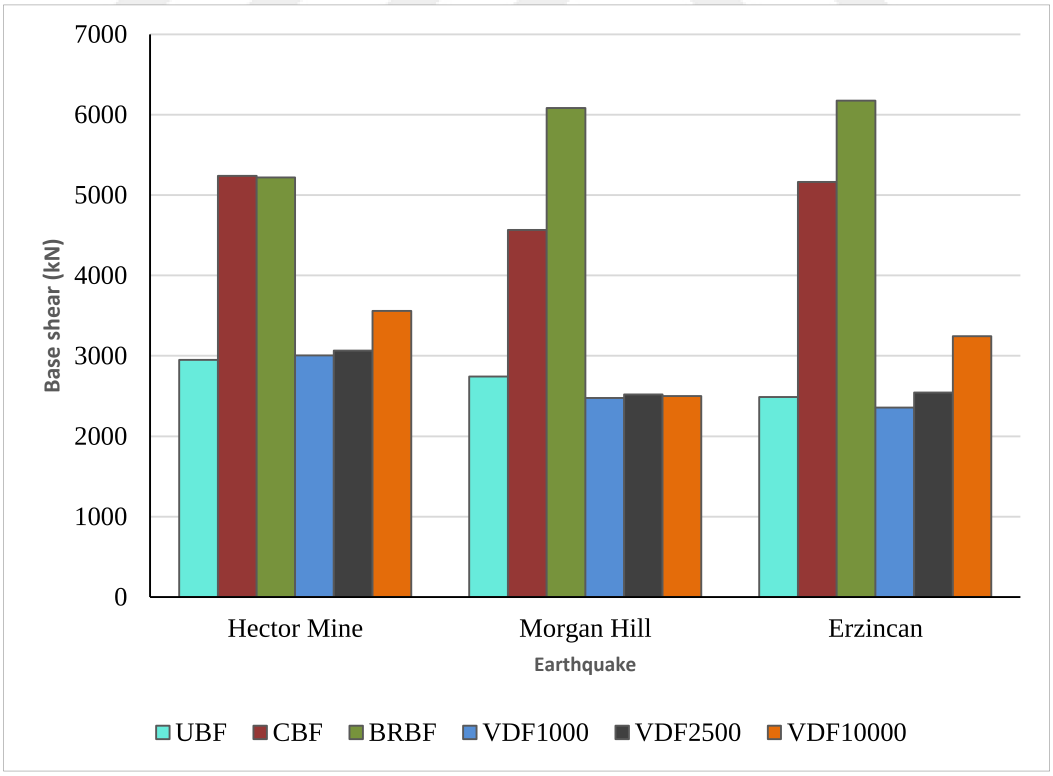


Figure 4.21 Comparison of the maximum base shear demand of 12 storey steel frames

4.2 Reinforced Concrete Frames

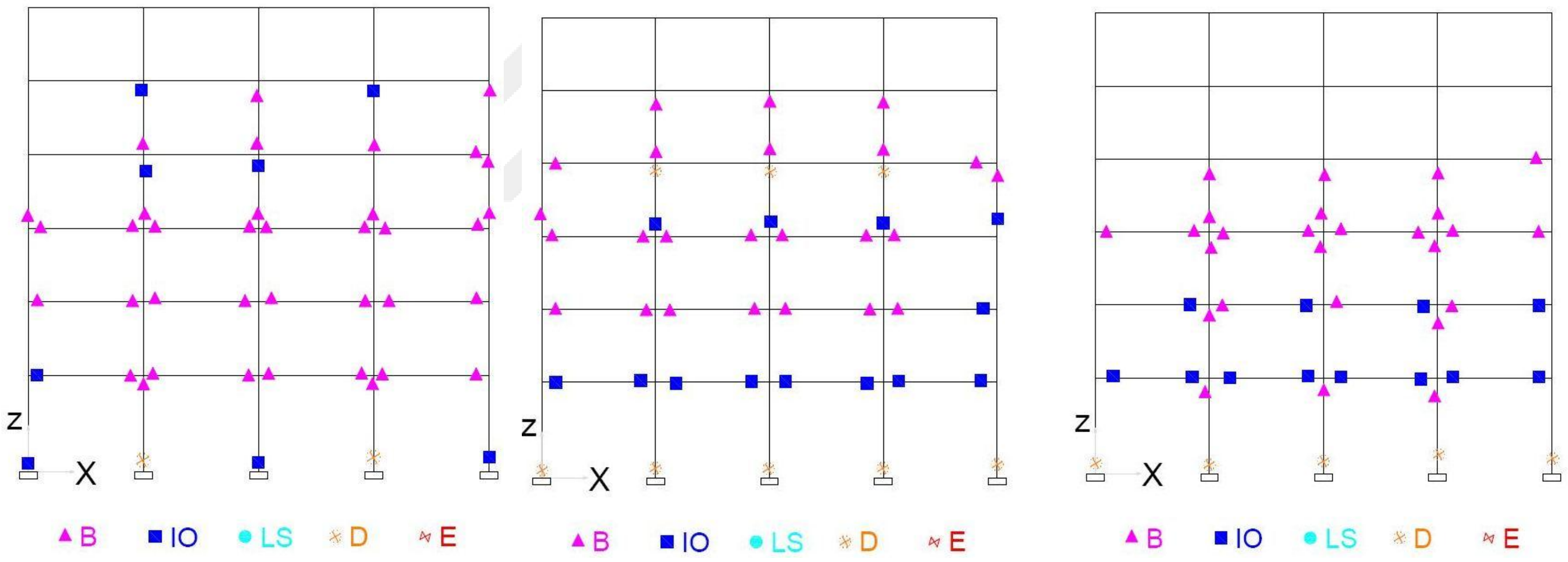
A total of 18 reinforced concrete models have been analyzed and the results were given bellow:-

4.2.1 Plastification in Structural Members

Considering sample reinforced concrete (RC) frames and excitations described in this study, a comparison between different retrofitting strategies was made. For this purpose, a series of nonlinear dynamic analysis was performed using three different ground motion records for 18 different cases. These cases cover three storey, six storey, and twelve storey RC unbraced frames (UBFs), conventional braced frames (CBFs), buckling-restrained braced frames (BRBFs), and viscoelastically damped frames (VDFs) with three different damping coefficients of 1000, 2500, and 10000 kNs/m.

In order to address the local deformations of the structural members, the plastifications of the structural members were investigated. Plastic hinge formations and their performance levels for the 6 storey frames considered in this study are shown in Figures 4.22 to 4.27. The notations IO, LS, and CP stand for immediate occupancy, life safety, and collapse prevention performance levels, respectively. The results revealed that especially the buckling restrained braces and the viscoelastic dampers with high damping coefficients are effective in improving seismic response of the structures and limiting the nonlinear behavior in the UBFs.

Comparing the performance of different seismic resisting systems considered in this study, it was observed that BRBs and VDs with damping coefficient 10000 kNs/m (c=10000) offer the best reduction for the plastic hinge rotations in beams and columns for 3, 6 and 12 storey RC frames. Furthermore, VDs with damping coefficient of 2500 kNs/m provided considerable improvement in seismic response of the UBFs by decreasing the plastification demand in critical structural members, but still some CP hinges remain. Generally, this fact holds true independent of the frame height and the exposed earthquake acceleration. For example, the frames upgraded with BRBs remained mainly in the elastic range of deformation except the BRBs. On the other hand, the VDF with c=1000 kNs/m was not effective for upgrading the seismic behavior of the UBFs resisting system. Addition of CBs to the unbraced frames was found to be the least effective in reducing the plastification.



a) b) c)

Figure 4.22 Plastic hinge formations in 6 storey RC UBFs subjected to a) Hector Mine, b) Morgan Hill and c) Erzincan earthquakes

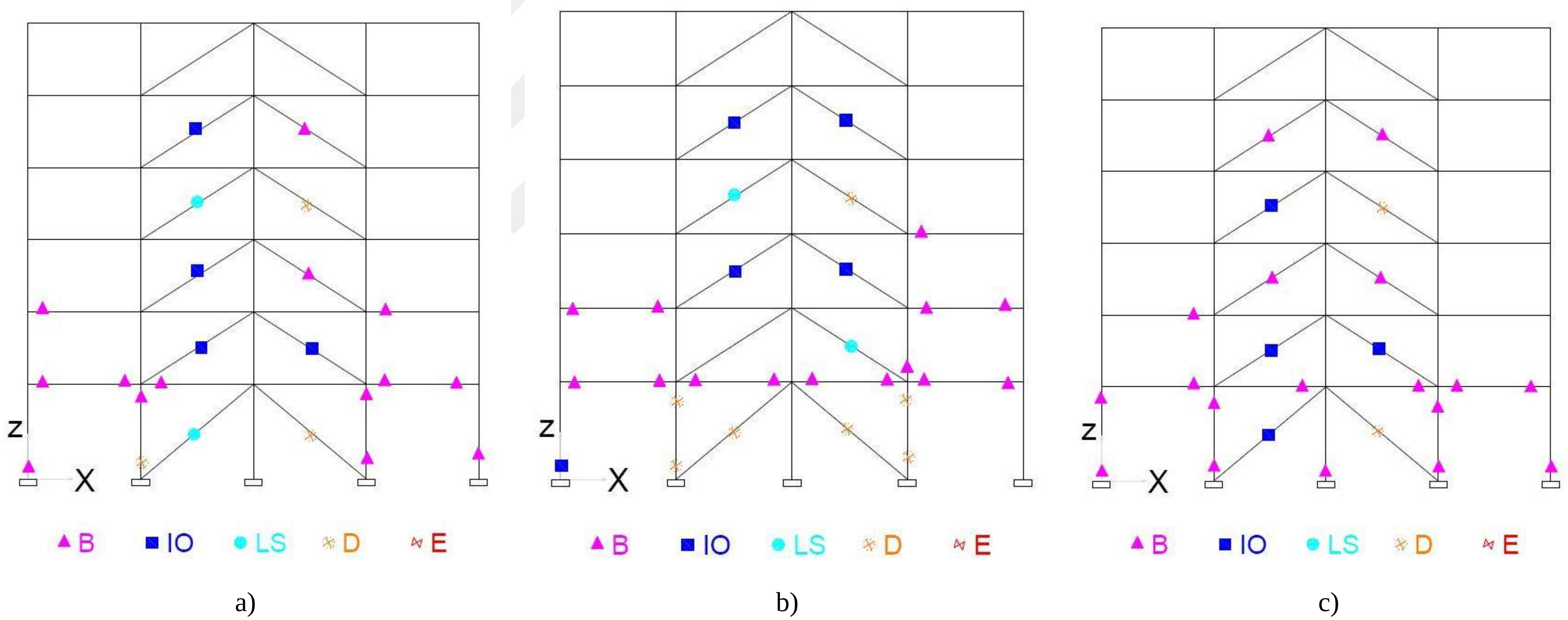


Figure 4.23 Plastic hinge formations in 6 storey RC CBFs subjected to a) Hector Mine, b) Morgan Hill and c) Erzincan earthquakes

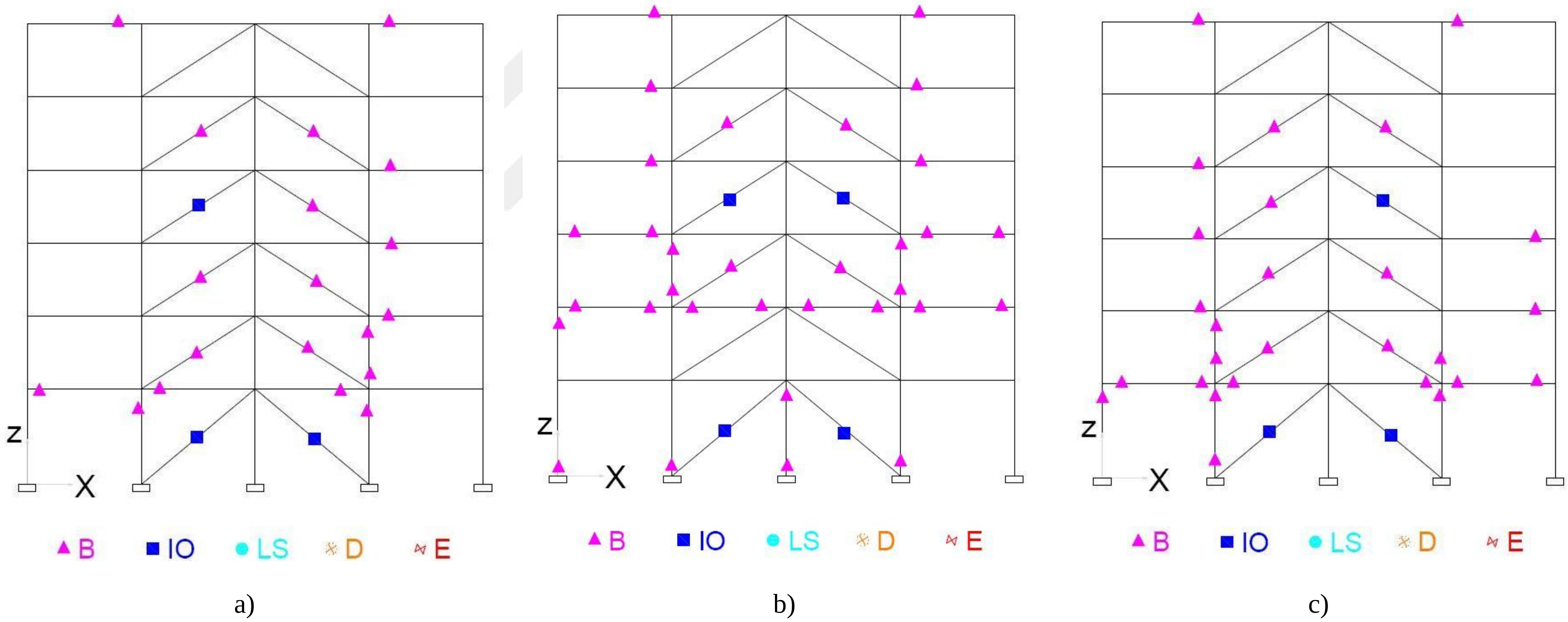


Figure 4.24 Plastic hinge formations in 6 storey RC BRBFs subjected to a) Hector Mine, b) Morgan Hill and c) Erzincan earthquakes

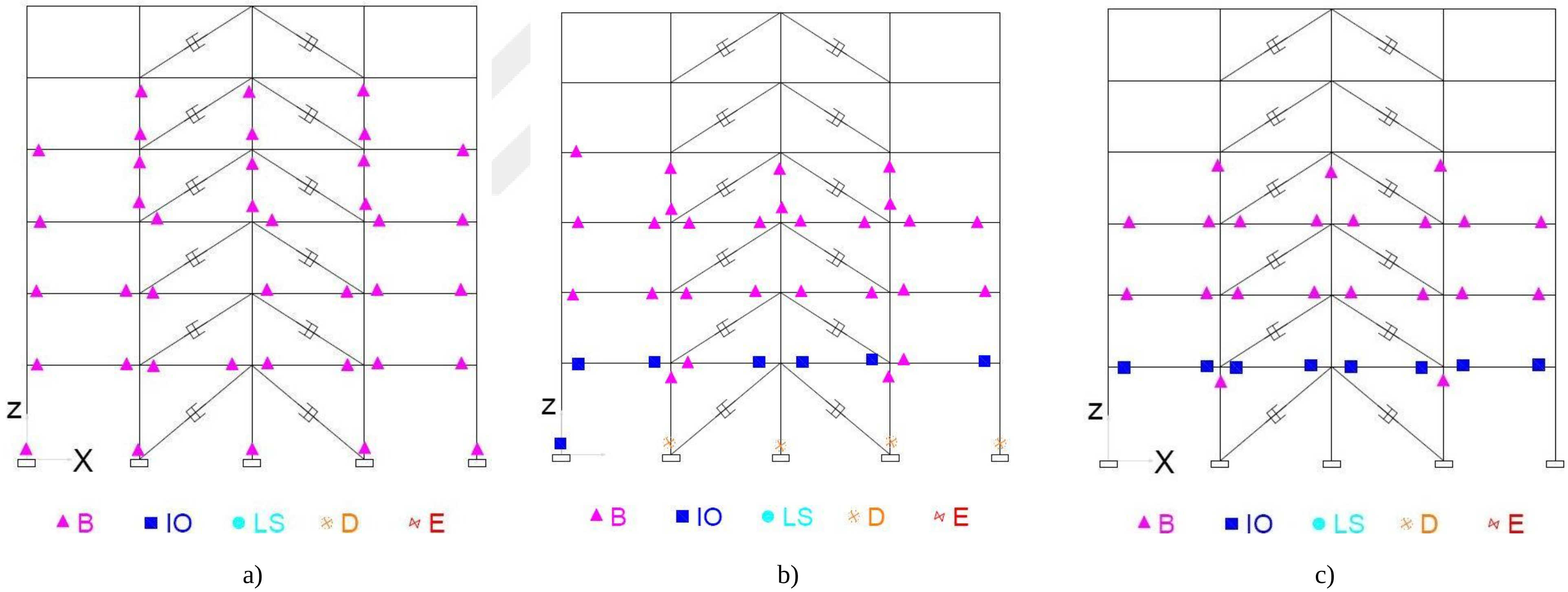


Figure 4.25 Plastic hinge formations in 6 storey RC VDF with c=1000 kNs/m subjected to a) Hector Mine, b) Morgan Hill and c) Erzincan earthquakes

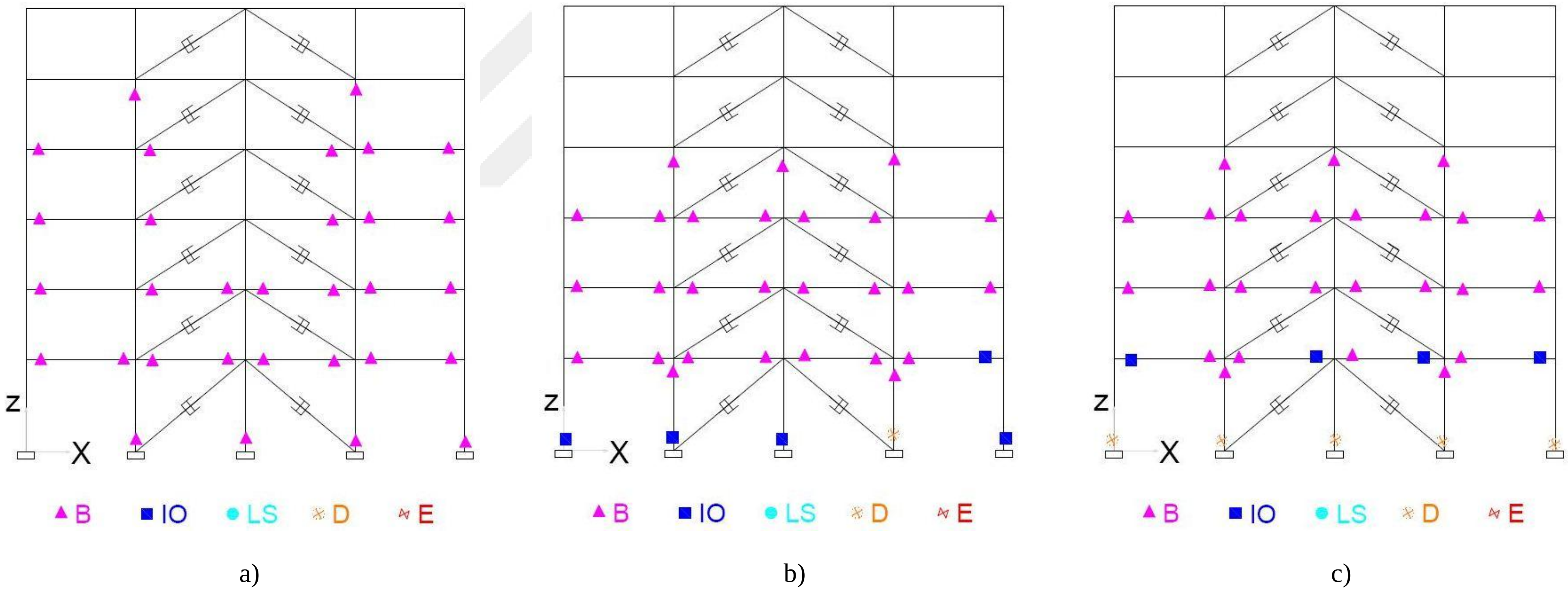


Figure 4.26 Plastic hinge formations in 6 storey RC VDF with c=2500 kNs/m subjected to a) Hector Mine, b) Morgan Hill and c) Erzincan earthquakes

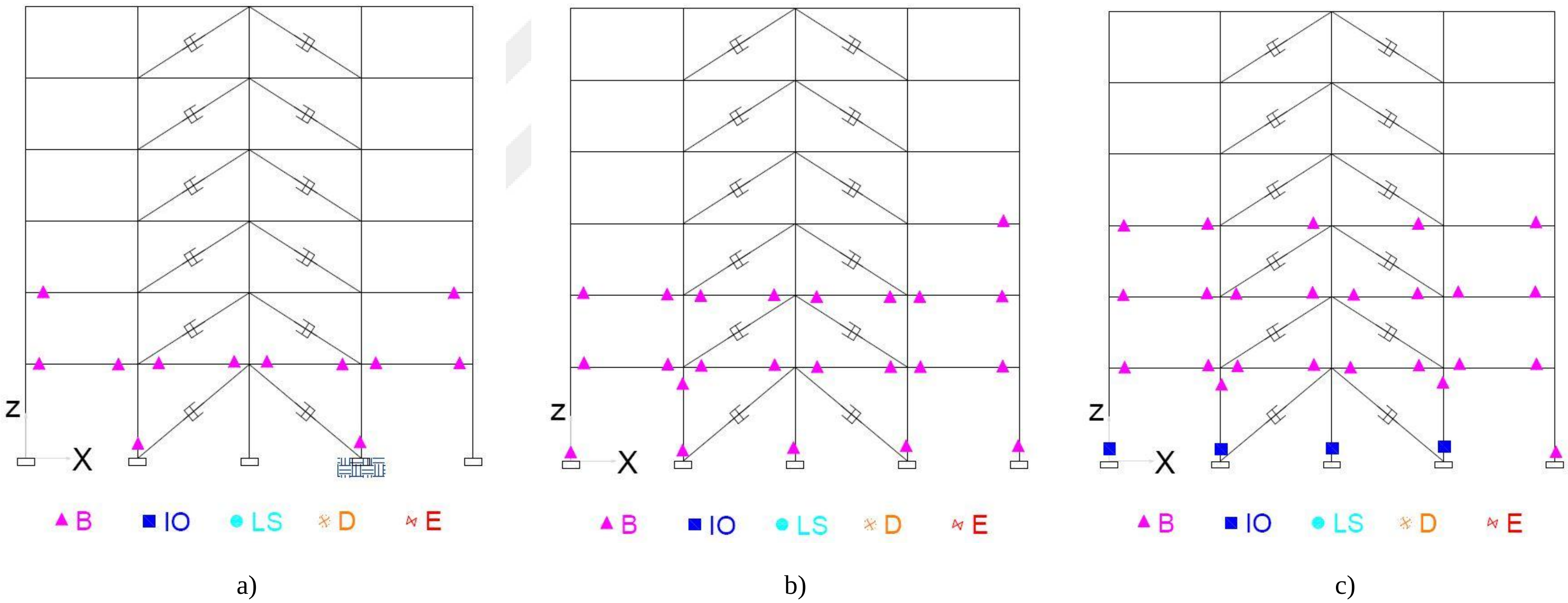


Figure 4.27 Plastic hinge formations in 6 storey RC VDF with c=10000 kNs/m subjected to a) Hector Mine, b) Morgan Hill and c) Erzincan earthquakes

4.2.2 Interstorey Drift Index

As mentioned before that the ratio of the maximum interstorey drift (δ_{max}) to the storey height (h) was defined as "Maximum Interstorey Index". The interstorey index obtained for 3, 6 and 12 storey frames are presented in Figures 4.28, 4.29 and 4.30. It is clear from the figures that generally the frames with passive energy dissipation devices have interstorey index values less than those of the unbraced frames. Among the passive energy dissipation systems utilized, the BRBs and VDs with damping coefficient of c=10000 kNs/m appear to be very effective in reducing the displacement demand of the UBFs. For example, with addition of BRBs, a significant reduction in interstorey drift index values was obtained when subjected to Hector Mine, Morgan Hill and Erzincan earthquakes. The average reduction obtained in the interstorey drift index with respect to the original frame is 60% and 63.5% by the use of BRBs and VDs with damping coefficient of c=10000 kNs/m, respectively.

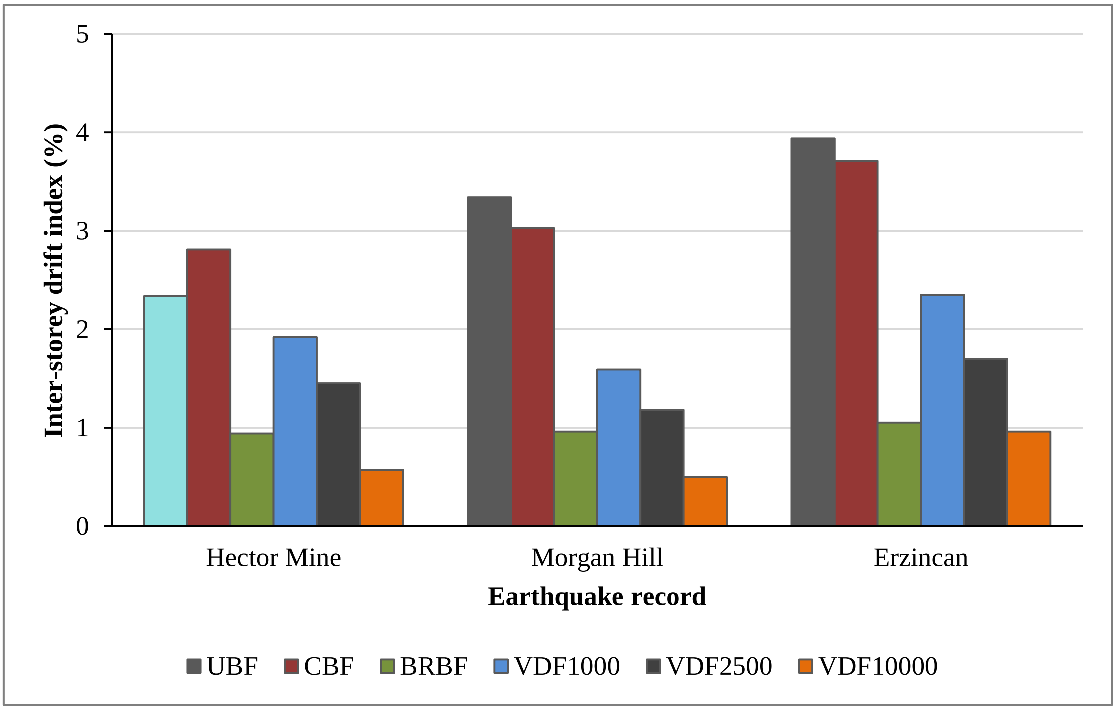


Figure 4.28 Interstorey drift index for 3 storey RC frames

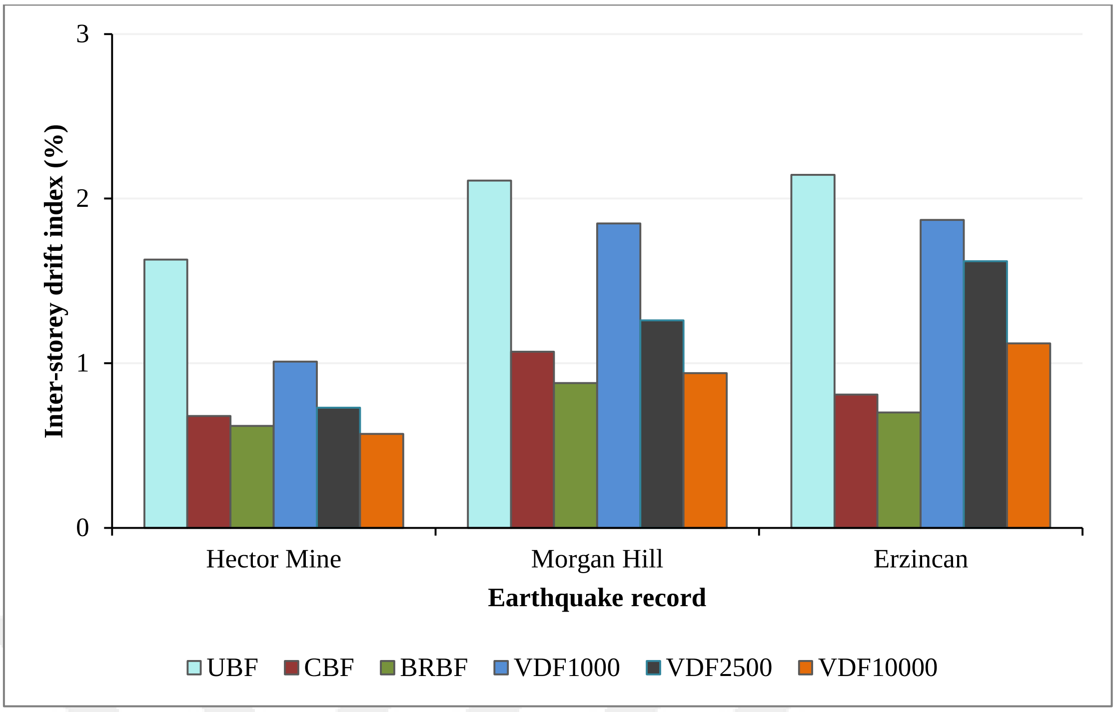


Figure 4.29 Interstorey drift index for 6 storey RC frames

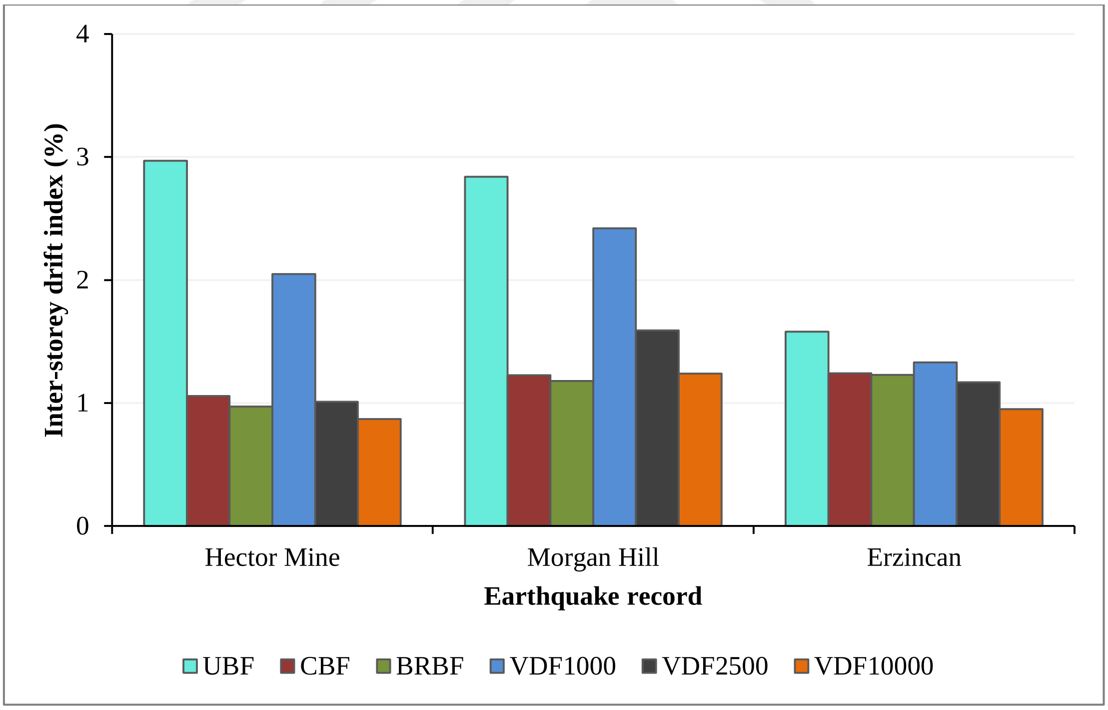


Figure 4.30 Interstorey drift index for 12 storey RC frames

4.2.3 Global Damage Index

As discussed before ratio of the roof displacement to the building height (D/H) was called as “Global Damage Index”. The global damage index obtained for 3, 6 and 12 storey RC frames are presented in Figures 4.31, 4.32 and 4.33. The results clearly showed that using passive energy control devices improve nonlinear behavior of the frames and reduce the damage index. The results also showed that BRBs and VDs with damping coefficient of c=10000 kNs/m appear to be very effective in reducing the displacement demand of the UBFs. The reduction of global damage index with respect to the original frame on average equal to 60% and 58% for BRBFs and VDFs with damping coefficient of c=10000 kNs/m, respectively.

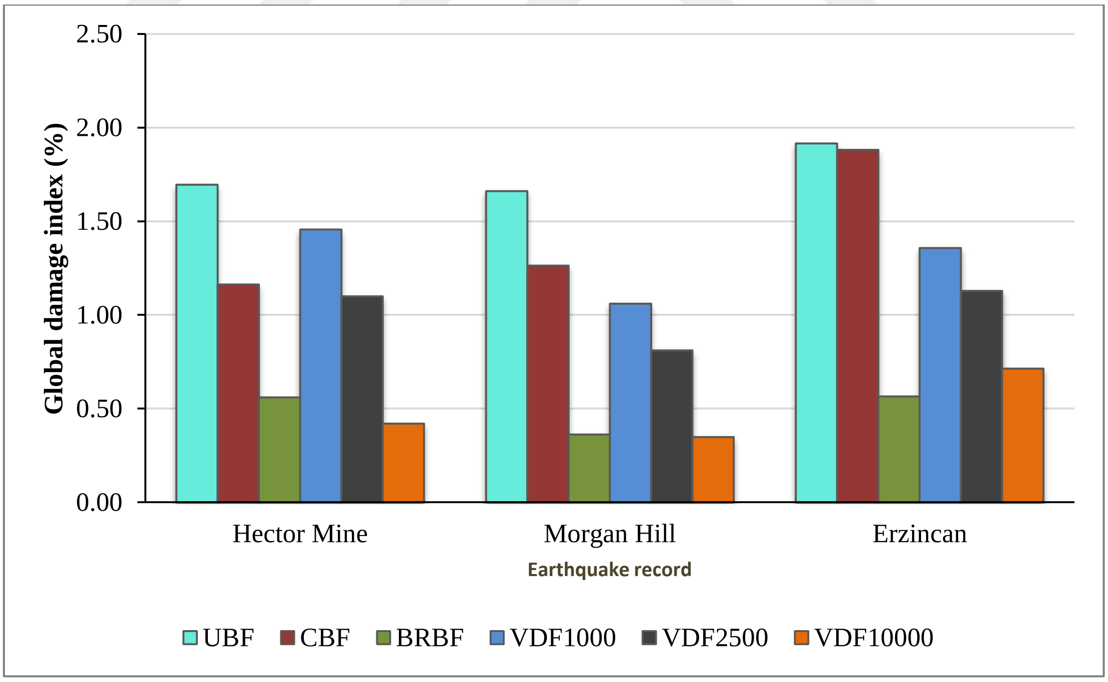


Figure 4.31 Global damage index for 3 storey RC frames

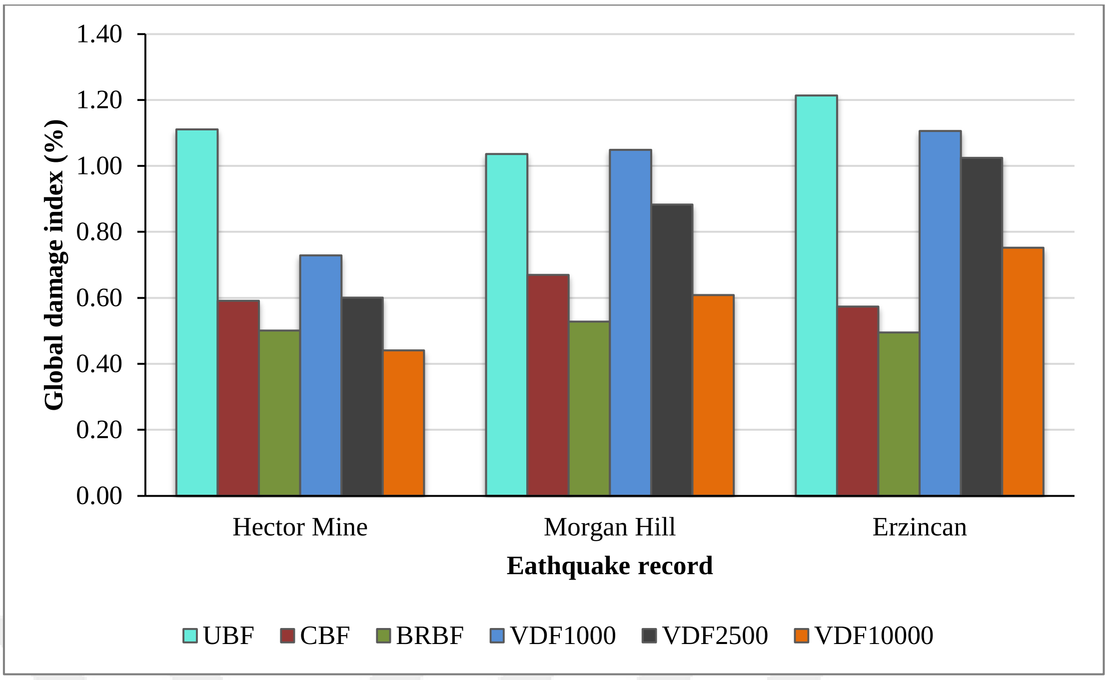


Figure 4.32 Global damage index for 6 storey RC frames

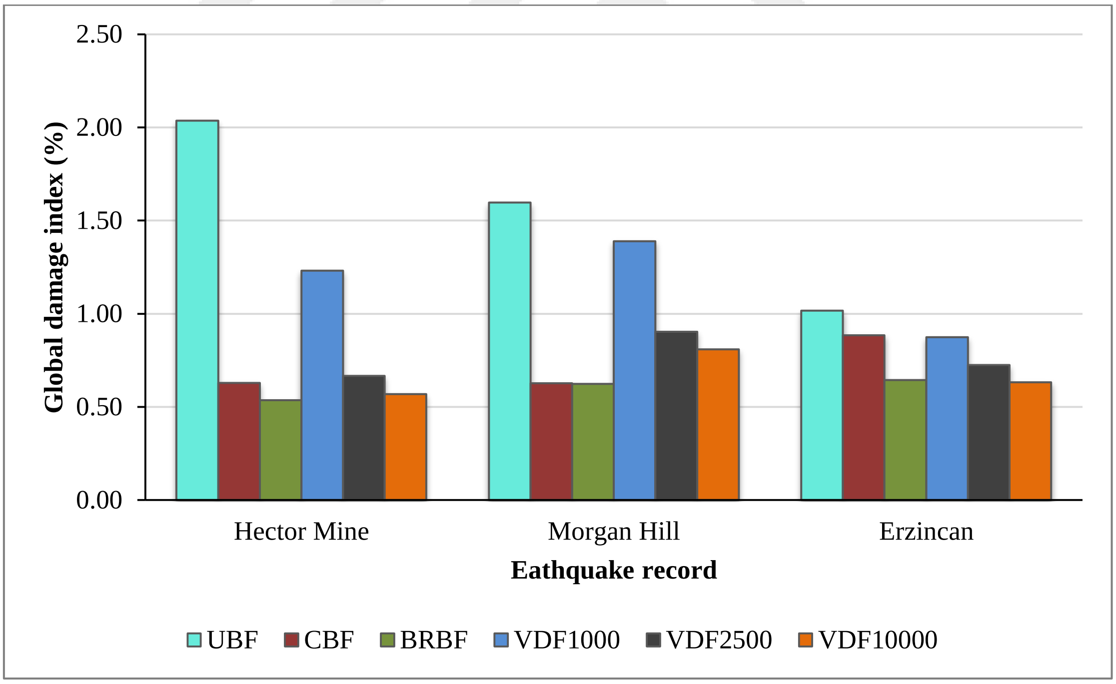
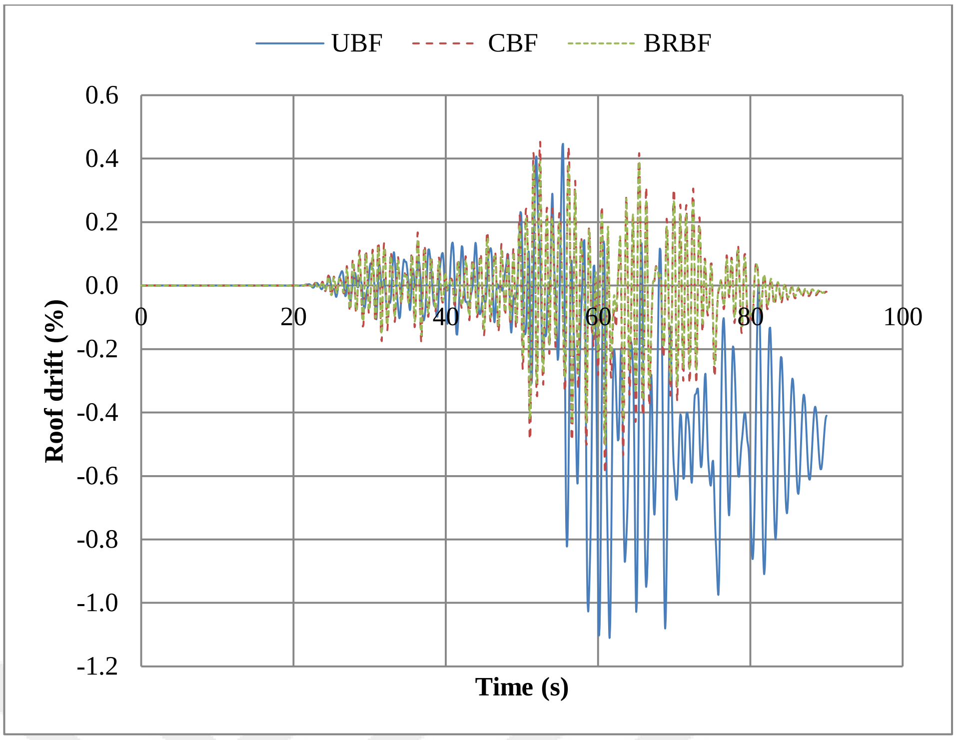


Figure 4.33 Global damage index for 12 storey RC frames

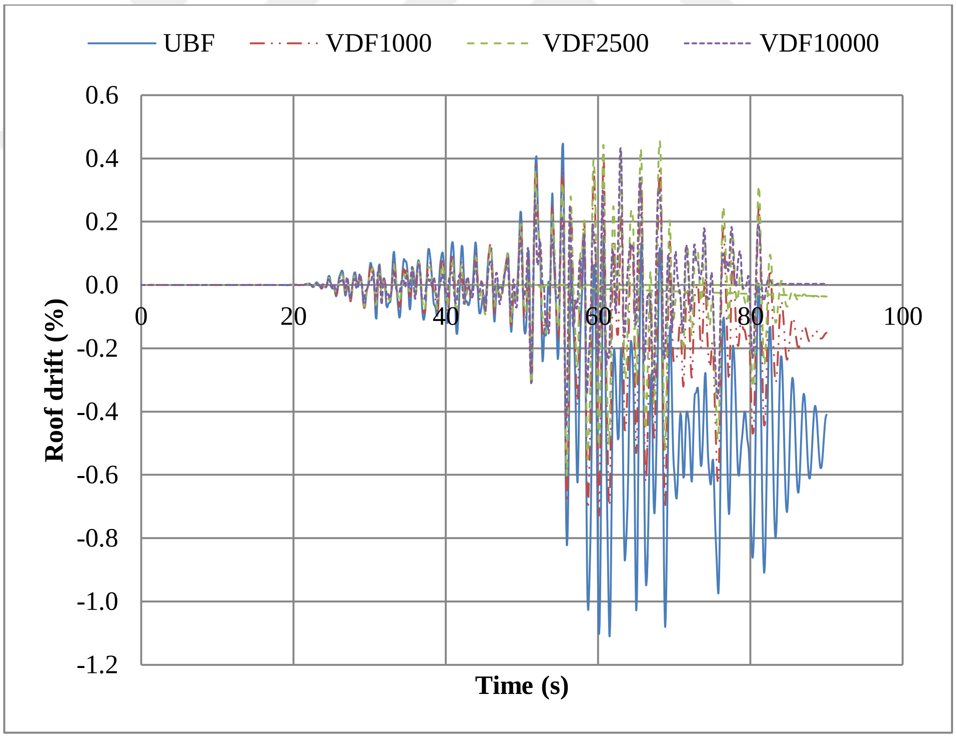
4.2.4 Roof Drift Time History

Figures 4.34, 4.35 and 4.36 provide roof drift time history response of the 6 storey frame before and after addition of brace systems when subjected to Hector Mine, Morgan Hill, and Erzincan earthquake accelerations. Additionally, roof drift time history of 3 and 12 storey frames were presented in Appendix C.

The results of the drift response histories confirmed the beneficial effects of using innovative energy dissipation systems especially BRBs and VDs with high damping coefficient, in reducing the lateral displacements. On contrary, the traditional bracing systems and the VDs with low damping coefficient (c=1000 kNs/m) were not as efficient as the other systems.

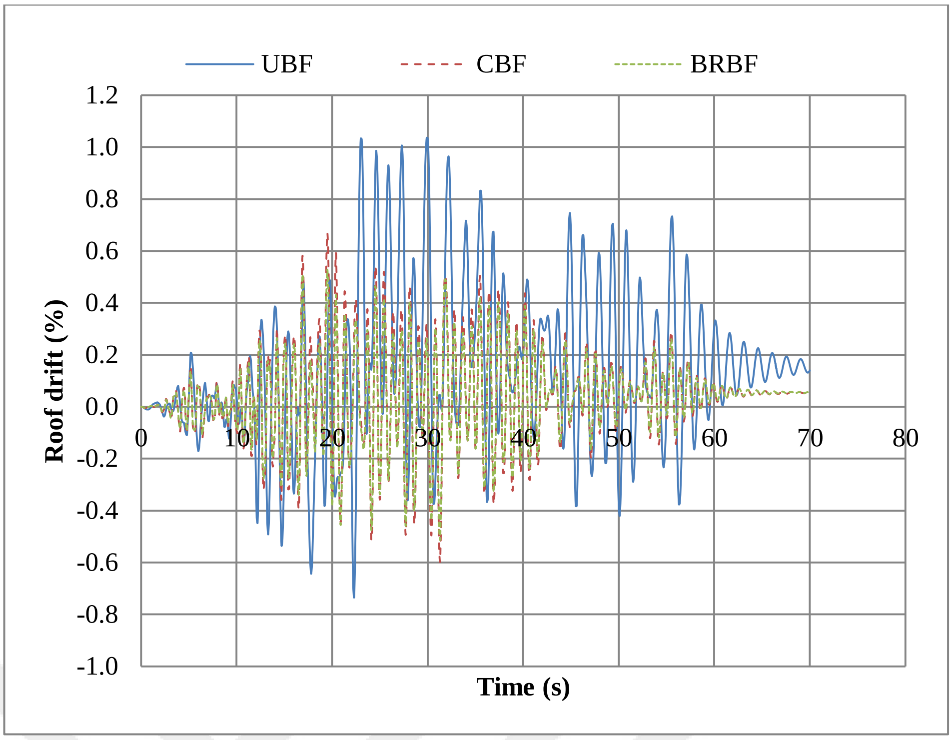


(a)

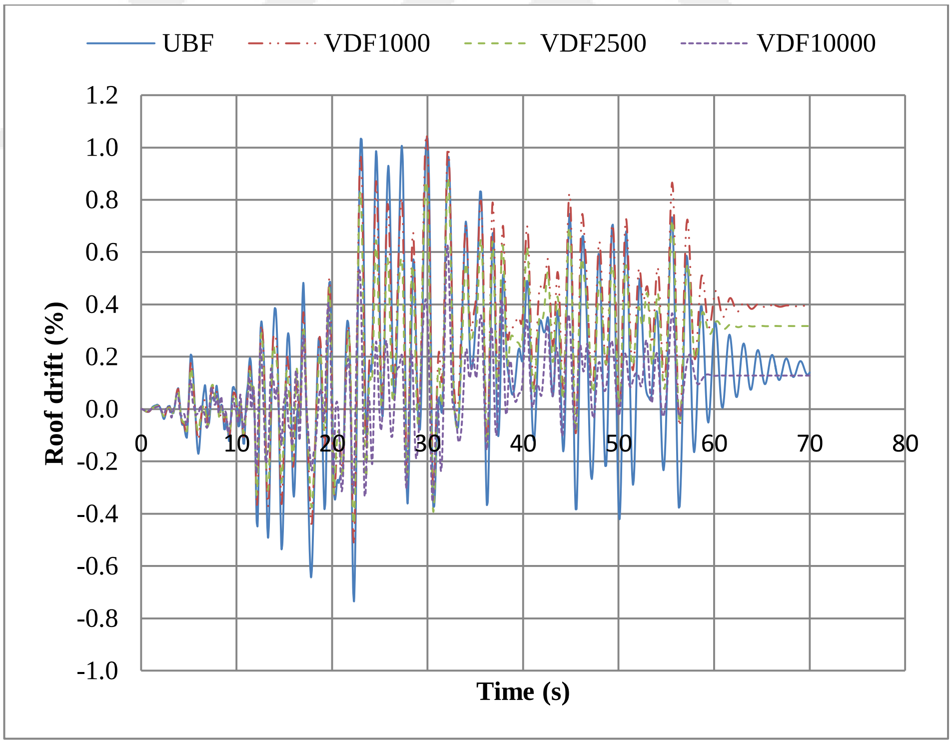


(b)

Figure 4.34 Roof drift time history response of the 6 storey RC a) UBF, CBF and BRBF, b) UBF and VDFs under Hector Mine earthquake.

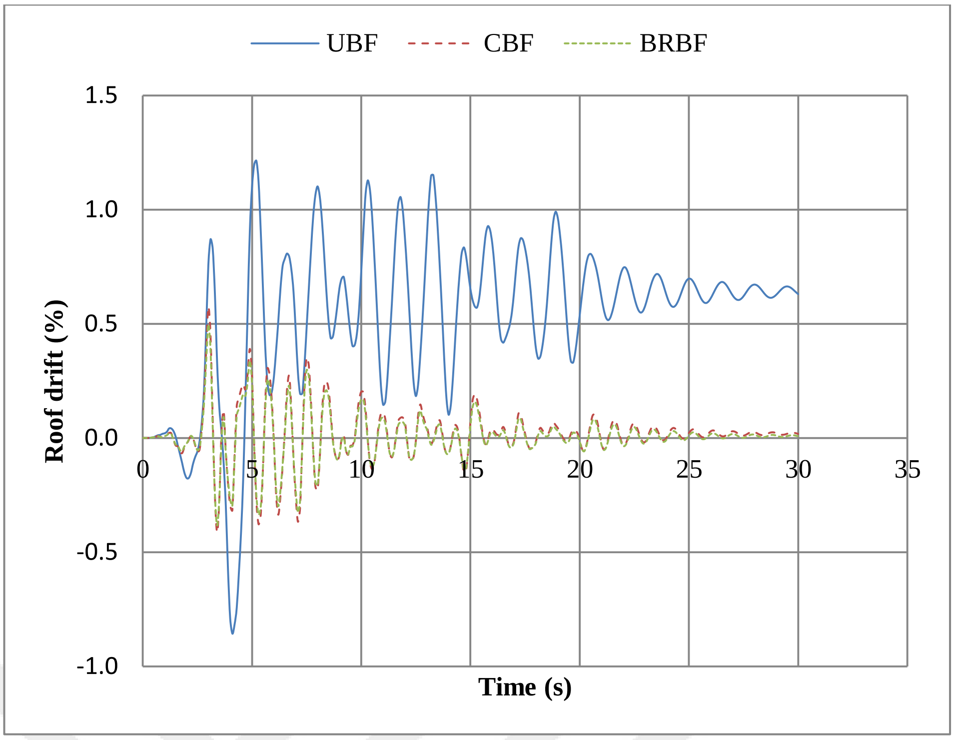


(a)



(b)

Figure 4.35 Roof drift time history response of the 6 storey RC a) UBF, CBF and BRBF, b) UBF and VDFs under Morgan Hill earthquake.



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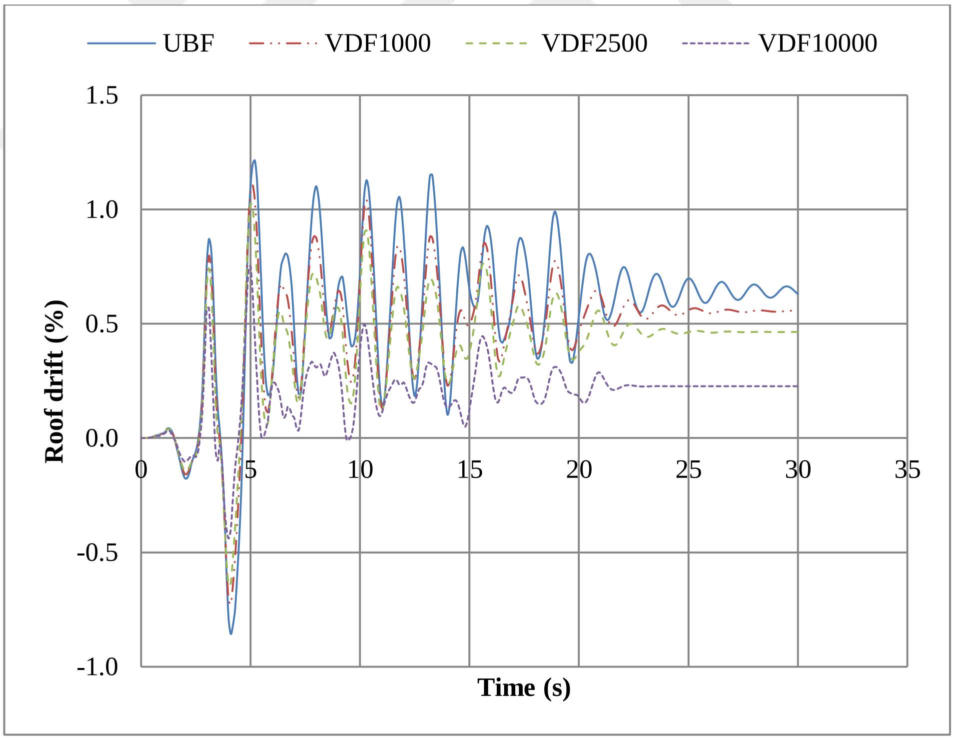
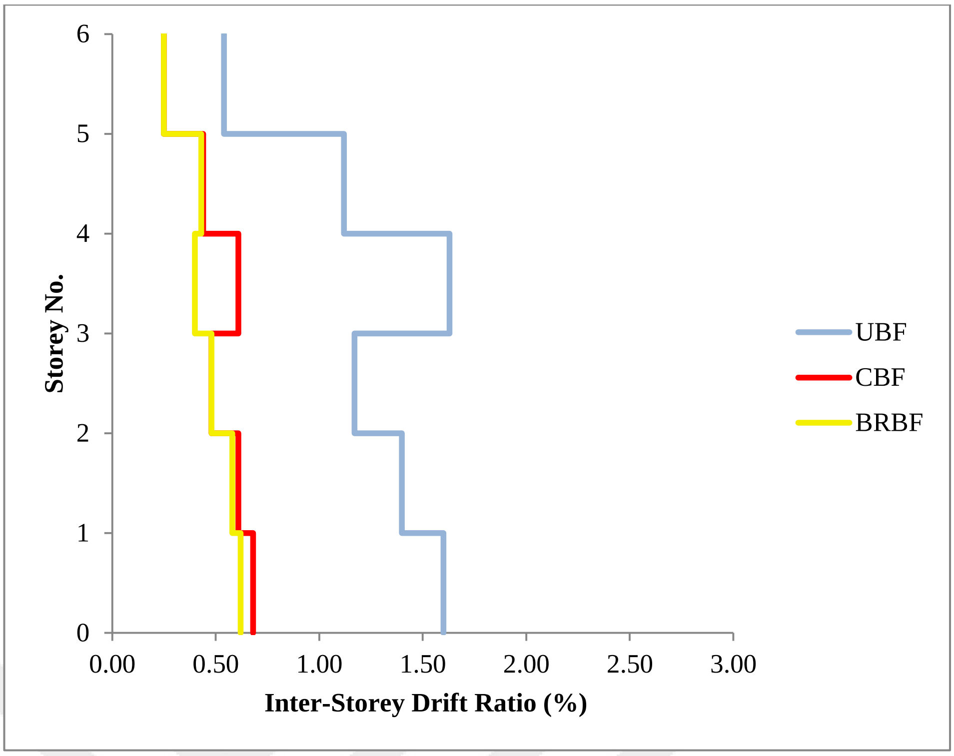


(b)

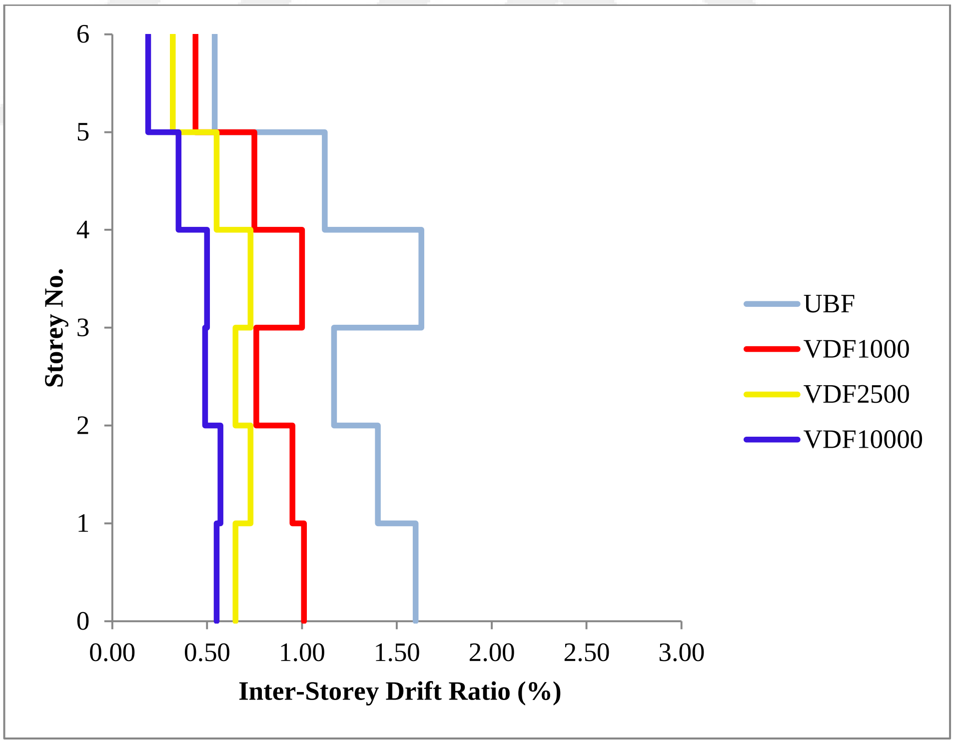
Figure 4.36 Roof drift time history response of the 6 storey RC a) UBF, CBF and BRBF, b) UBF and VDFs under Erzincan earthquake.

4.2.5 Height – Wise Distribution of Peak Storey Drift

Distribution of the drift demands over the height of the structure is important for understanding performance of the frames; therefore it should be evaluated. Figures 4.37, 4.38 and 4.39 show the height-wise distribution of the maximum inter-storey drift demands of the original and upgraded 6 storey frames when subjected to ground excitations of Hector Mine, Morgan Hill, and Erzincan earthquake. In addition to this, the plots regarding the height-wise distribution of the maximum inter-storey drift of 3 and 12 storey RC frames were presented in Appendix D. BRBs and VDF with c=10000 provided a uniform distribution of the interstorey drift demand throughout the height of the structure and also minimized interstorey drift demands. In UBF, CBFs, VDF1000 and VDF2500 there is a concentration of large drift demand in one or more storeys of the frames. Examination of the maximum interstorey drift throughout the height of the viscoelastic damped frames showed that the drift demand in all storeys is effectively controlled for the case with a damping coefficient of c=10000 kNs/m.

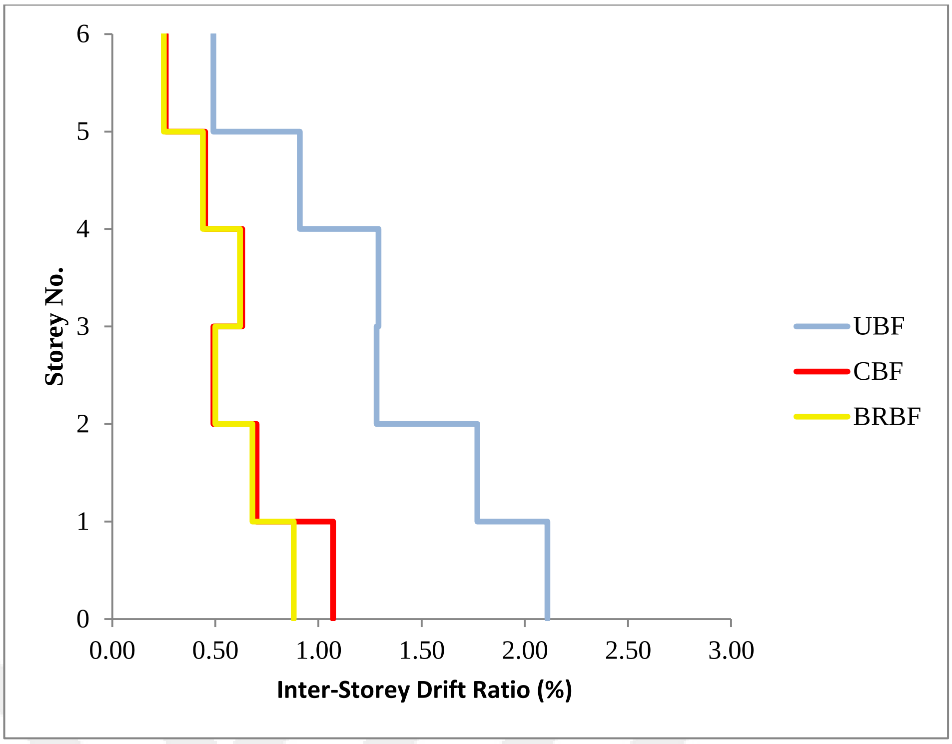


(a)

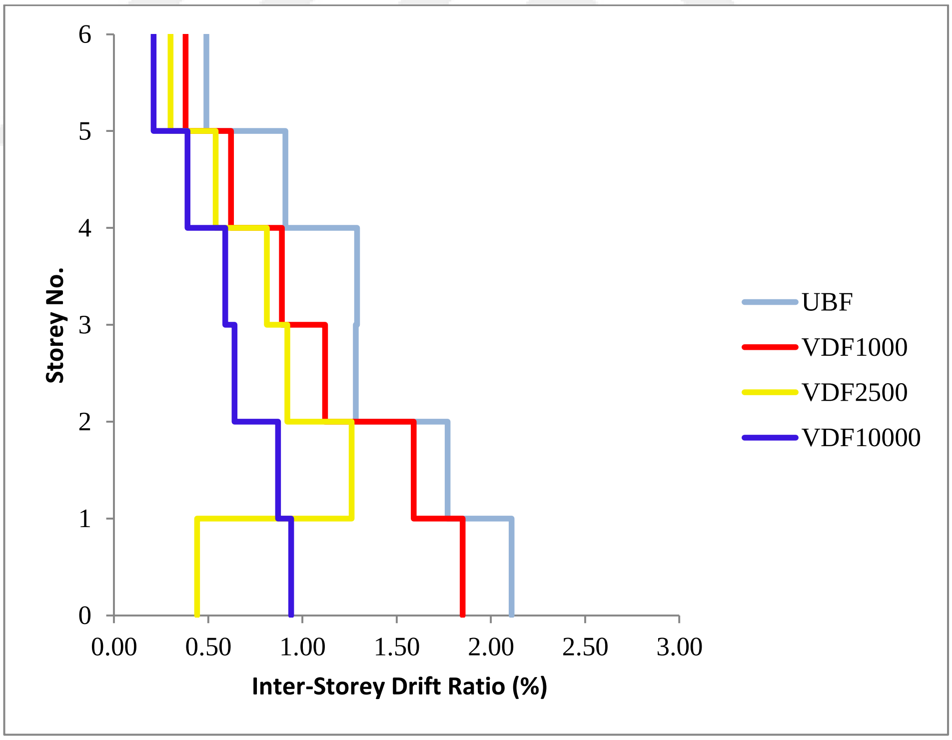


(b)

Figure 4.37 Height – wise distribution of peak storey drift of the 6 storey RC a) UBF, CBF and BRBF, b) UBF and VDFs under Hector Mine earthquake.

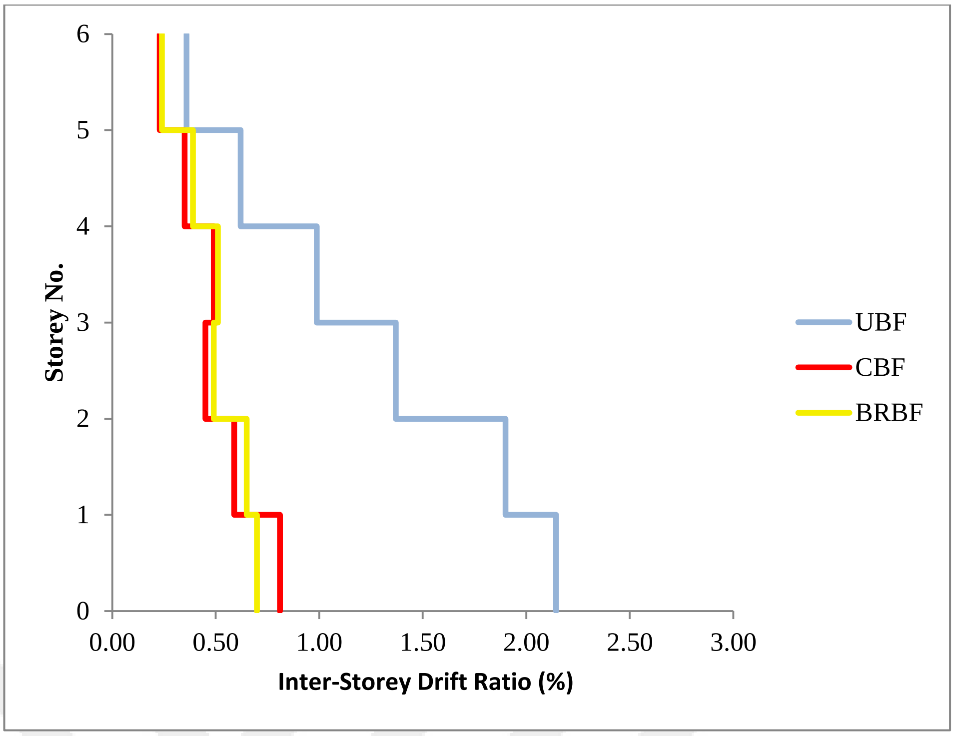


(a)

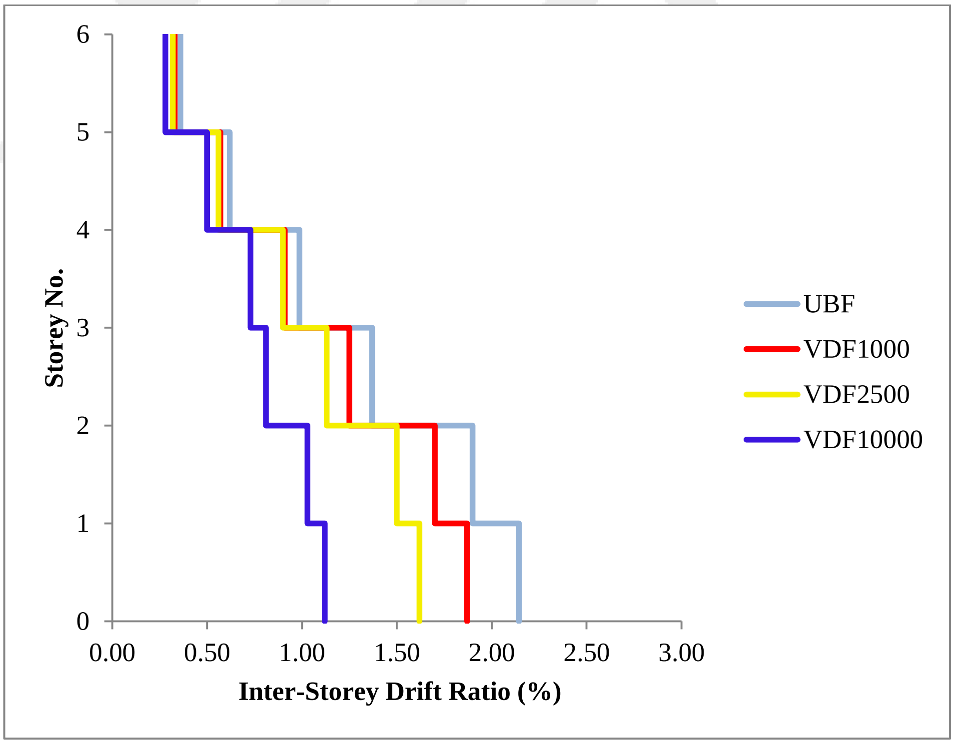


(b)

Figure 4.38 Height – wise distribution of peak storey drift of the 6 storey RC a) UBF, CBF and BRBF, b) UBF and VDFs under Morgan Hill earthquake.



(a)



(b)

Figure 4.39 Height – wise distribution of peak storey drift of the 6 storey RC a) UBF, CBF and BRBF, b) UBF and VDFs under Erzincan earthquake.

4.2.6 Base Shear

The base shear demand of RC UBFs, CBFs, BRBFs and VDFs subjected to three ground motions (Hector Mine, Morgan Hill and Erzincan) are given in Figures 4.40, 4.41 and 4.42 The outcome of the analysis demonstrated that the use of BRBs lead to significant increase in the base shear demand of the original frame. Stiffness of frame, peak ground acceleration, and earthquake type affected the variation of base shear forces. Position of the fundamental period of the frame with respect to earthquake excitation acceleration spectrum defined this variation in the elastic stage. Nevertheless, the base shear distribution was affected significantly by T_s/T_p ratio.

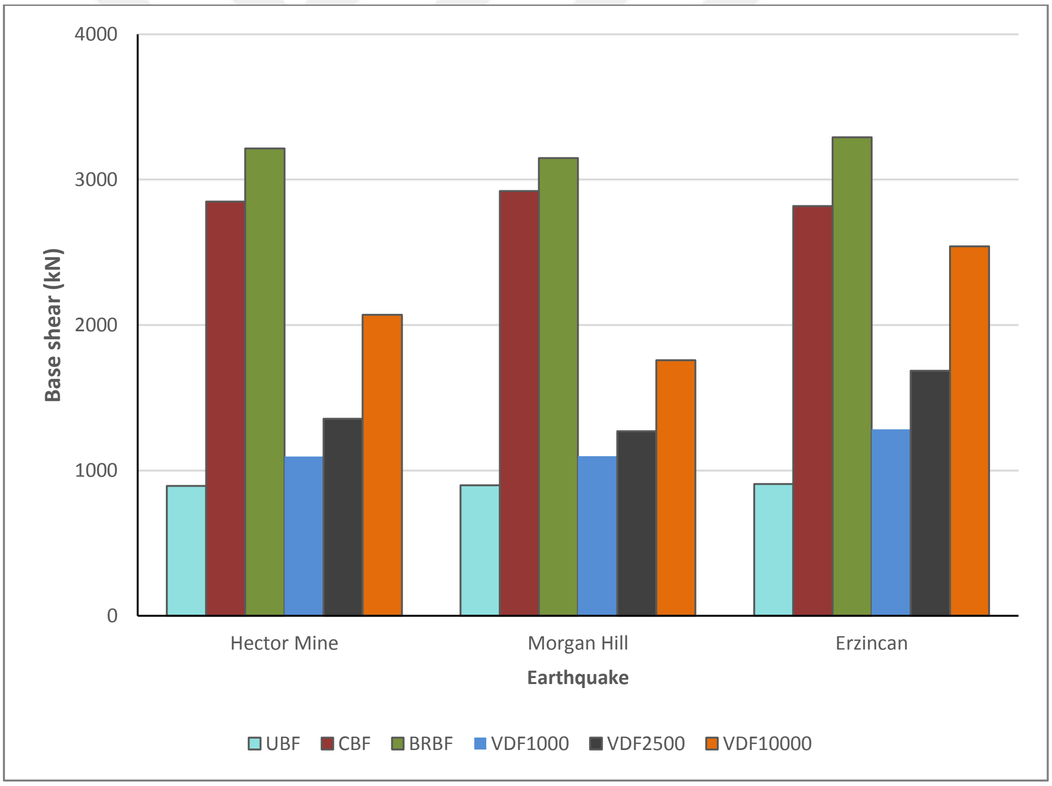


Figure 4.40 Comparison of the maximum base shear demand of 3 storey RC frames

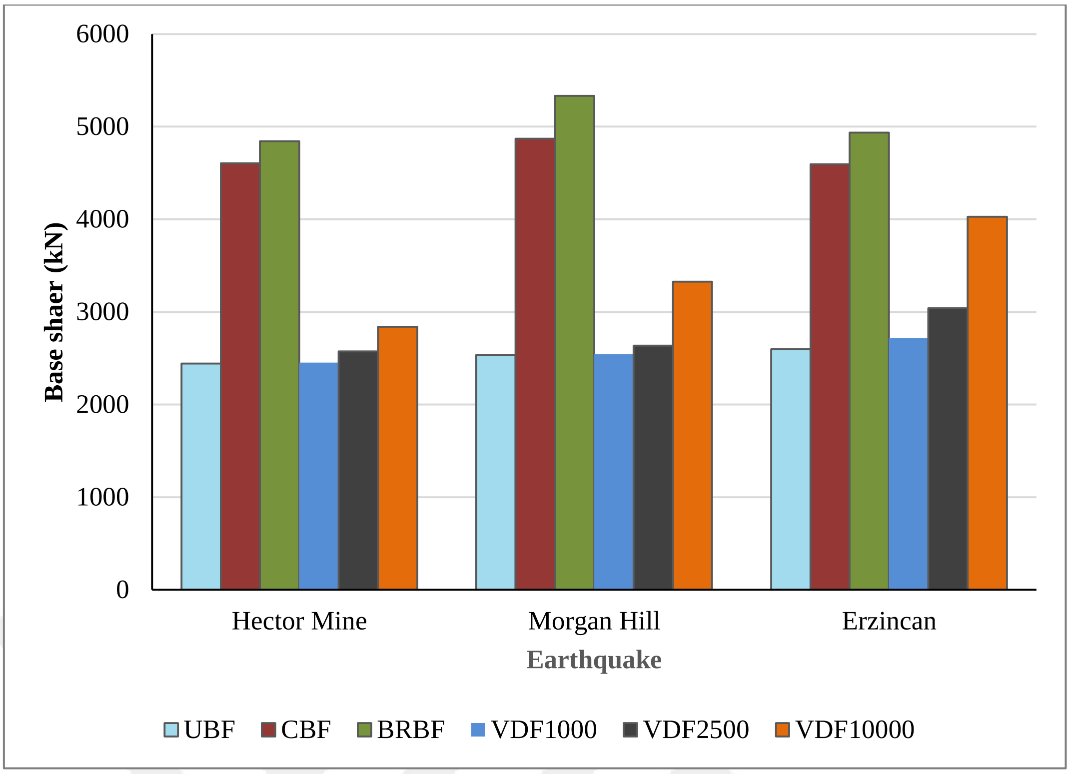


Figure 4.41 Comparison of the maximum base shear demand of 6 storey RC frames

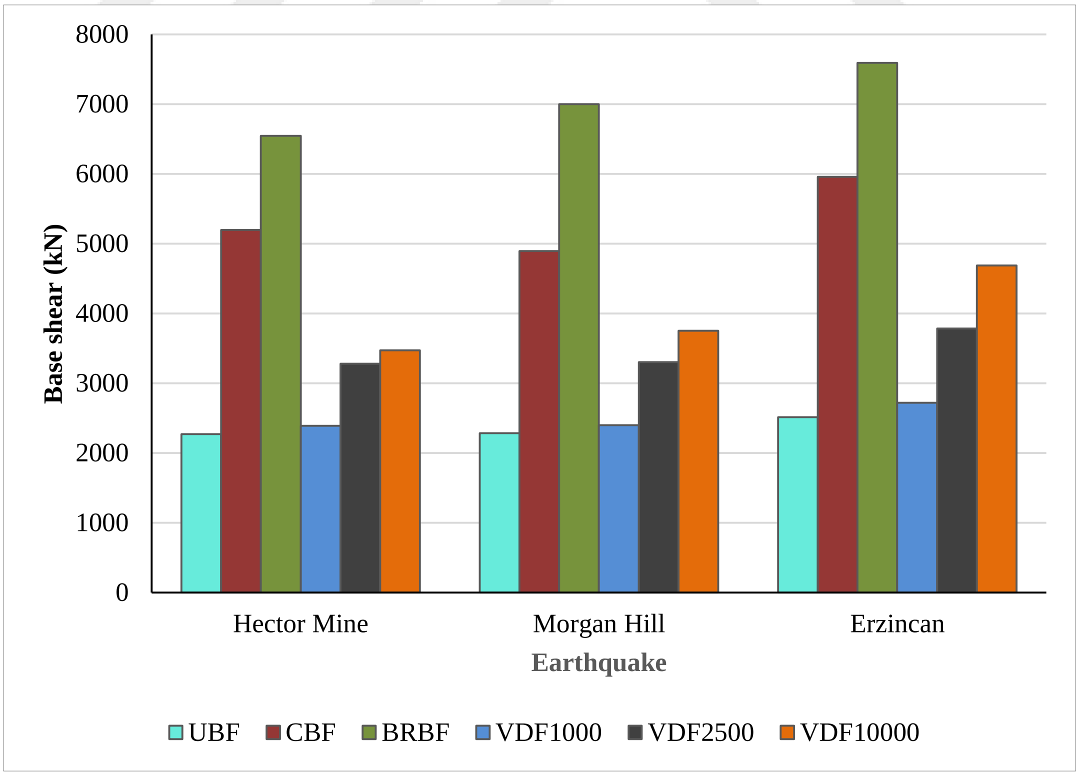


Figure 4.42 Comparison of the maximum base shear demand of 12 storey RC frames

CHAPTER 5

CONCLUSIONS

This study aimed to investigate the seismic performance of steel and reinforced concrete moment resisting frame buildings equipped with diagonal conventional braces (CBs), buckling restrained braces (BRBs), and viscoelastic dampers (VDs) sunder a set of spectrum compatible natural earthquake accelerations. Depending on analysis results, the following conclusions can be drawn:

- In general, it was evident that steel and RC MRFs equipped with VDs with a damping coefficient of c=10000 kNs/m and BRBs kept it in the elastic range of deformation. In the case of the frames with BRBs, the plastification was concentrated in the braces which might be replaced easily after the earthquake.
- Use of VDs with appropriate damping coefficient and BRBs resulted in decrease in displacement demands and enhanced the performance state of the both steel and RC MRFs.
- Using VDs with a damping coefficient of c=10000 kNs/m and BRBs reduced interstorey drift index of steel and RC MRFs. The average reductions in the interstorey drift index were 66.2% and 59% for VDs with c=10000 kNs/m and BRBs, respectively for SMRFs. For RC-MRFs the average reduction in the interstorey drift index were 63.5% and 60%, for VDs with c=10000 kNs/m and BRBs, respectively.

- Using VDs with a damping coefficient of c=10000 kNs/m and BRBs reduced global damage index of steel and RC MRFs. The average reductions in the global damage index were 65.2% and 61.6% for VDs with c=10000 kNs/m and BRBs, respectively for SMRFs. For RC-MRFs the average reduction in the interstorey drift index were 57.3% and 61.12%, for VDs with c=10000 kNs/m and BRBs, respectively.
- Both VDs of damping coefficient c= 10000 kNs/m and BRBs significantly reduced the roof residual drifts of steel and RC MRFs, and that is due to the fact that addition of such controlling systems reduces the plastic hinge formations in the structural members.
- Increasing damping coefficient of VDs led to increasing damping ratio of the devices and improved the seismic performance of steel and RC MRFs.
- During strong and damaging earthquakes, the use of VDs and BRBs as a retrofitting strategy is expected to be a viable solution for steel and reinforced concrete structures. They provide a uniform distribution of the storey drifts throughout the height of the structure.
- Addition of CBs may result in sudden change in drift pattern and concentration of large deformation in one storey due to the buckling deformation of CBs in that storey prior to others.
- Additions of VDs reduced the global deformations with providing base shear demand close to the original frame for both steel and reinforced concrete frames. On contrary, the results of the performed inelastic analyses demonstrate that the use of conventional or buckling restrained bracing systems increased the base shear demand with respect to original frame.

REFERENCES

Abou-Elfath, H. and Ghobarah, A. (2011). Behaviour of reinforced concrete frames rehabilitated with concentric steel bracing. *Canadian Journal of Civil Engineering*, **27**(3), 433-444·

Aiken, I.and Kimura, I. (2001). The Use of Buckling-Restrained Braces in the United States. Proceedings, Japan Passive Control Symposium, Tokyo Institute of Technology, Yokohama, Japan, December.

AISC (2005). seismic provisions for structural steel buildings, American institute of steel construction, Inc., Chicago, IL. AISC (2005b).

Ameen, N. M. (2012). Nonlinear Analysis of Conventional and Buckling Restrained Braced Frames under Near-Field Ground Motions. *M.Sc. thesis*, University of Gaziantep, Gaziantep.

ANSI/AISC 358-05. Prequalified connections for special and intermediate steel moment frames for seismic applications including supplement No. 1, American institute of steel construction, Inc., Chicago, IL.

Armouti N.S. (2013). Effect of Dampers on Seismic Demand of Short Period Structures in Soft Sites. *Research Journal of Applied Science*, *Engineering and Technology*, **5**(6), 2203-2211.

Asgarian, B. and Amirhesari, N. (2008). Comparison of dynamic nonlinear behavior of ordinary and buckling restrained braced frames subjected to strong ground motion. *The Structural Design of Tall and Special Buildings*, **17**(2), 367-386.

Asgarian B. and Shokrgozar, H. R. (2009). BRBF response modification factor. *Journal of Constructional Steel Research*, **65**, 290-298.

ATC 40 (Applied Technology council) (1996). Seismic Evaluation and Retrofit of concrete buildings. **1-2**, California.

Barroso L.R., Breneman S.E., and Smith H.A. (2002). Performance Evaluation of Controlled Steel Frames under Multilevel Seismic Loads. *Journal of Structural Engineering, ASCE*, **128**(11), 1368-1378.

Bianc, V. and Granati, S. (2015). Expeditious seismic assessment of existing moment resisting frame reinforced concrete buildings: Proposal of a calculation method. *Journal of Engineering Structures*, **101**, 715‑732.

Bitaraf M., Hurlebaus S. and Barraso L.R. (2012), “Active and Semi-active Control for Undamaged and Damaged Building Structures Under Seismic Load”, *Computer-Aided Civil and Infrastructure Engineering*, **27**, 48-64.

Black C, Makris N. and Aiken I. A. (2004). Component testing, seismic evaluation and characterization of buckling-restrained braces. *Journal of Structural Engineering ASCE*, **130**(6), 880–94.

Bozorgnia Y. and Bertero VV. (2004). Earthquake engineering. From engineering seismology to performance-based engineering. Boca Raton (FL, USA): CRC Press.

Building Code Requirements for Structural Concrete (ACI 318-08).

Bussell M. (1997). Appraisal of existing iron and steel structures". The Steel Construction Institute, Ascot, England, SCI Publication 138.

Chung, B. T. (1993). The influence of P-Delta effects 011 the seismic design of structures, Dept. of Civil Engineering University of Auckland, Auckland, N.Z.

CSI (2009). CSI Analysis Reference Manual for SAP2000, ETABS and SAFE, Computers and Structures. Inc., Berkeley, CA, USA.

Deulkar W. N., Modhera C. D. and Patil H S (2010). Buckling restrained braces for vibration control of building structure. *International Journal of Research and Reviews in Applied Sciences,* **4**, 363-372.

Di Paola, M. and Fileccia Scimemi, G. (2016). Finite element method on fractional viscoelastic frames. *Journal of Computers and Structures*, **164**, 15–22.

Di Sarno, L. and Elnashai, A.S. (2009). Bracing systems for seismic retrofitting of steel frames. *Journal of Constructional Steel Research*, **65**, 452–465.

Di Sarno, L. and Manfredi, G. (2010). Seismic Response of Reinforced Concrete Buildings Retrofitted with Dissipative Steel Braces. *Journal of Civil Engineering and Architecture*, ISSN 1934-7359, USA, **4**, No.2 (Serial No.27).

El-Hifnawy, L., Abou-Elfath, H., El-Heweity, M. and Mashaly, M. (2005). Ductility Characteristics of Braced Steel Frames Designed According to the Egyptian Code. 11th International Colloquium Structural and Geotechnical Engineering, Cairo-Egypt.

FEMA-273 (1997). NEHRP Guidelines for the Seismic Rehabilitation of Buildings. Federal Emergency Management Agency, Washington (DC).

FEMA-356, (2000). NEHRP Guidelines for the Seismic Rehabilitation of Buildings. Federal Emergency Management Agency, Washington (DC).

Ferrario, F., Iori, F., Pucinott, R. and Zandonini, R. (2016). Seismic performance assessment of concentrically braced steel frame buildings with high strength tubular steel columns. *Journal of Constructional Steel Research*, **121**, 427–440.

Gerb geselischaft fur isolierung mbH and CO KG (1978). Vibration Isolation systems. 13407 Berlin **51**, Roedmallee, 174-176.

Gu, Q., Zona, A., Peng, Y. and Dall'Asta, A. .(2014). Effect of buckling-restrained brace model parameters on seismic structural response. *Journal of Construction and Steel Research*, **98**, 100-113.

Gultekin, A. (2014). Earthquake response of reinforced concrete buildings retrofitted with inverted-V and zipper braces. *M.Sc. thesis*, University of Gaziantep, Gaziantep.

Güneyisi, E. M. (2011). Seismic reliability of steel moment resisting framed buildings retrofitted with buckling restrained braces. Earthquake Engineering and Structural Dynamics. Published on line in Wiley Online Library (wileyonlinelibrary.com). DOI: 10.1002/eqe.1161.

Han, S. W., Ryu, H. W., Kim, W. T. and Lee1, L. H. (2015). Evaluation of energy dissipation capacity of hss bracing members according to the width-thickness ratio. The Ninth East Asia-Pacific Conference on Structural Engineering and Construction.

Heidari, P/ S., Kayhani, H. and Jazany, R. A. (2011). Experimental study on the effects of different concentric bracing configuration on steel frames inelastic behavior. *Australian Journal of Basic and Applied Sciences*, **5**(12), 2716-2728.

Hjelmstad, K. D. and Haikal, G. (2006). Analysis of steel moment frames with deformable panel zones. *Journal of Steel Structures*, **6**, 129-140.

Iwata, M., Kato, T., Wada, A. (2000). Buckling-restrained braces as hysteretic dampers. Proceeding of Behavior of Steel Structures in Seismic Areas. Rotterdam: Balkema.

Jain A.K., Hanson R.D. and Goel S.C. (1980). Hysteretic cycles of axially loaded steel members. *Journal of the Structural Division*, ASCE, **106**(8), 1777–1795.

Jones S. L., Fry G. T. and Engelhardt M. D. (2002). Experimental evaluation of cyclically loaded reduced beam section moment connections. *Journal of Structural Engineering*. **128**(4): pp. 441–451.

Khatib I. F., Mahin S. A. and Pister K. S. (1988). Seismic behavior of concentrically braced steel frames. Report No. UCB/EERC-88/01, Earthquake Engineering Research Center, University of California at Berkeley.

Kiggins, S. and Uang, C. M. (2006). Reducing residual drift of buckling-restrained braced frames as a dual system. *Journal of Engineering Structures*, **28**, 1525–1532.

Kim, J. and Choi, H. (2004). Behavior and design of structures with buckling-restrained braces. *Journal of Engineering Structures*, **26**, pp.693–706.

Kim, J and Choi, H. (2005). Response modification factors of chevron-braced frames. *Journal of Engineering Structures*. **27**, pp. 285–300.

Kim, J., Lee, Y and Choi, Y. (2011). Progressive collapse resisting capacity of braced frames. *The Structural Design of Tall And Special Buildings*, **20**, 257–270.

Kim, J. and Seo, Y. (2004). Seismic design of low-rise steel frames with buckling-restrained braces. *Journal of Engineering Structures*, **26**, 543–551.

Krawinkler, H., and Gupta, A. (1998). Story drift demands for steel moment frame structures in different seismic regions. Proc., 6th National Conf. on Earthquake Engineering, Seattle.

Kumbhar, Y. D. and Shiyekar, M. R. (2014). Study of Buckling Restrained Braces in Steel Frame Building. *Int. Journal of Engineering Research and Applications*. ISSN : 2248-9622, **4**, Issue 8(Version 5), 71-74.

Langenbach, R. and Kelley, J. (1991). Soft Energy Dissipating Design for the Seismic Strengthening of Masonry Infill Frame Midrise Buildings: The Woodrow Hotel, Oakland. Conference on the Seismic Strengthening of Historic Buildings Proceedings, San Francisco.

Lewandowski, R., Bartkowiak, A. and Maciejewski, H. (2012). Dynamic analysis of frames with viscoelastic dampers: a comparison of damper models. *Structural Engineering and Mechanics*, **41**, No. 1, 113-137.

Maddala, P. (2013). Pushover analysıs of steel frames. M Sc. Thesis, Natıonal Instıtute of Technology, Rourkela, India.

MagarPatil, H. R. and Jangid, R..S. (2015). Development and analysis of passive hybrid energy dissipation system for steel moment resisting frame. *International Journal of Civil and Structural Engineering*, **5**, No. 3.

Maheri, M. R. and Sahebi, A. (1997). Use of steel bracing in reinforced concrete flames. *Journal of Engineering Structures*, **19**, No. 12, 1018-1024.

Mahmoudi, M. , Zaree, M. (2010). Evaluating response modification factors of concentrically braced steel frames. *Journal of constructional steel research*, **66,** 1196-1204.

Marino, E. M., Nakashima, M. (2006). Seismic performance and new design procedure for chevron-braced frames. *Earthquake Engineering and Structural Dynamics*, **35**(4), 433-452.

Marshall, J. D. and Charney, F. A. (2009). Dynamic Response of Steel Moment-Frame Structures with Hybrid Passive Control System. Conference on Improving the Seismic Performance of Existing Buildings and other Structures, *ATC and SEI 2009*, San Francisco, California, December 9-11, 1022-1033.

Massumi, A., Abbas, Tasnimi, A. and Saatcioglu, M. (2004). Prediction of seismic over-strength in concrete moment resisting frames using incremental static and dynamic analyses. 13th World Conference on Earthquake Engineering, Vancouver, B.C., Canada August 1-6, Paper No. 2826.

Mazza, F. and Vulcano, A. (2013). Nonlinear seismic analysis to evaluate the effectiveness of damped braces designed for retrofitting RC framed structures. *International Journal of Mechanics*, **7,** Issue 3.

Min, K. W., Kim, J. and Lee, S. H. (2004). Vibration tests of 5-storey steel frame with viscoelastic dampers. *Journal of Engineering Structures*, **26**, 831-839.

Molaei, A. (2014). Seismic Retrofit of Reinforced Concrete Frames with Diagonal Prestressing Cables. *M. Sc. thesis*, University of Ottawa, Canada.

Mortezaeia, A. and Zahrai, S. M. (2009). Seismic response of reinforced concrete building with viscoelastic damper under near field earthquake. *Asıan Journal of Civil Engineering (Building And Housing)*, **9**, No. 3, 347-359.

Munshi, J. A. (1997). Effect of hysteretic response concrete elements. *Journal of Engineering Structures*, **19**, No. 11, 921-935.

Munshi, J. A. and Kasai, K. (1995). Seismic Retrofit of Moment Resisting Frame with Viscoelastic Dampers. ETH Zürich, Rämistrasse 101, 8092 Zürich, Schweiz, www.library.ethz.ch.

Nakamura, N., Suzuki, T., Kasai, K. and Ooki, Y. (2011). Nonlinear Response Analysis Method of Viscoelastic Damper Considering Temperature, Frequency, and Strain Dependencies. Proceedings of the 8th International Conference on Structural Dynamics, EURODYN, Leuven, Belgium, July 4-6.

Özçelik, R. (2011). Seismic upgrading of reinforced concrete frames with structural steel elements. PH.D. thesis, Middle East Technical University, Ankara.

Özel, A. E. and Güneyisi, E. M. (2011). Effects of eccentric steel bracing systems on seismic fragility curves of mid-rise R/C buildings: A case study. *Journal of Structural Safety*, **33**, 82‑95.

PEER, (2011). Users manual for the PEER Ground motion database application.

Phung, N. D. and Plumier, A. (2010). Retrofitting and upgrading reinforced concrete moment resisting frame by using expanded metal panels. 14th European Conference on Earthquake Engineering (ECEE) in Macedonia.

Popov, E. E. (2001). Connections the EERI oral history. By Earthquake Engineering Research Institute and the Regents of University of California.

Qiang, X. (2005). State of the art of buckling-restrained braces in Asia. *Journal of Constructional Steel Research*, **61**(6), 727–748.

Rai, D. C. and Goel, S. C. (2003). Seismic evaluation and upgrading of chevron braced frames. *Journal of Constructional Steel Research*, **59**(8), 971-994.

Ravi Kumar, G., Satish Kumar, S.R. and Kalyanaraman, V. (2007). Behaviour of frames with Non-Buckling bracings under earthquake loading. *Journal of Constructional Steel Research*. **63**, 254–262.

Remennikov, A. M. and Walpole W. R. (1997). Analytical prediction of seismic behavior for concentrically-braced steel systems. *Earthquake Engineering and Structural Dynamics*, **26**(8): 859–874.

Ricles, J. M., Mao, C., Lu, L. W. and Fisher, J. W. (2002). Inelastic cyclic testing of welded unreinforced moment connections. *Journal of Structural Engineering*, **128**(4): 429–440.

Rodgers, J. E. and Mahin, S. A. (2004). Effects of Connection Hysteretic Degradation on the Seismic Behavior of Steel Moment-Resisting Frames. PEER Report 2003/13, Pacific Earthquake Engineering Research Center, College of Engineering, University of California, Berkeley.

Roeder, C. W. (1989). Seismic behavior of concentrically braced frame. *Journal of Structural Engineering ASCE*, **115**(8): 1837–1856.

Ronald, O. H., Helmut. K., James, O. M. and Scott, M. A., NEHRP (2009). Seismic design technical brief No.2, seismic design of steel special moment frames. National institute of standards and technology (NIST).

Roy Becker, S. E. (1995). Seismic design of special concentrically Braced steel frames. Structural Steel Educational Council, Technical Information and Product Service, November.

Saatcioglu M., Mitchell D., Tinawi R., Gardner R. J., Gillies A. G., Ghobarah A., Anderson D. L. and Lau, D. (2001). The August 17, 1999, Kocaeli (Turkey) earthquake-damage to structures. *Canadian Journal of Civil Engineering*, **28**(4), 715-737.

Sabelli, R., Mahin, S. and Chang, C. (2003). Seismic demands on steel braced frame buildings with buckling restrained braces. *Journal of Engineering Structures*, **25**, 655–666.

Saeki, E., Maeda, Y., Nakamura, H.,, Midorikawa, M. and Wada, A. (1995). Experimental study on practical-scale unbonded braces. *Journal of Structure Construction Engineering*, AIJ, **476**, 149–158.

Sangle, K. K., Bajoria, K .M. and Mhalungkar, V. (2012). Seismic analysis of high rise steel frame building with and without bracing. 15th World Conference on Earthquake Engineering.

SEAOC (1999). Recommended Lateral Force Requirements and Commentary. Structural Engineers Association of California, Sacramento, California.

Sezen, H., Whittaker, A. S., Elwood, K. J. and Mosalam, K. M. (2003). Performance of reinforced concrete buildings during the August 17 1999 Kocaeli, Turkey earthquake, and seismic design and construction practice in Turkey. *Journal of Engineering Structures*, **25**(1), 103-114.

Shen, K. L. and Soong, T. T., Chang, K. C. and Lai, M. L. (1995). Seismic behaviour of reinforced concrete frame with added viscoelastic dampers. *Journal of Engineering Structures*, **17**, No. 5, 372-380.

Tafheem, Z. and Khusru, S. (2013). Structural behavior of steel building with concentric and eccentric bracing: A comparative study. *International Journal of Civil and Structural Engineering*, **4**, No 1.

TEC-2007 (2007). Specifications for Buildings to be Built in Earthquake Regions. Ministry of Public Works and Settlement, Government of the Republic of Turkey.

Teran-Gilmore, A. and Ruiz-Garcia, J. (2011). Comparative seismic performance of steel frames retrofitted with buckling-restrained braces through the application of force-Base and Displacement-Based approaches. *Journal of Soil Dynamic and Earthquake Engineering*, **31**, 478-490.

Tezcan, S. S. and Uluca, O. (2003). Reduction of earthquake response of plane frame buildings by viscoelastic dampers. *Journal of Engineering Structures*, **25**, 1755-1761.

Torunbalci, N. (2014). Seismic isolation and energy dissipating systems in earthquake resistant design. 13th World Conference on Earthquake Engineering, Vancouver, B.C., Canada, August 1-6, Paper No. 3273.

Tremblay, R. (2002). Inelastic seismic response of steel bracing members. *Journal of Constructional Steel Research*, **58**(5–8), 665–701.

Tremblay, R., Bolduc, P., Neville, R. and DeVall , R. (2006). Seismic testing and performance of buckling restrained bracing systems. *Canadian Journal of Civil. Engineering*, **33**, 183–198.

Tremblay, R., Filiatrault, A., Timler, P. and Bruneau, M. (1995). Performance of steel structures during the 1994 Northridge earthquake. *Canadian Journal of Civil Engineering*, **22**(2), 338–360.

Tremblay, R. and Robert, N. (2001). Seismic performance of low- and medium-rise chevron braced steel frames. *Canadian Journal of Civil Engineering*, **27**, 1192-1206.

Uang C. M. and Bertero, V. V. (1986). Earthquake simulation tests and associated studies of a 0.3-scale model of a 6-story concentrically braced steel structure. Report No. UCB/EERC-86/10, *Earthquake Engineering Research Center*, University of California at Berkeley.

Uang, C. M., and Bertero, V. V. (1988). Implications of recorded earthquake ground motions on seismic design of building structures. Report No. UCB/EERC-88/13, *Earthquake Engineering Research Center*, University of California, Berkeley.

Uniform Building Code (1997). International Conference of Building Officials, Whittier, Calif.

Varghese, V. and Borkar, Y. R. (2013). Comparative study of S.M.R.F. building over O.M.R.F. building with seismic and wind effect. *International Journal of Engineering Research and Applications (IJERA),* ISSN: 2248-9622, **3**, Issue 3, 1501-1503.

Whittaker, A. S., Aiken, I. D., Bergman, D., Clark, P. W., Cohen, J., Kelly, J. M. and Scholl, R. E. (1993). Code requirements for design and implementation of passive energy dissipation systems. *Proc. ATC-17–1* Seminar on Seismic Isolation, Passive Energy Dissipation, and Active Control, **2**, ATC, Redwood City, Calif., 497–508.

Whittaker, A. S., Uang, C. M. and Bertero, V. V. (1990). An experimental study of the behavior of dual steel systems. Report No. UCB/EERC-88/14, *Earthquake Engineering Research Center*, University of California at Berkeley.

Yakut, A., Gülkan, P., Bakır, S. and Yılmaz, M. T. (2005). Re-examination of damage distribution in Adapazarı: Structural considerations. *Journal of Engineering Structures*, **27**(7), 990-1001.

Yang C.S.W., DesRoches R. and Leon R.T. (2010). Design and analysis of braced frames with shape memory alloy and energy-absorbing hybrid devices. *Journal of Engineering Structures*, **32**, 498-507.

Youssef, M. A. , Ghaffarzadehb, H. and Nehdia, M. (2007). Seismic performance of RC frames with concentric internal steel bracing. *Journal of Engineering Structures*, **29**, 1561–1568.

Youssef, M. A., Mashaly, M. E. and Abou-Elfath, H. (2010). Use of SMA and buckling restrained braces to reduce seismic residual deformations in low-rise RC frames. Proceedings of the 9th U.S. National and 10th Canadian Conference on Earthquake Engineering, July 25-29.

Zandi, Y., Sabetahd, R., Jafarzadeh, K. and Mahdizadeh, M. B. (2012). Evaluation of Performance of Viscoelastic Dampers in Reduction Seismic Base

Shear Structures in Near-fault Earthquakes Using Nonlinear Time-history Analysis. *Journal of Basic and Applied Scientific Research*, **2**(2), 1538-1545.

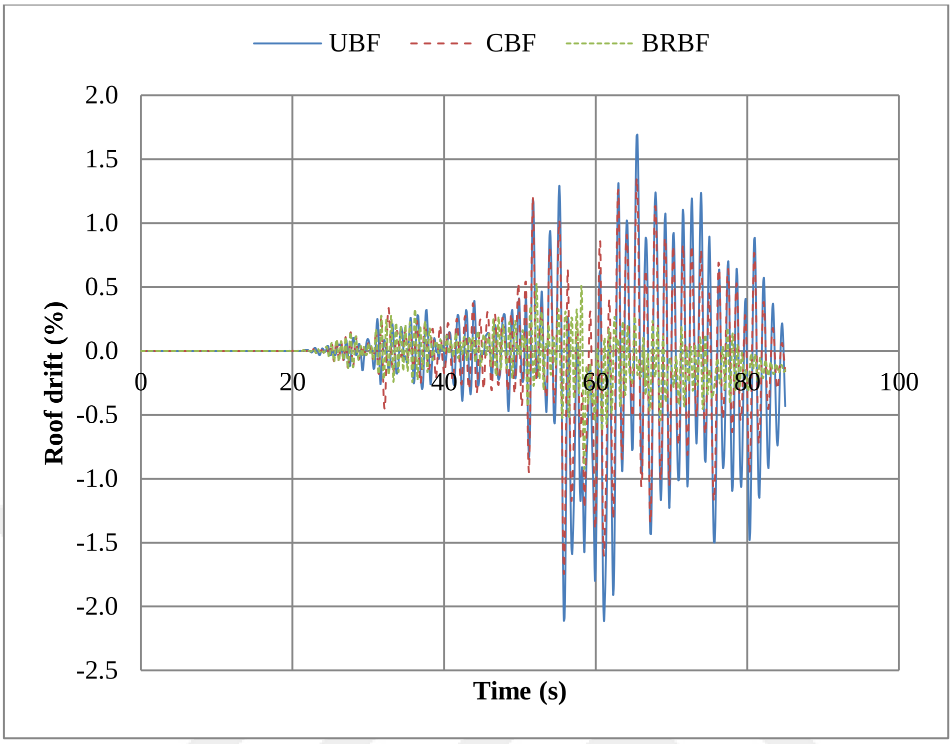
Zhao, J., Wu, B., Li, W. and Ou, J. (2014). Local buckling behavior of steel angle core members in buckling-restrained braces: cyclic tests, theoretical analysis, and design recommendations. *Journal of Engineer Structure*, **66**, 129-145.

Zhao, J., Wu, B. and Ou, J. (2012). Effect of brace end rotation on the global buckling behavior of pin-connected buckling-restrained braces with end collars. *Journal of Engineering Structures*, **40**, pp.240–253.

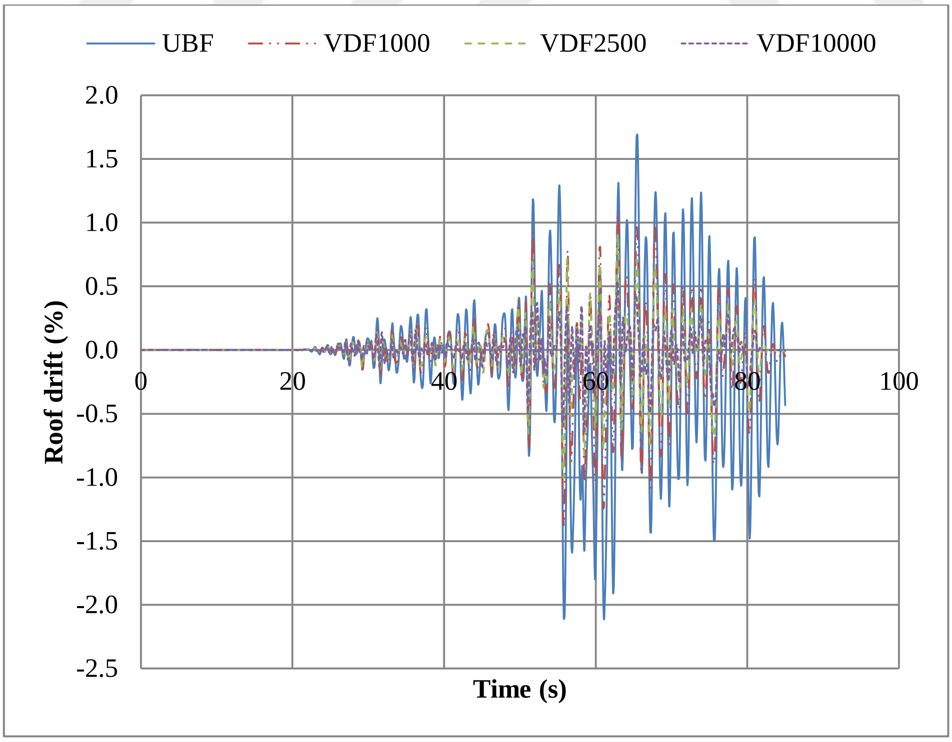
Zhe Qu, Kishiki, S., Maida, Y., Sakata, H. and Wada, A. (2015). Seismic responses of reinforced concrete frames with buckling restrained braces in zigzag configuration. *Journal of Engineering Structures*, **105**, 12–21.

APPENDIX

Appendix A: Roof Drift Time History for 3 and 12 Storey SMRFS

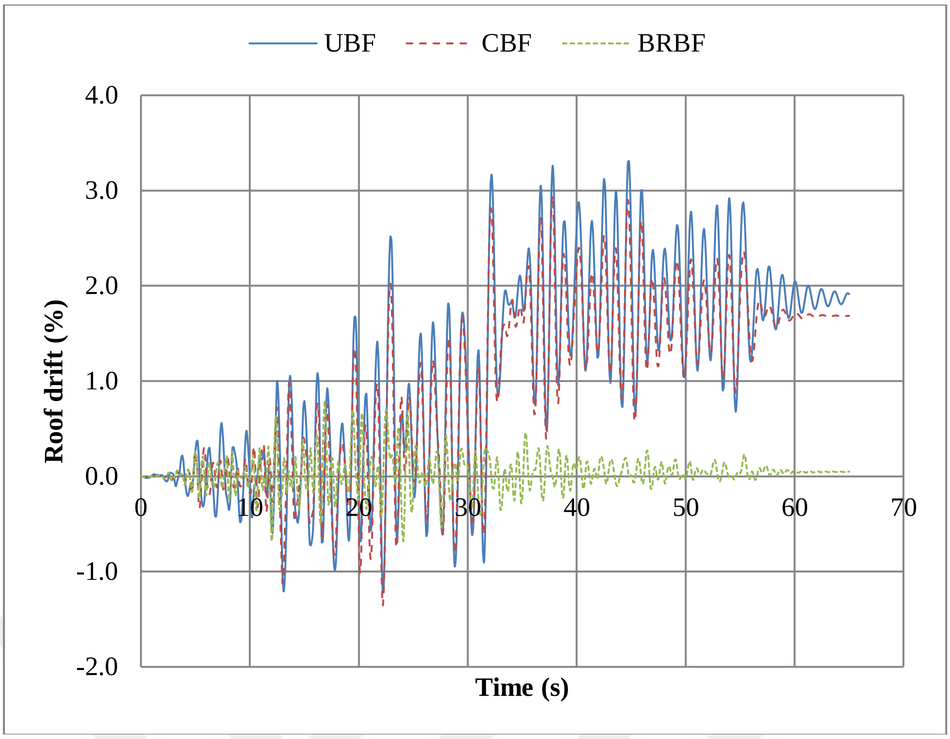


(a)

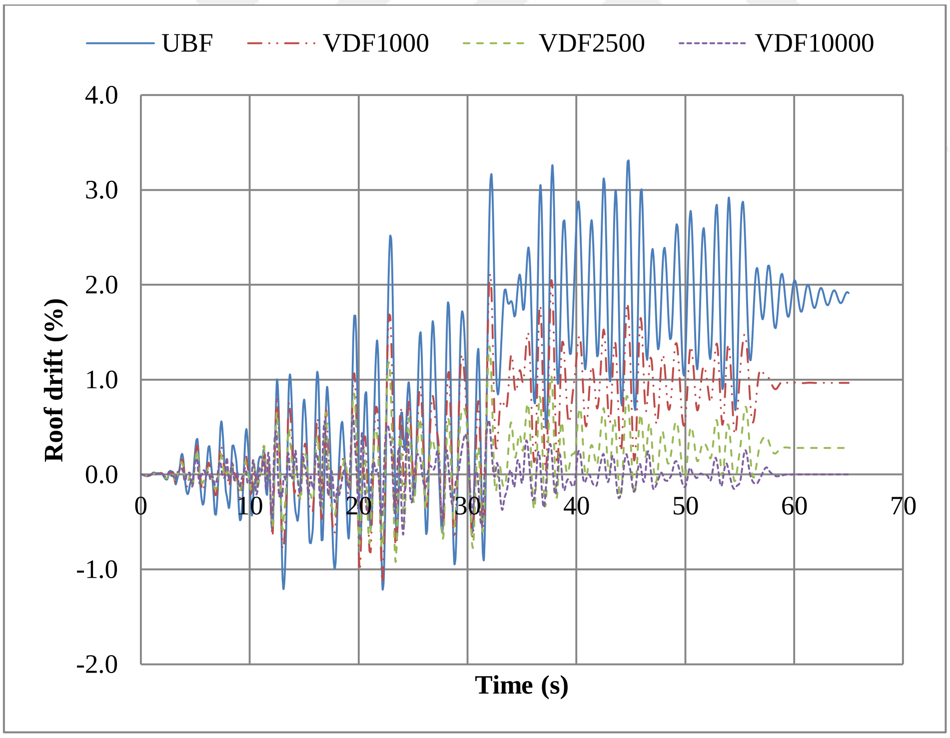


(b)

Figure A1 Roof drift time history response of the 3 storey steel a) UBF, CBF and BRBF, b) UBF and VDFs under Hector mine earthquake.

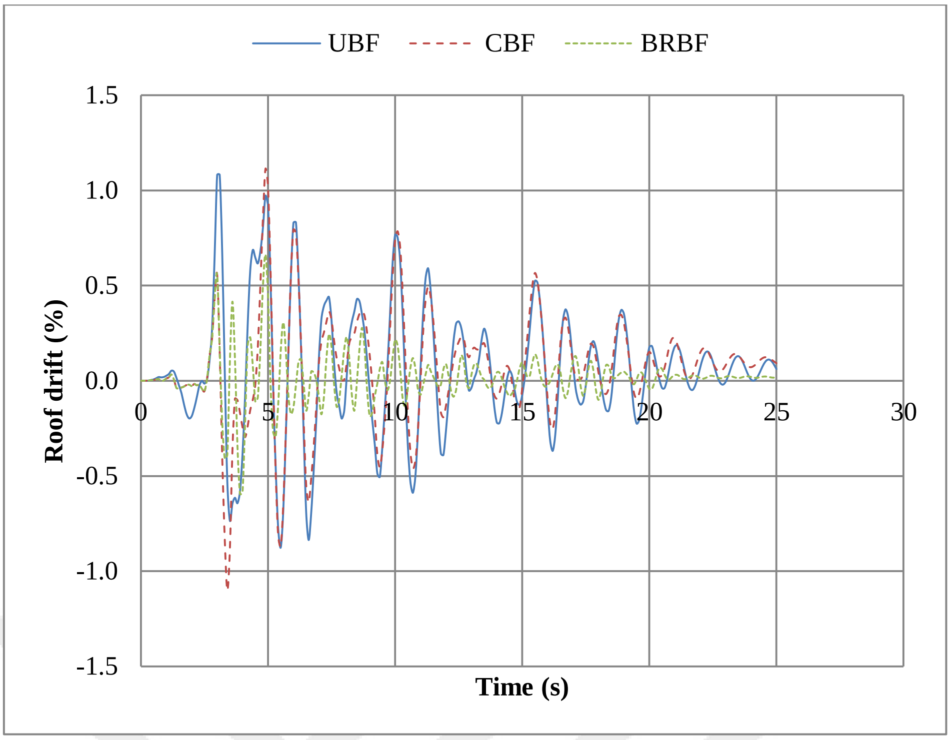


(a)

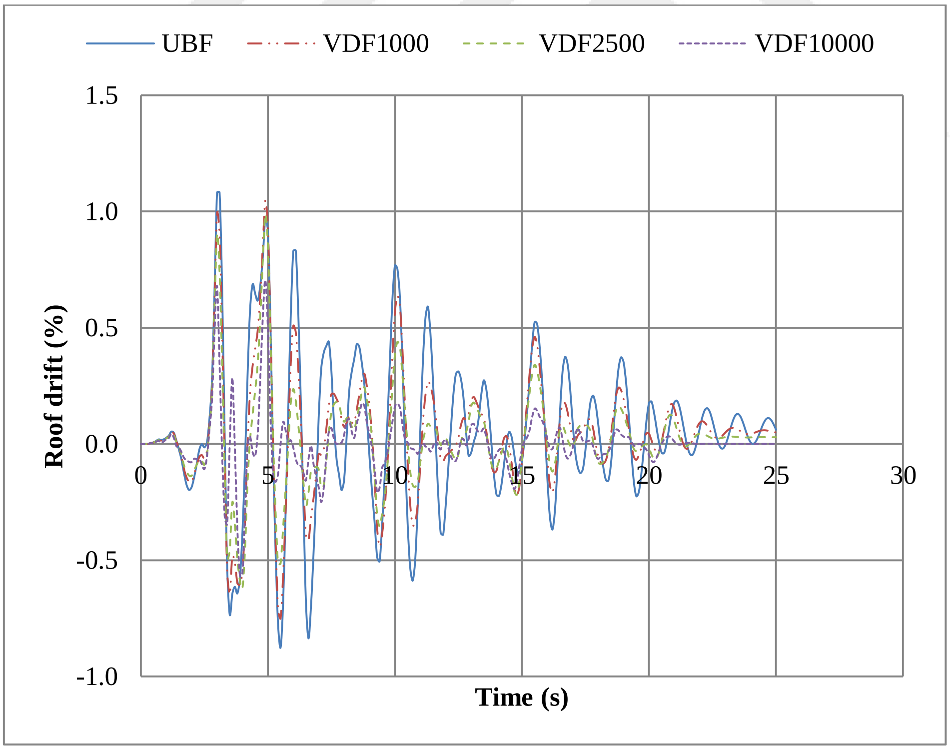


(b)

Figure A2 Roof drift time history response of the 3 storey steel a) UBF, CBF and BRBF, b) UBF and VDFs under Morgan Hill earthquake.

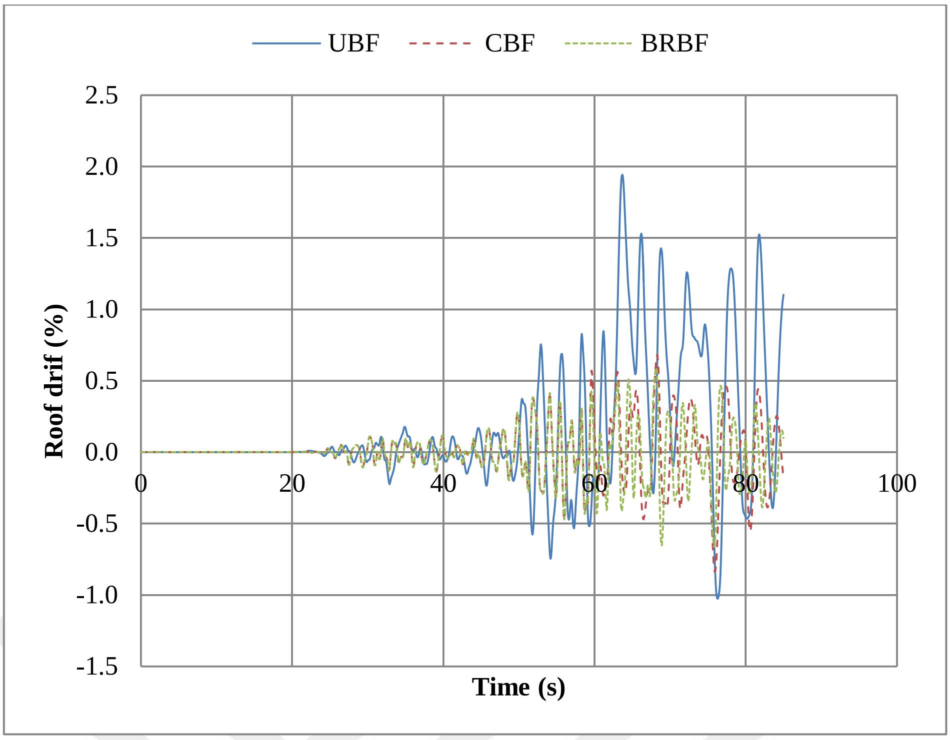


(a)

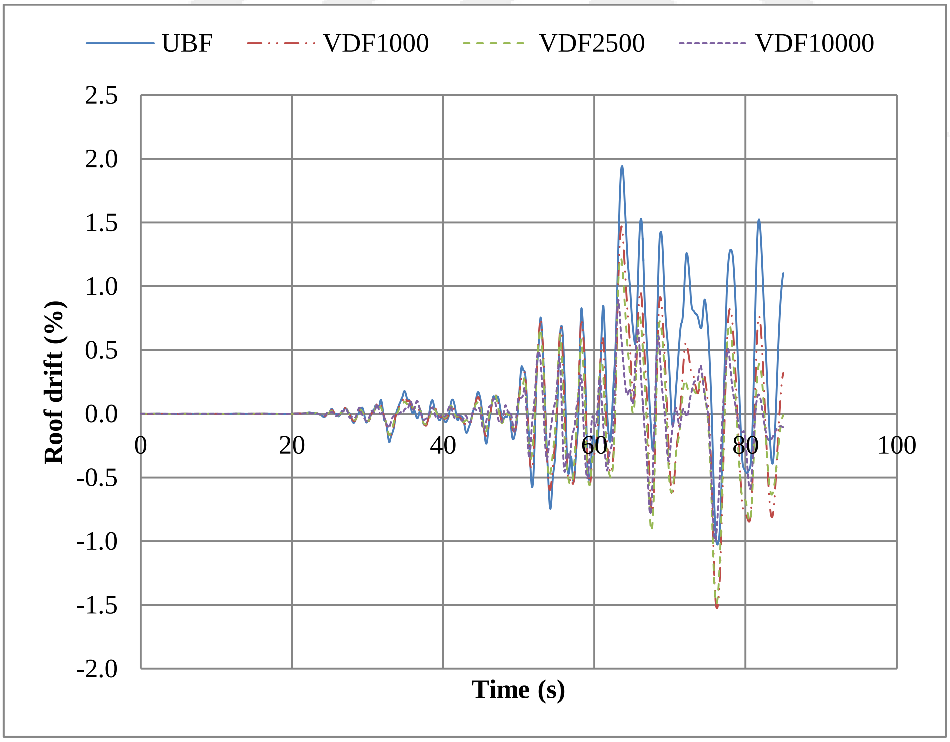


(b)

Figure A3 Roof drift time history response of the 3 storey steel a) UBF, CBF and BRBF, b) UBF and VDFs under Erzincan earthquake.

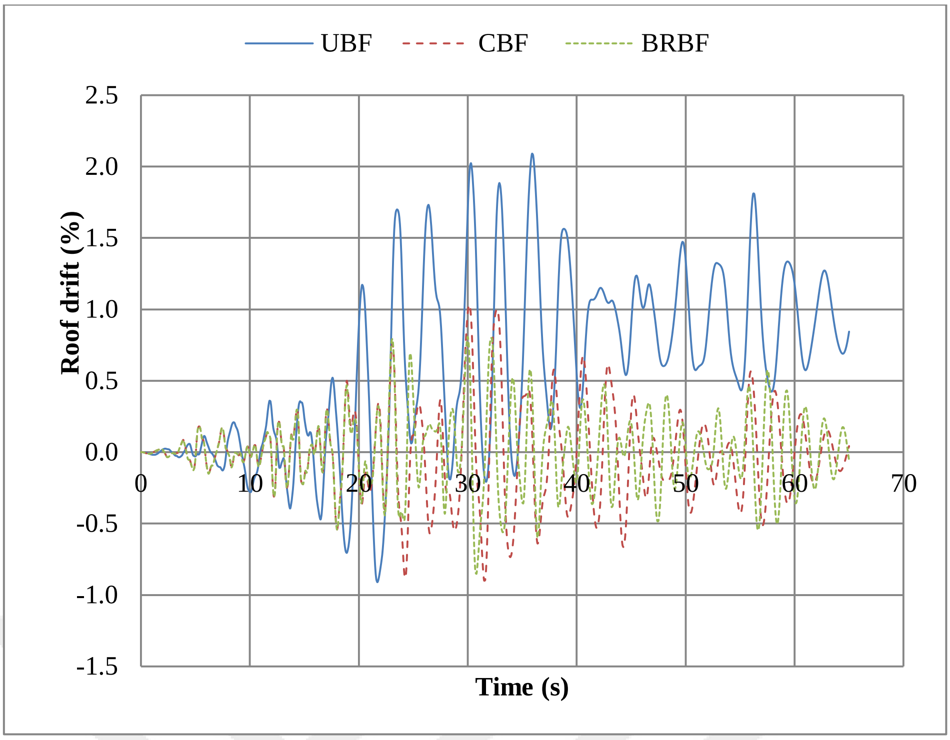


(a)

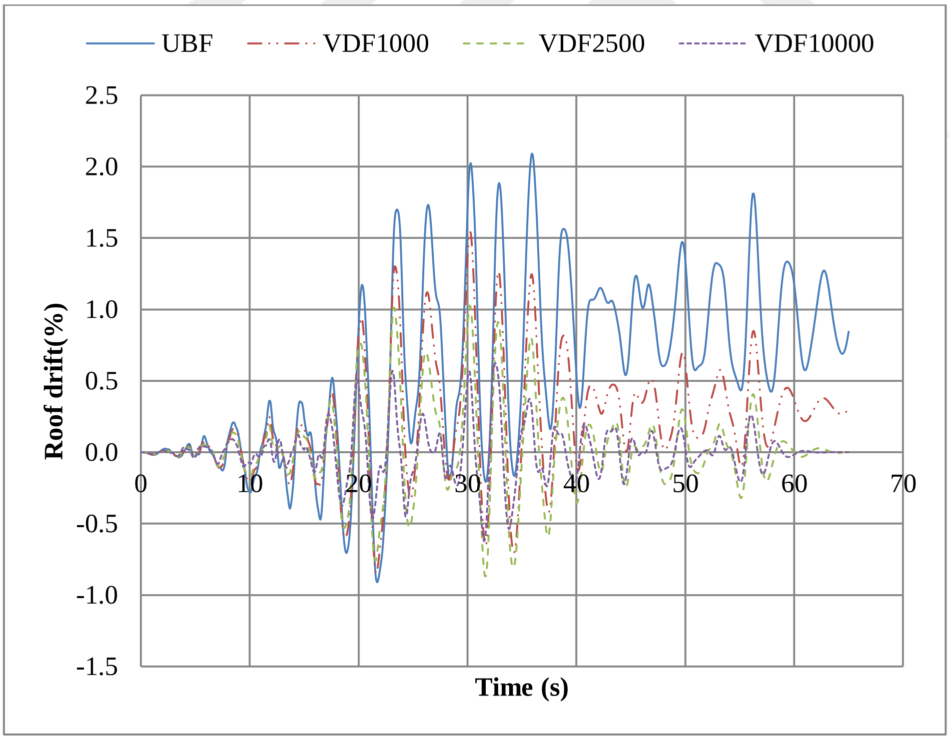


(b)

Figure A4 Roof drift time history response of the 12 storey steel a) UBF, CBF and BRBF, b) UBF and VDFs under Hector mine earthquake.

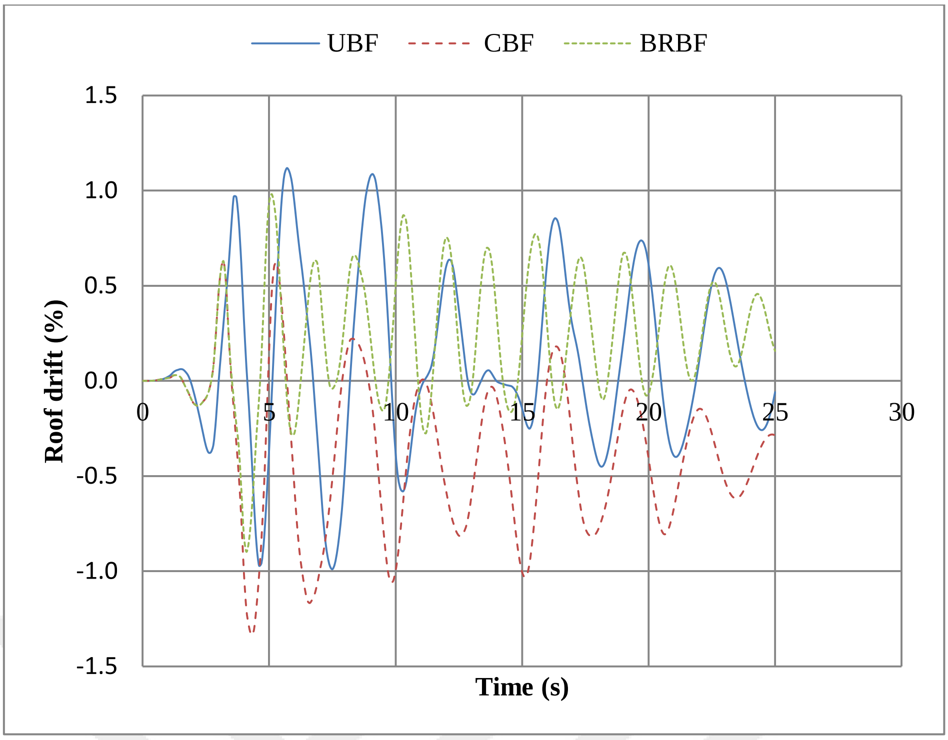


(a)



(b)

Figure A5 Roof drift time history response of the 12 storey steel a) UBF, CBF and BRBF, b) UBF and VDFs under Morgan Hill earthquake.



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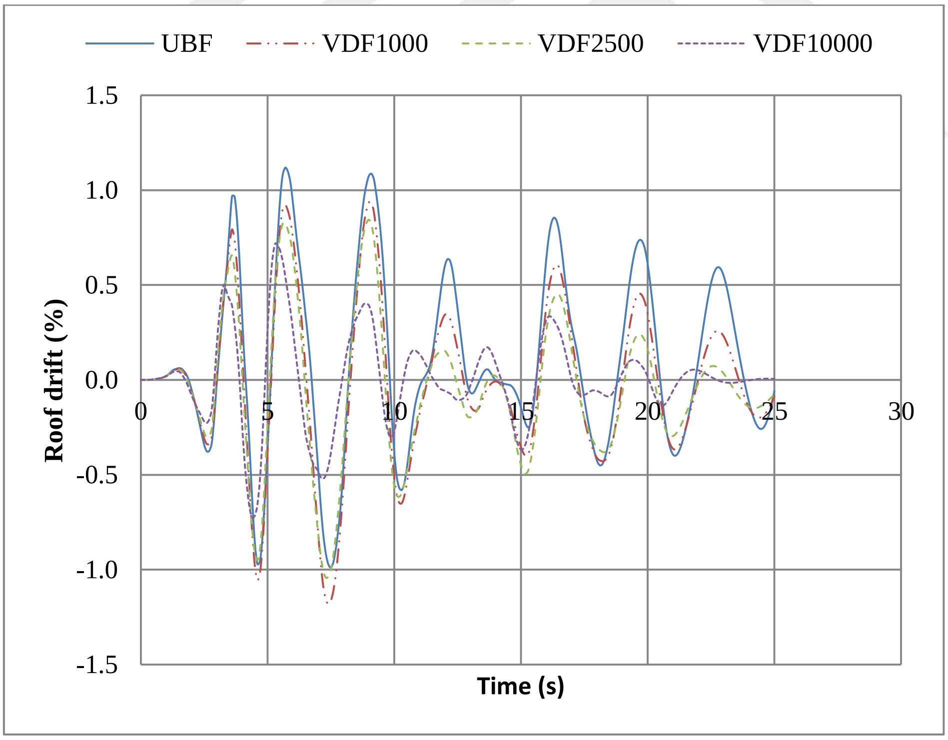
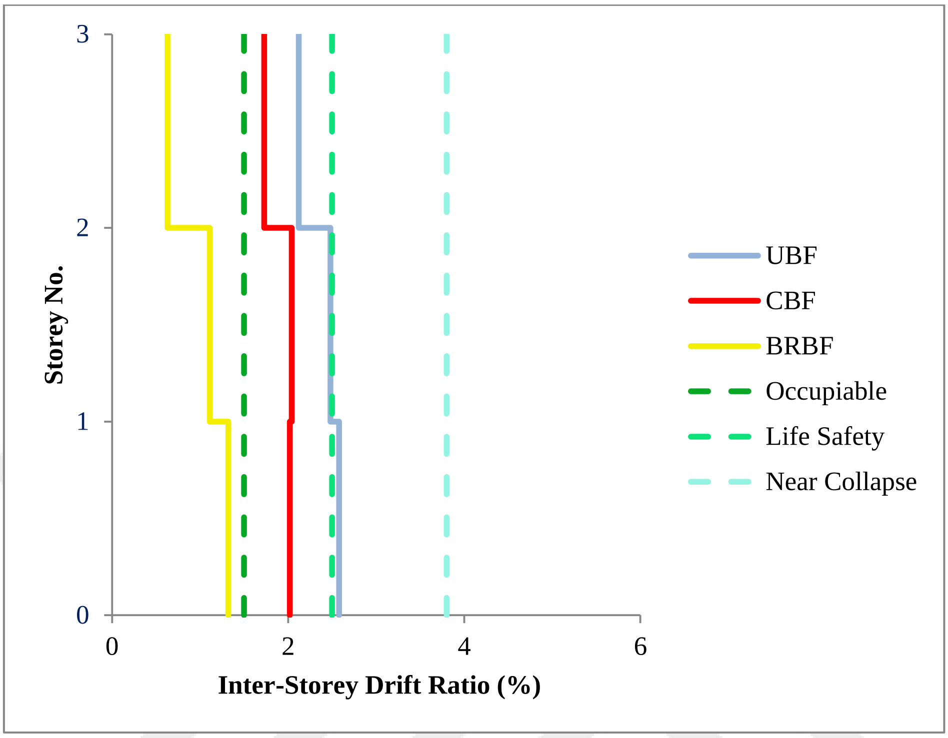


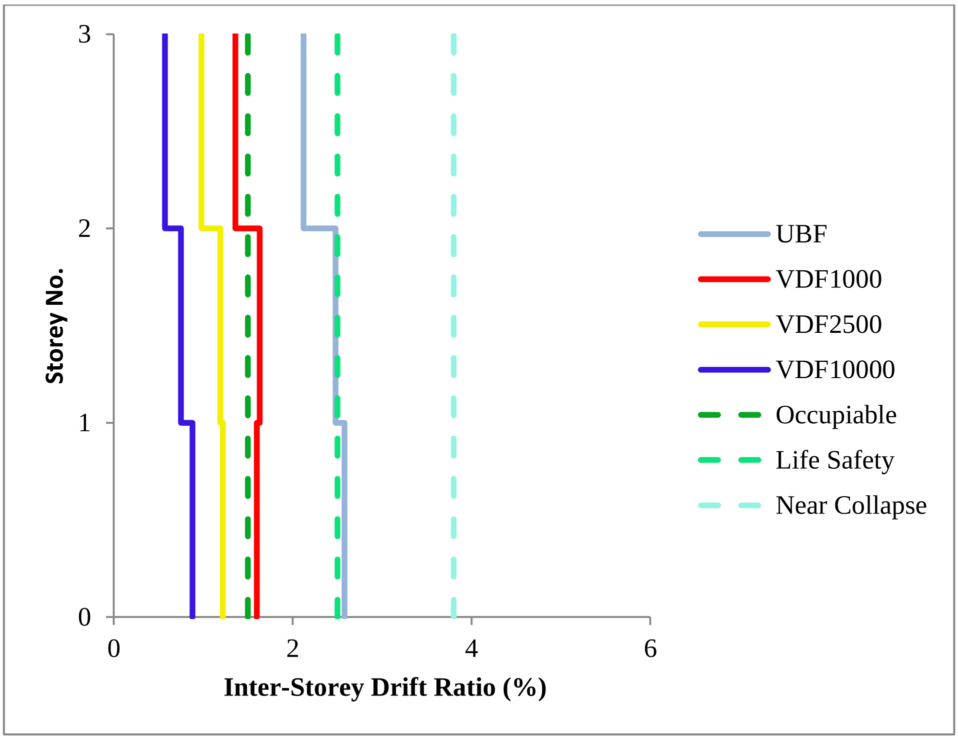
(b)

Figure A6 Roof drift time history response of the 12 storey steel a) UBF, CBF and BRBF, b) UBF and VDFs under Erzincan earthquake.

Appendix B: Height–Wise Distribution of Peak Storey Drift for 3 and 12 Storey SMRFs

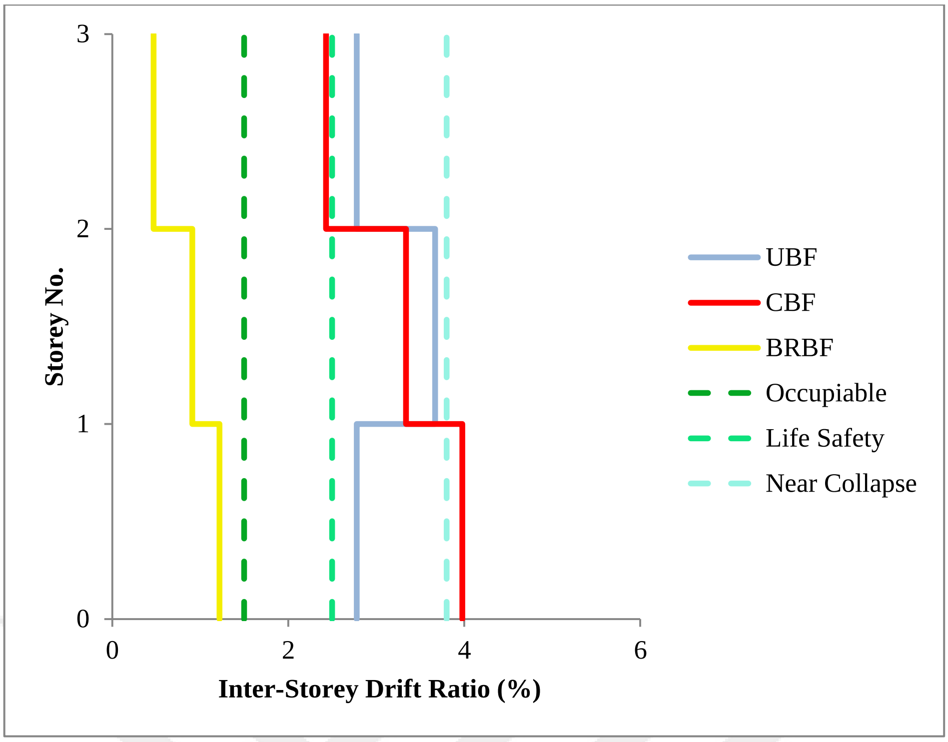


(a)

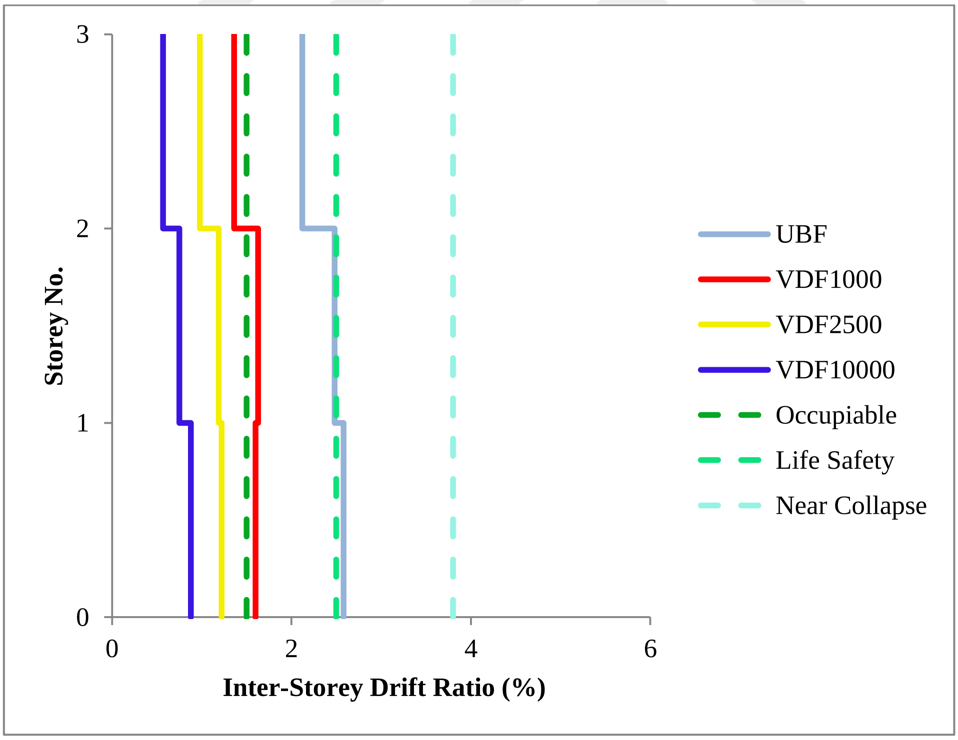


(b)

Figure B1 Height–wise distribution of peak storey drift of 3 storey steel a) UBF, CBF and BRBF, b) UBF and VDFs under Hector Mine earthquake.

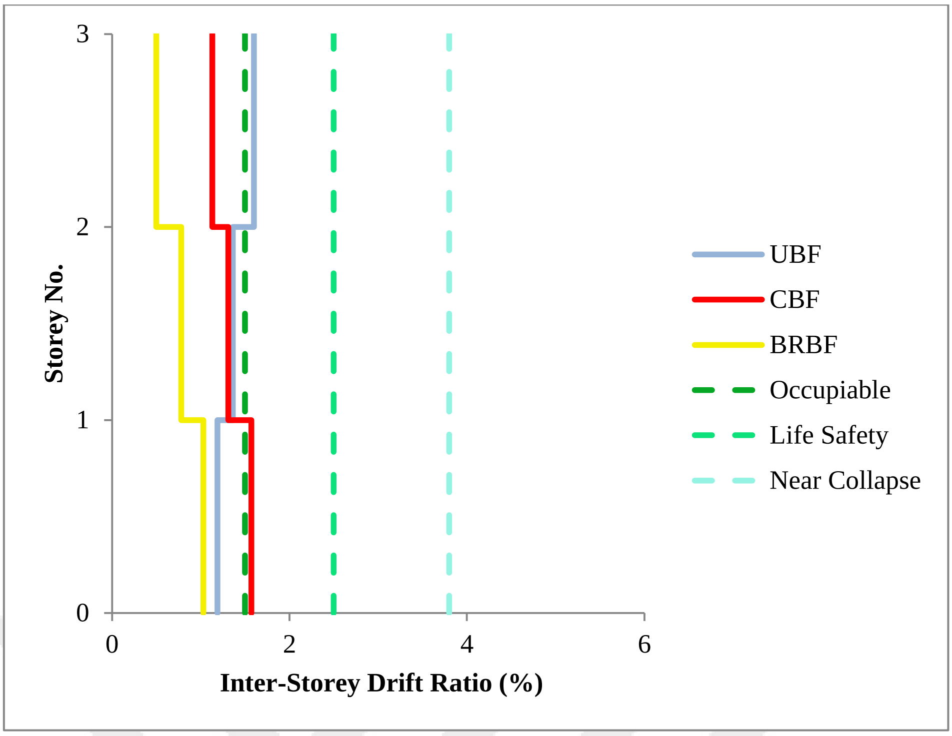


(a)

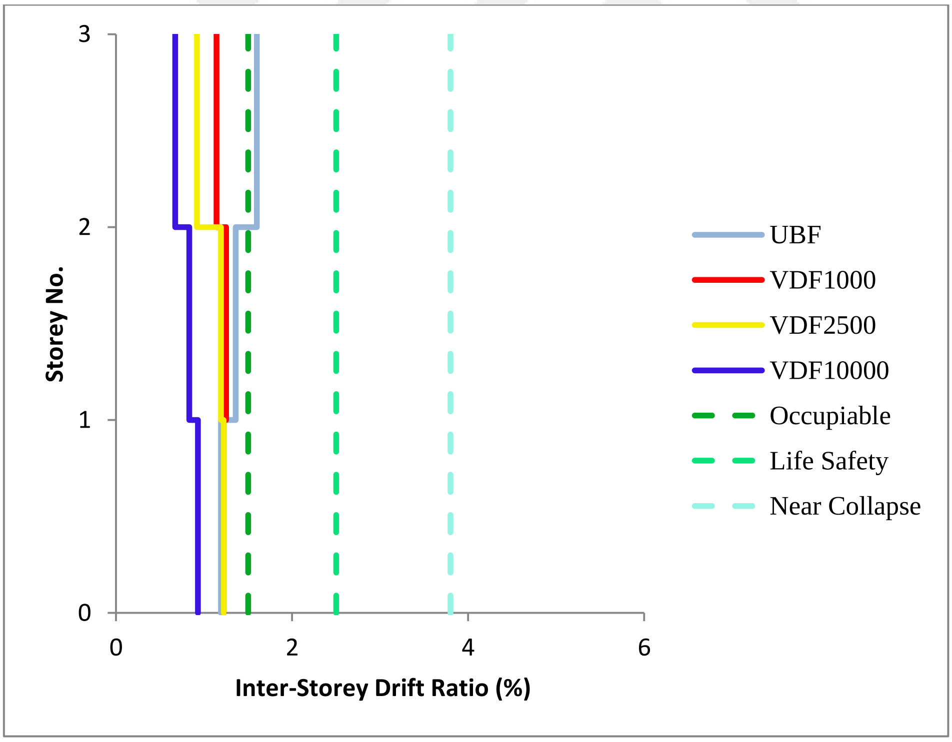


(b)

Figure B2 Height – Wise distribution of peak storey drift of 3 storeys steel a) UBF, CBF and BRBF, b) UBF and VDFs under Morgan Hill earthquake.

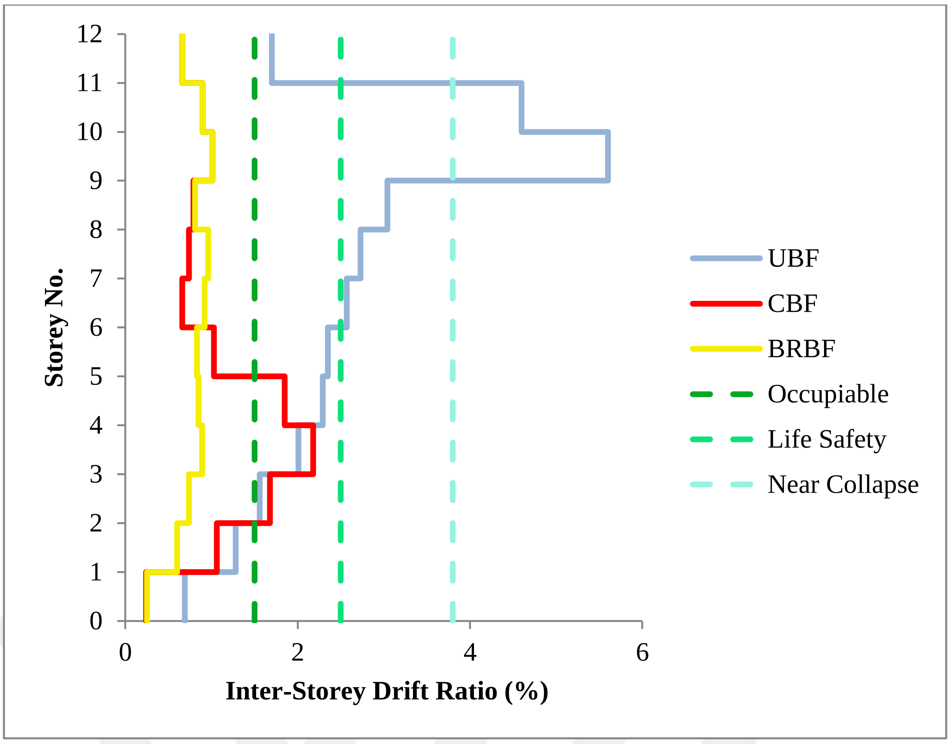


(a)

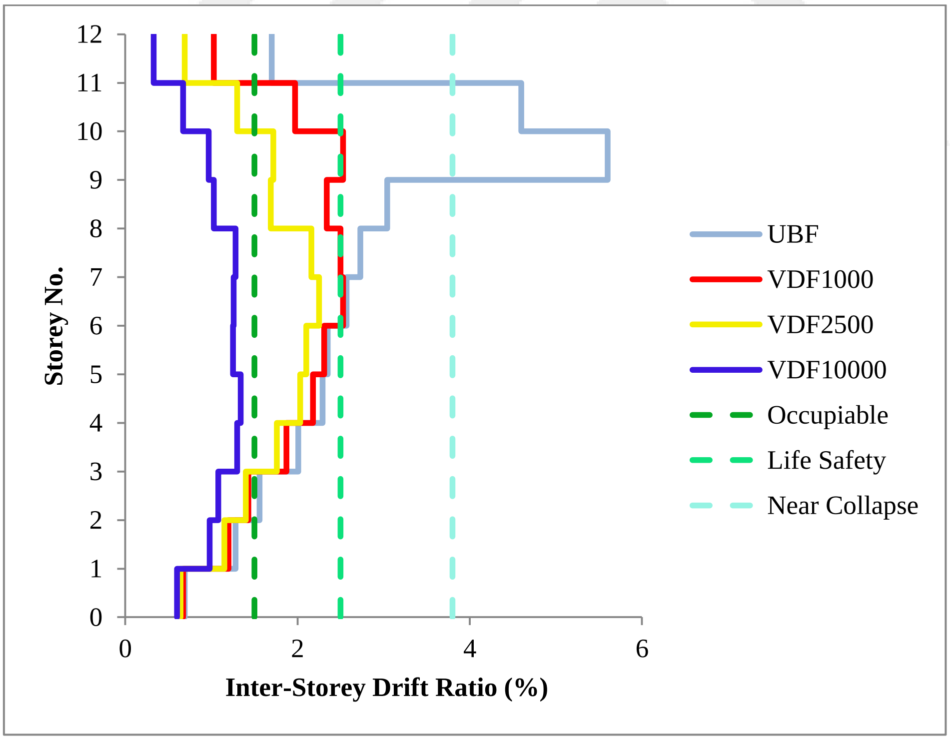


(b)

Figure B3 Height – wise distribution of peak storey drift of the 3 storey steel a) UBF, CBF and BRBF, b) UBF and VDFs under Erzincan earthquake.

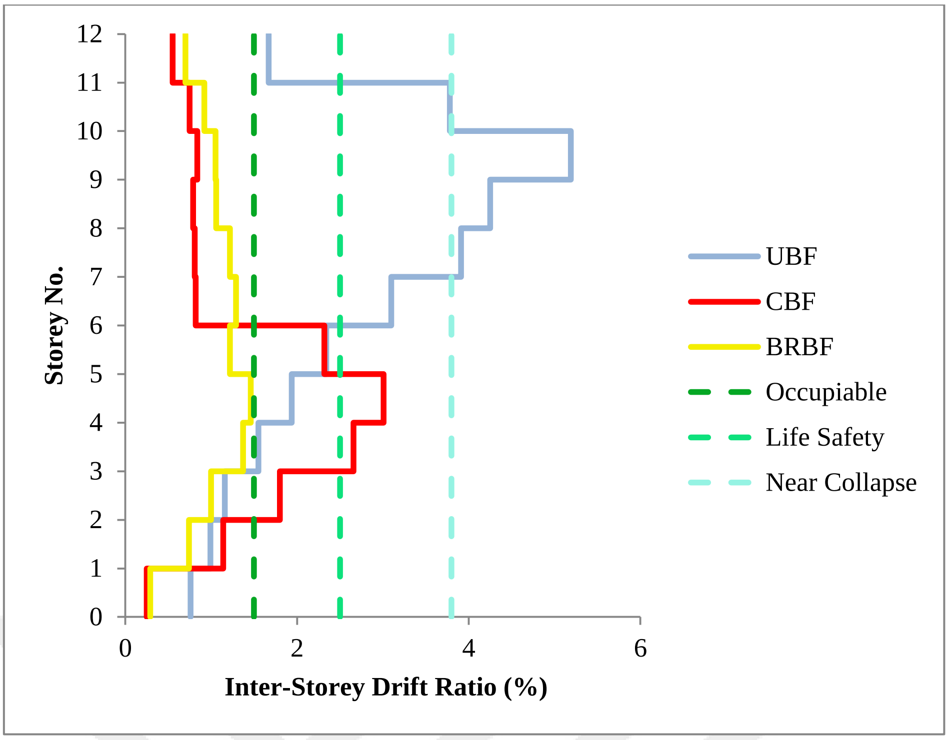


(a)

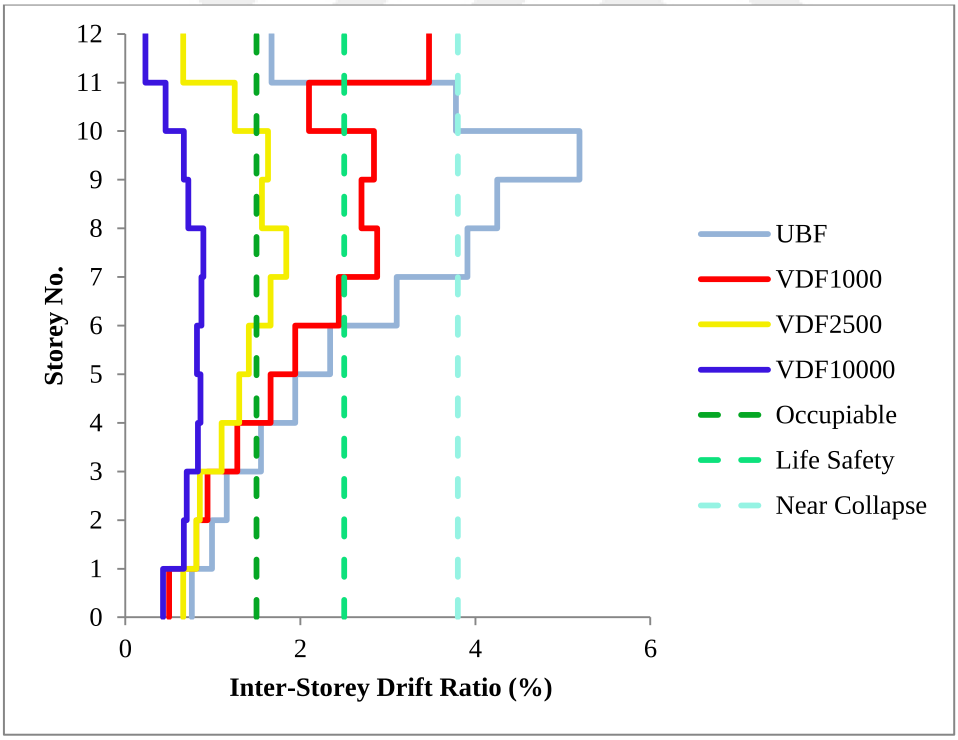


(b)

Figure B4 Height – wise distribution of peak storey drift of the 12 storey steel a) UBF, CBF and BRBF, b) UBF and VDFs under Hector Mine earthquake.

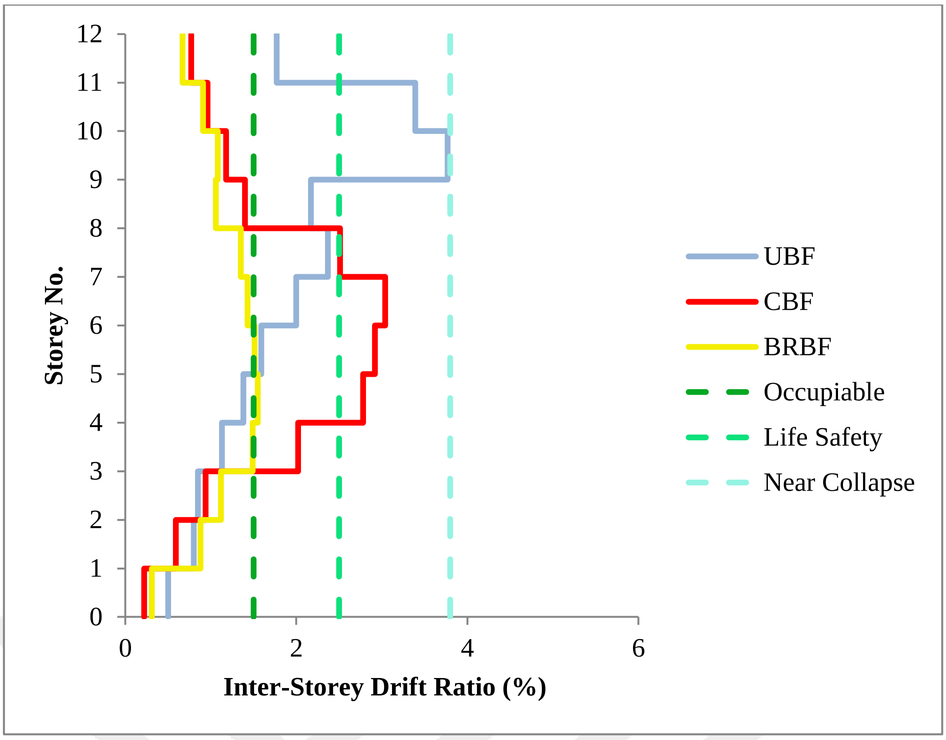


(a)

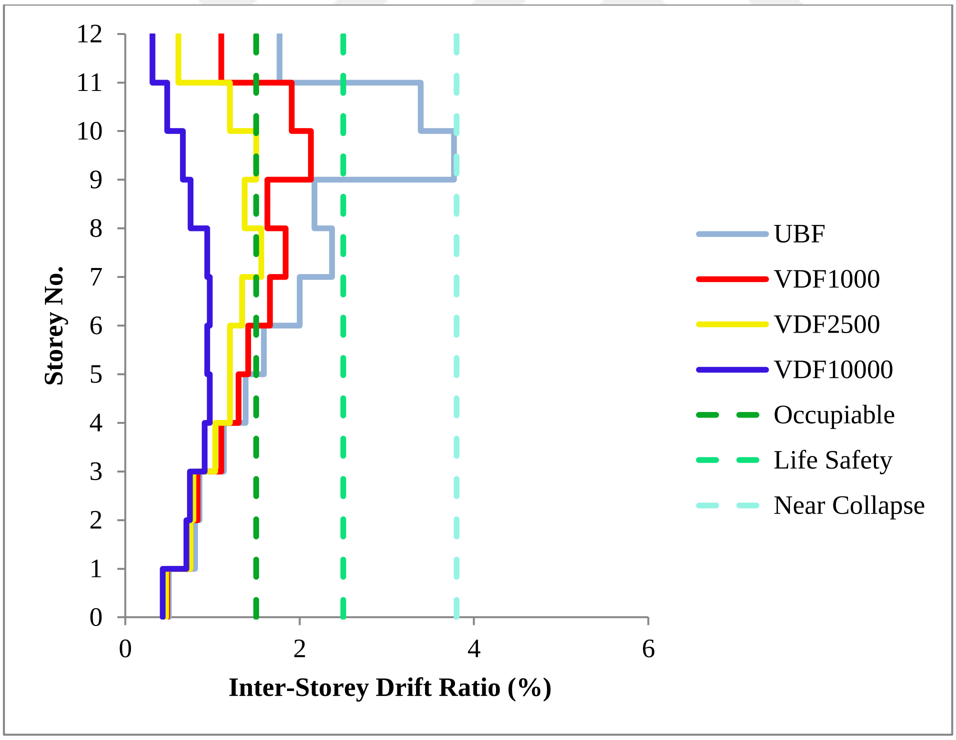


(b)

Figure B5 Height – wise distribution of peak storey drift of the 12 storey steel a) UBF, CBF and BRBF, b) UBF and VDFs under Morgan Hill earthquake.



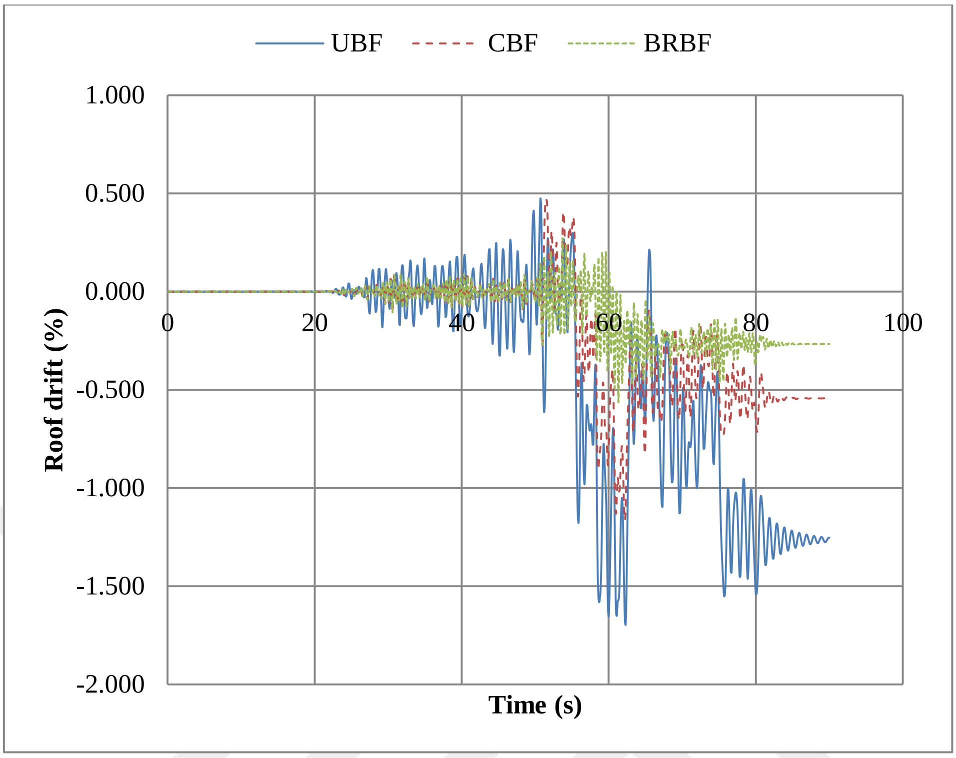
(a)



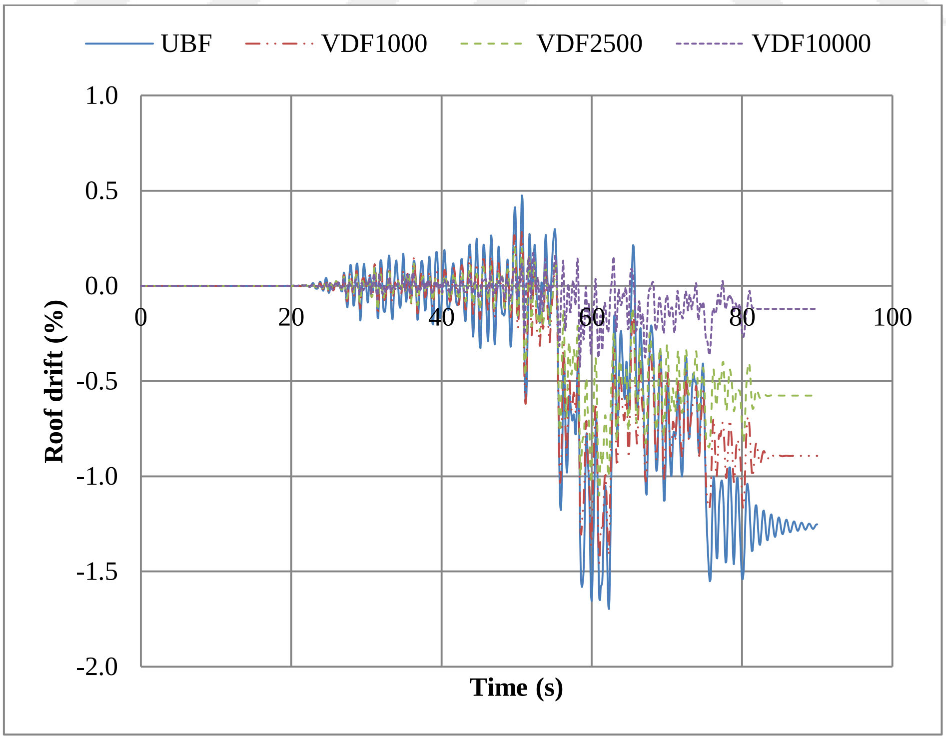
(b)

Figure B6 Height – wise distribution of peak storey drift of the 12 storey steel a) UBF, CBF and BRBF, b) UBF and VDFs under Erzincan earthquake.

Appendix C: Roof Drift Time History for 3 and 12 Storey RC-MRFs

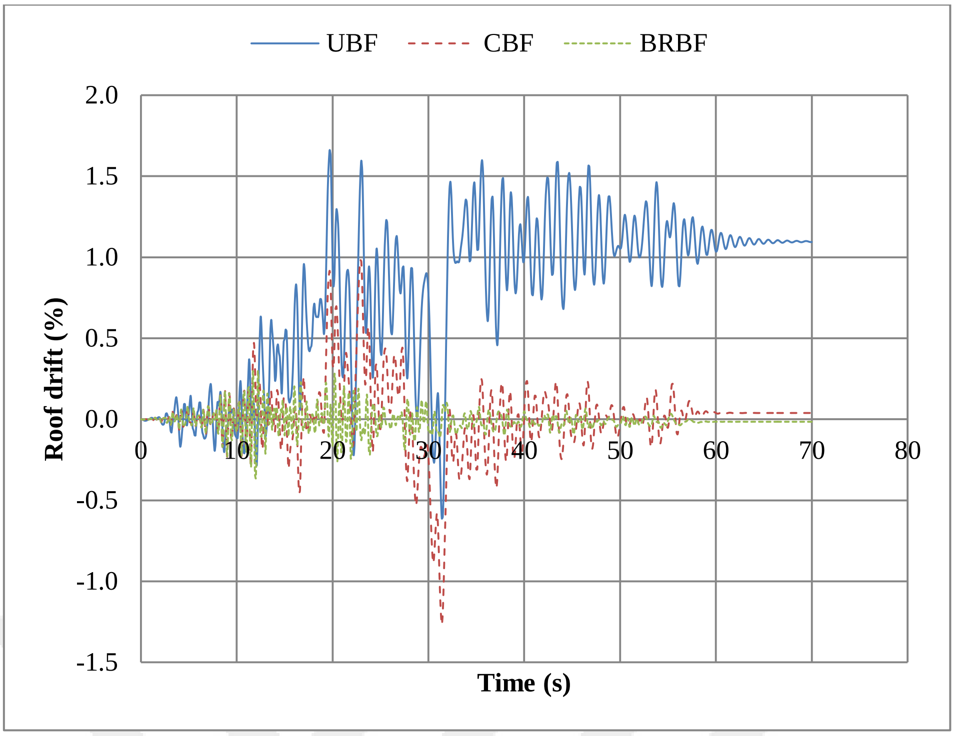


(a)

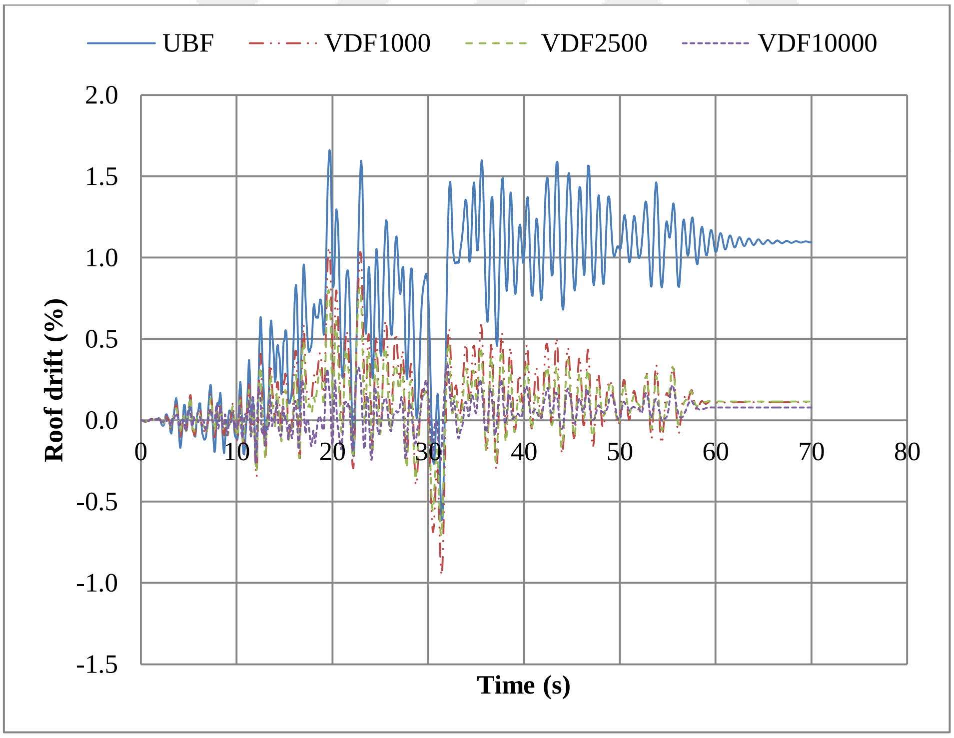


(b)

Figure C1 Roof drift time history response of the 3 storey RC a) UBF, CBF and BRBF, b) UBF and VDFs under Hector mine earthquake.

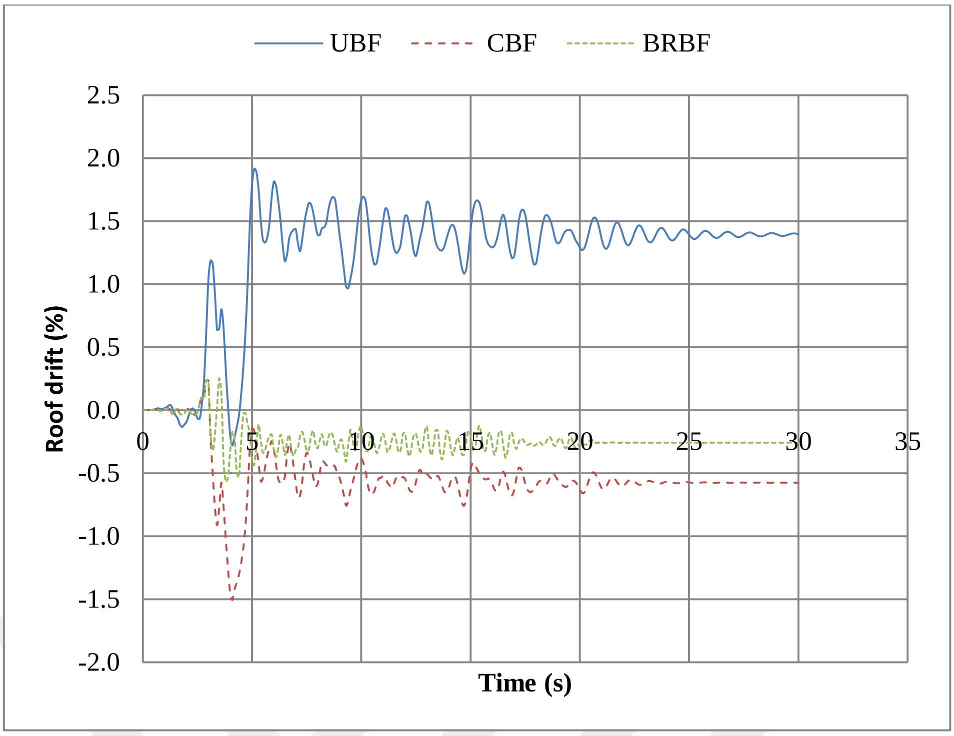


(a)

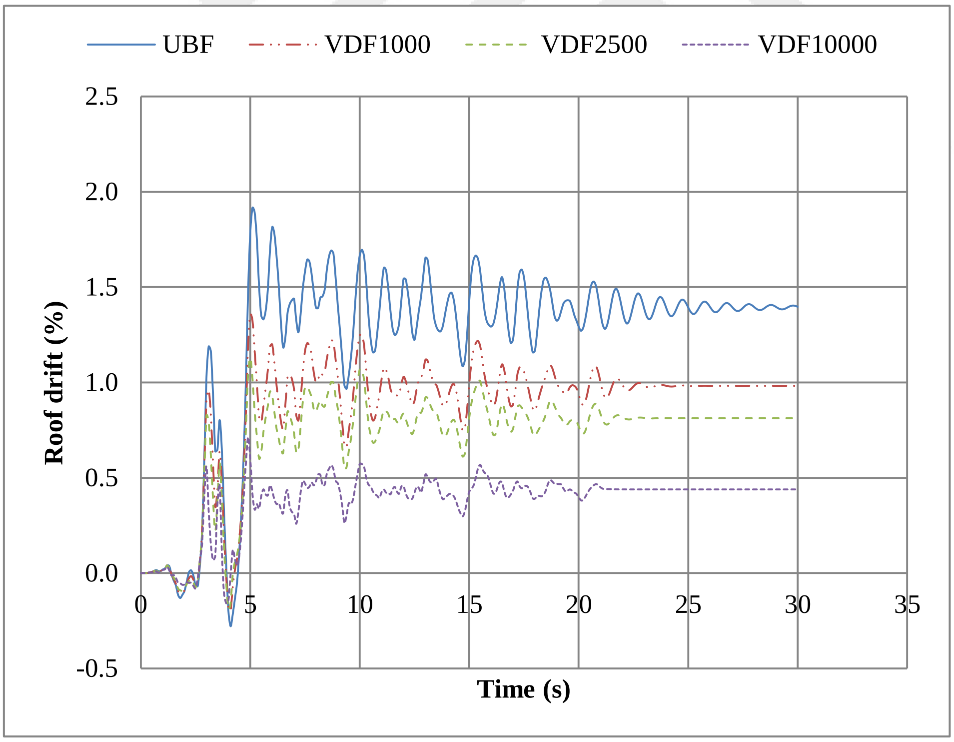


(b)

Figure C2 Roof drift time history response of the 3 storey RC a) UBF, CBF and BRBF, b) UBF and VDFs under Morgan Hill earthquake.

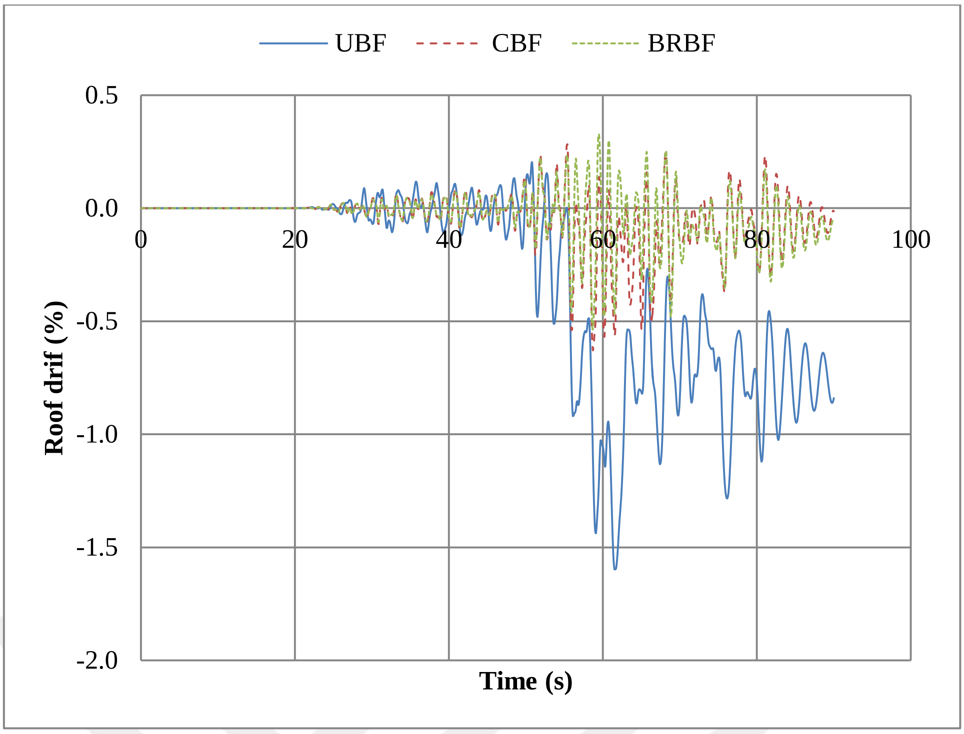


(a)

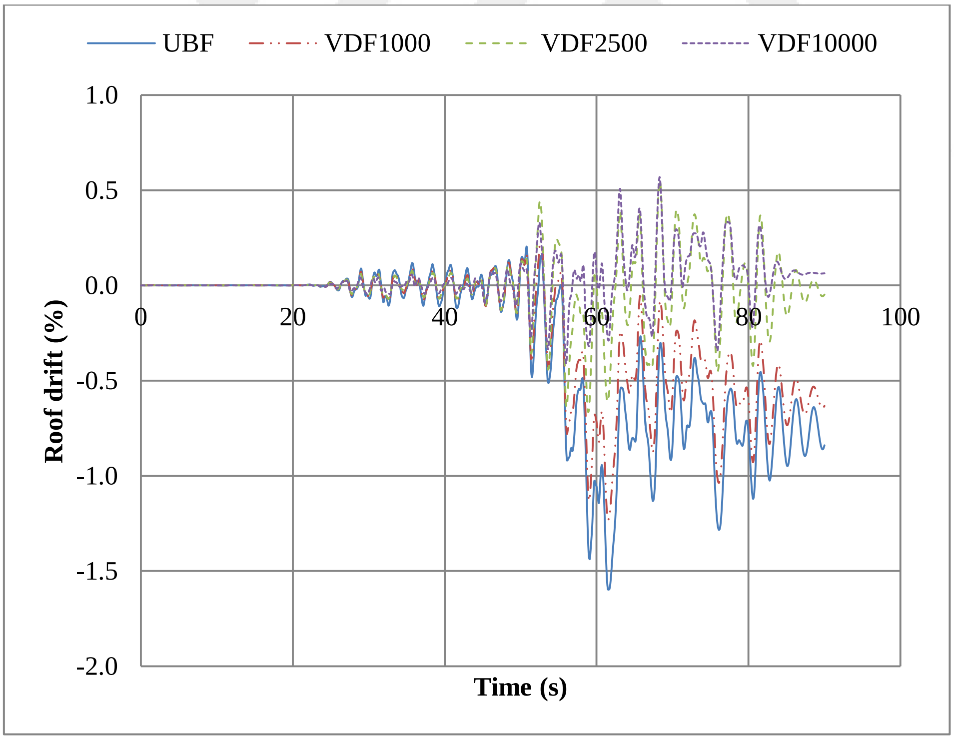


(b)

Figure C3 Roof drift time history response of the 3 storey RC a) UBF, CBF and BRBF, b) UBF and VDFs under Erzincan earthquake.

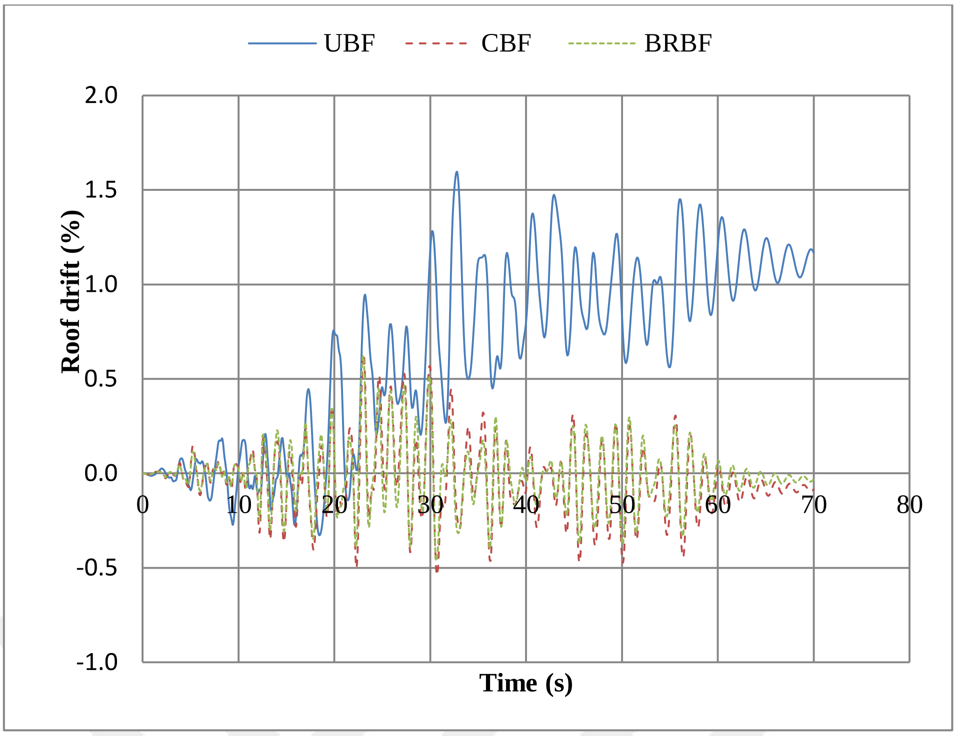


(a)



(b)

Figure C4 Roof drift time history response of the 12 storey RC a) UBF, CBF and BRBF, b) UBF and VDFs under Hector mine earthquake.



(a)

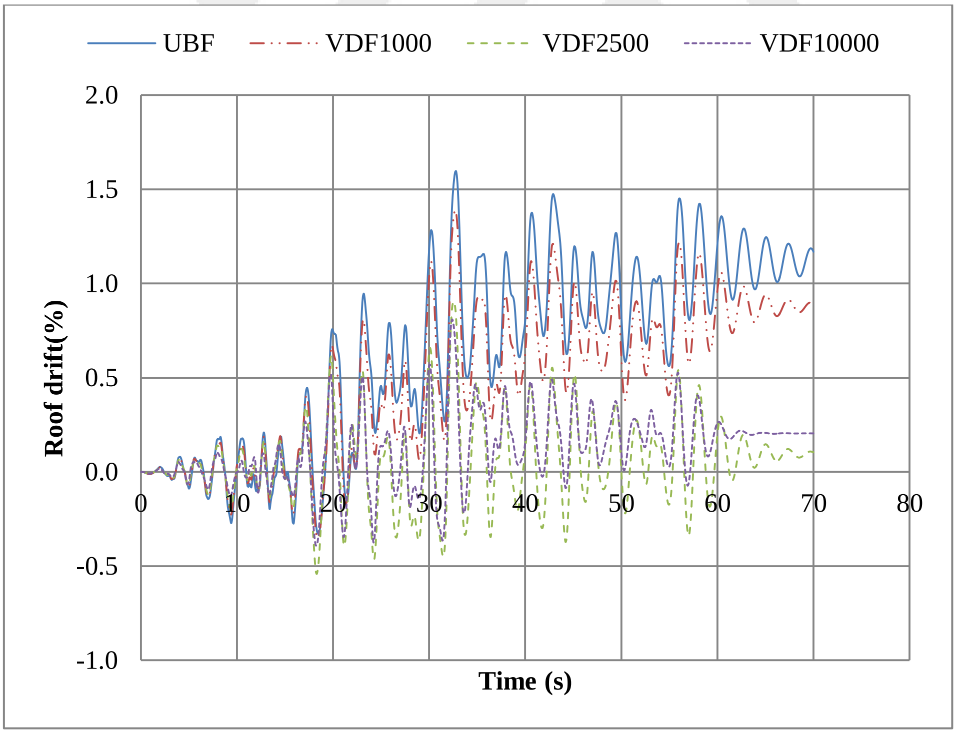


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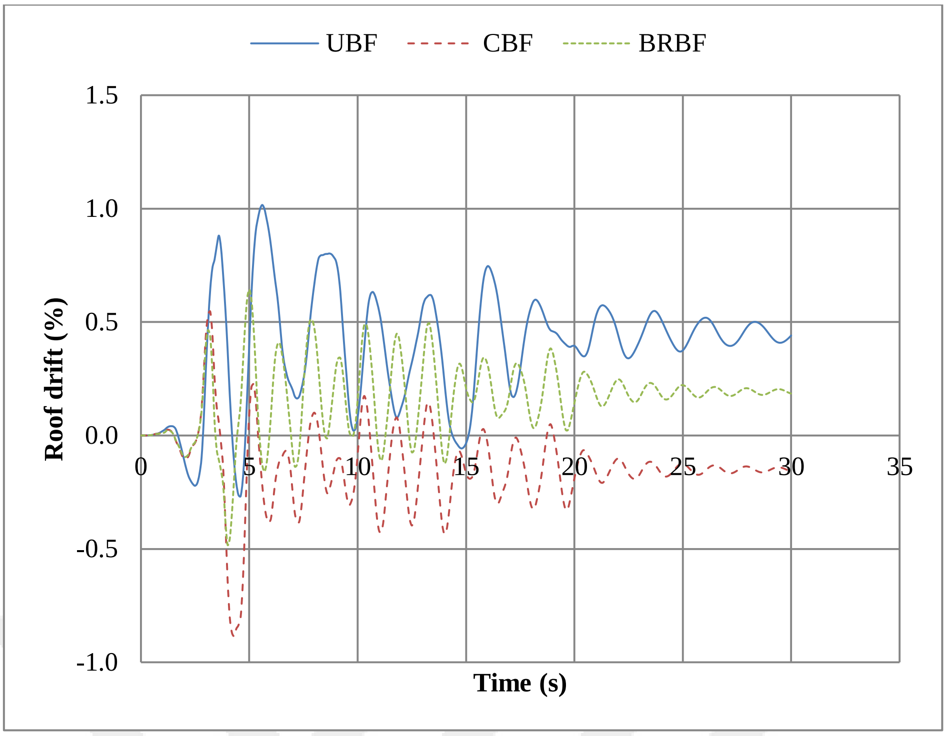
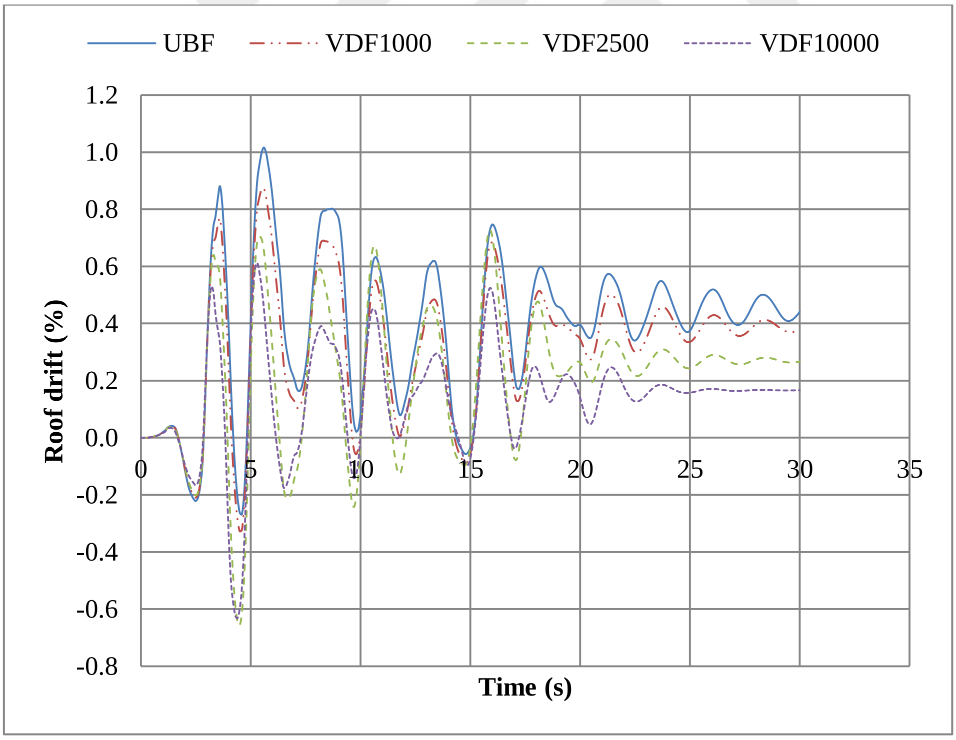
Figure C5 Roof drift time history response of the 12 storey RC a) UBF, CBF and BRBF, b) UBF and VDFs under Morgan Hill earthquake.



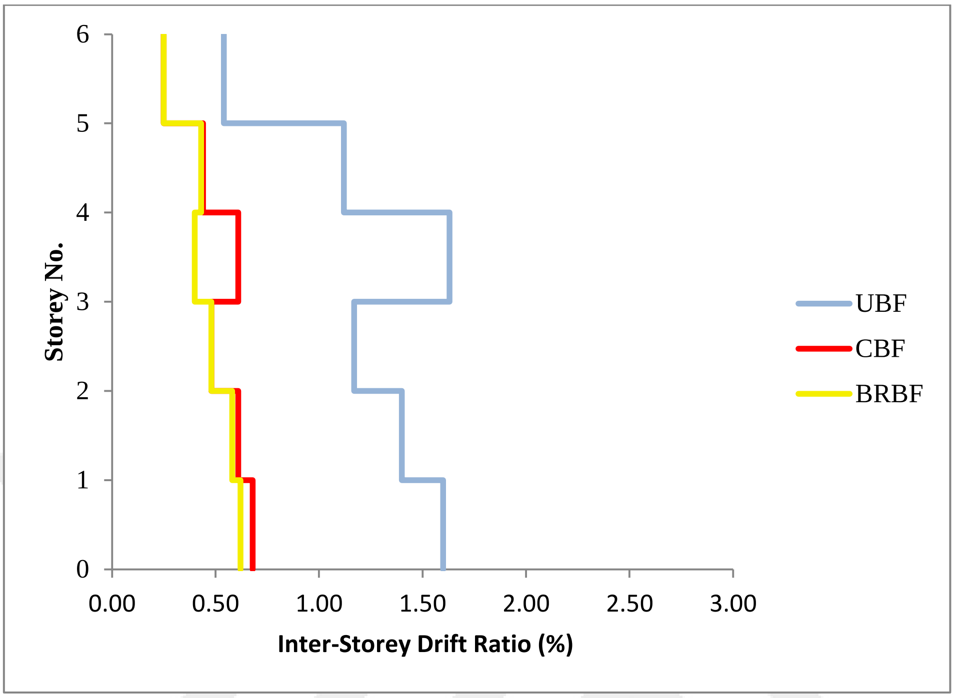
(a)



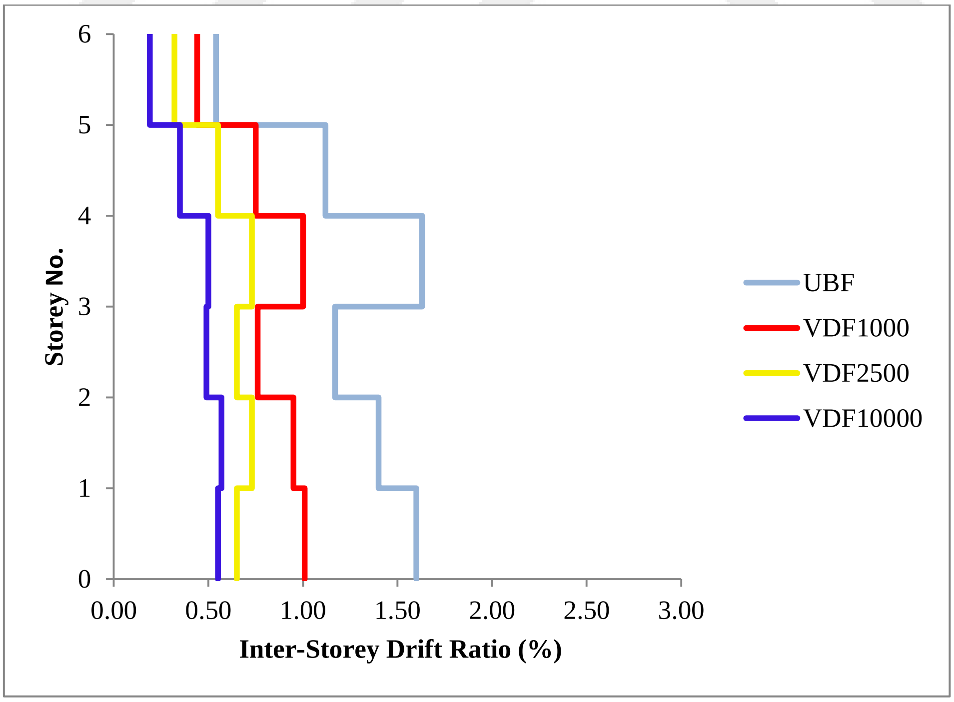
(b)

Figure C6 Roof drift time history response of the 12 storey RC a) UBF, CBF and BRBF, b) UBF and VDFs under Erzincan earthquake.

Appendix D: Height – Wise Distribution of Peak Storey Drift for 3 and 12 Storey RC-MRFs

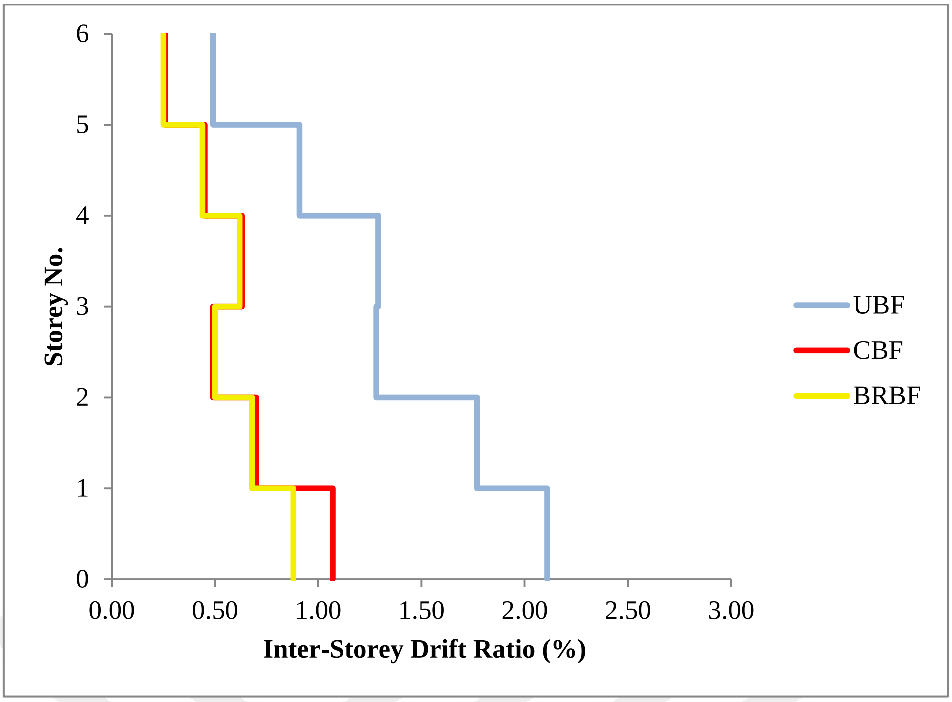


(a)

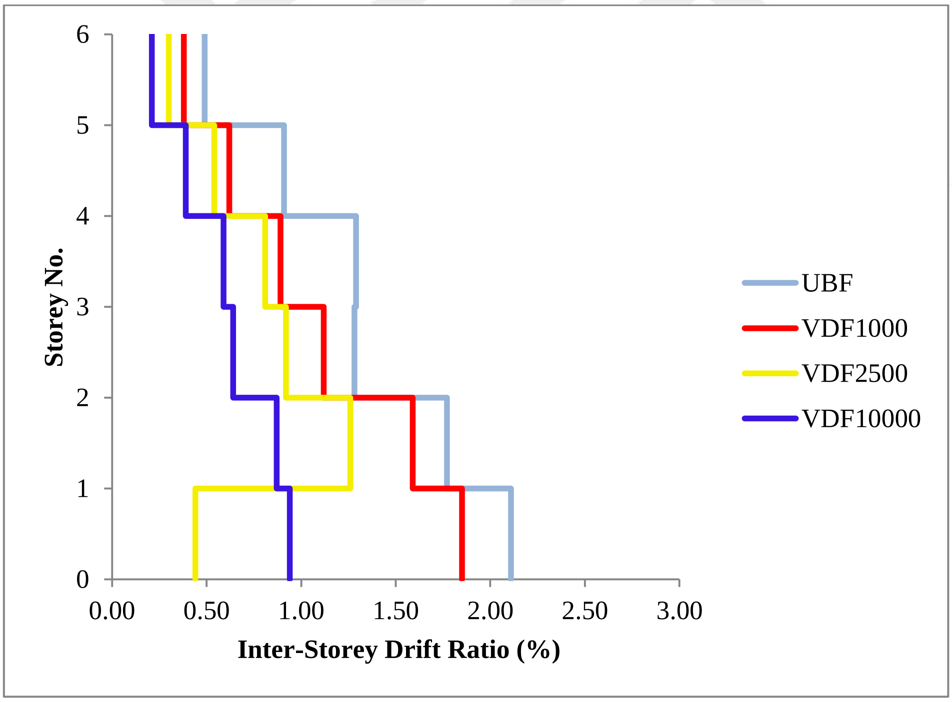


(b)

Figure D1 Height – wise distribution of peak storey drift of the 3 storey RC a) UBF, CBF and BRBF, b) UBF and VDFs under Hector Mine earthquake.

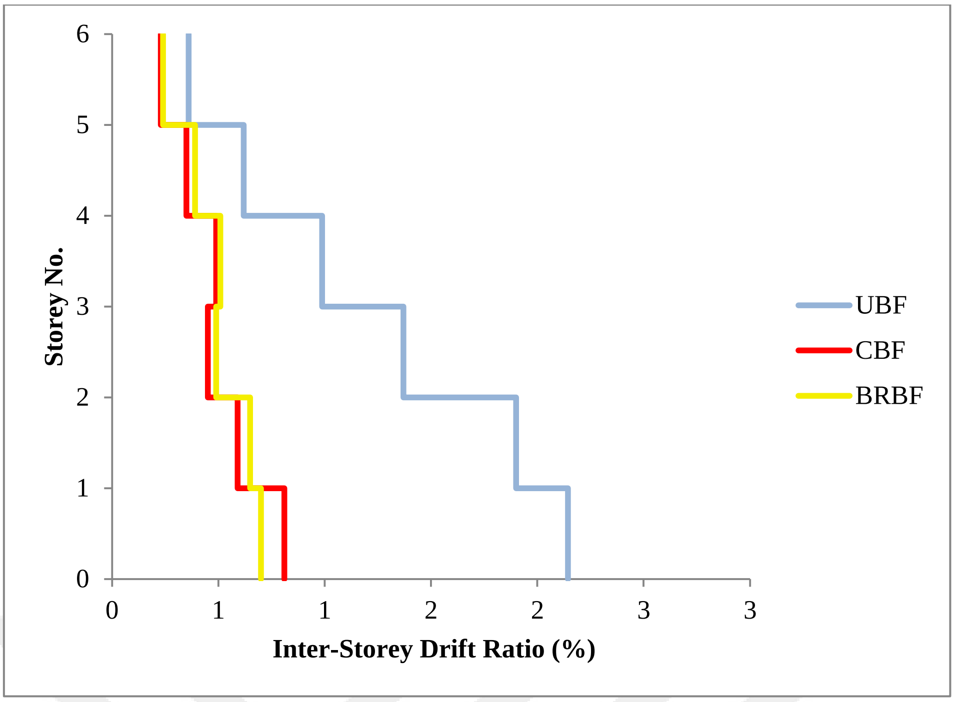


(a)

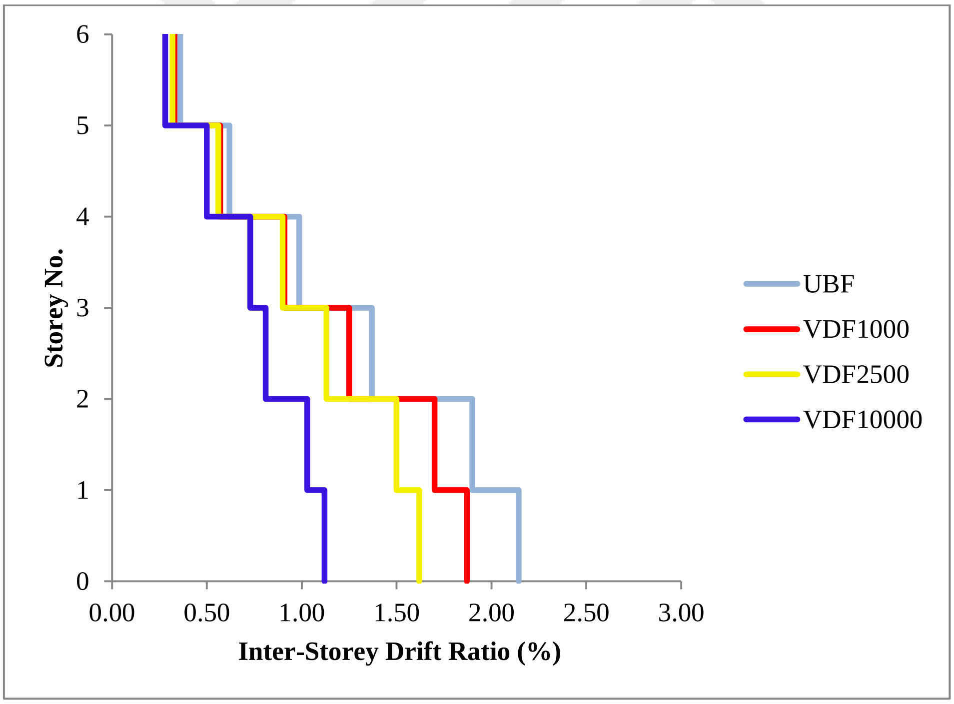


(b)

Figure D2 Height – wise distribution of peak storey drift of the 3 storey RC a) UBF, CBF and BRBF, b) UBF and VDFs under Morgan Hill earthquake.

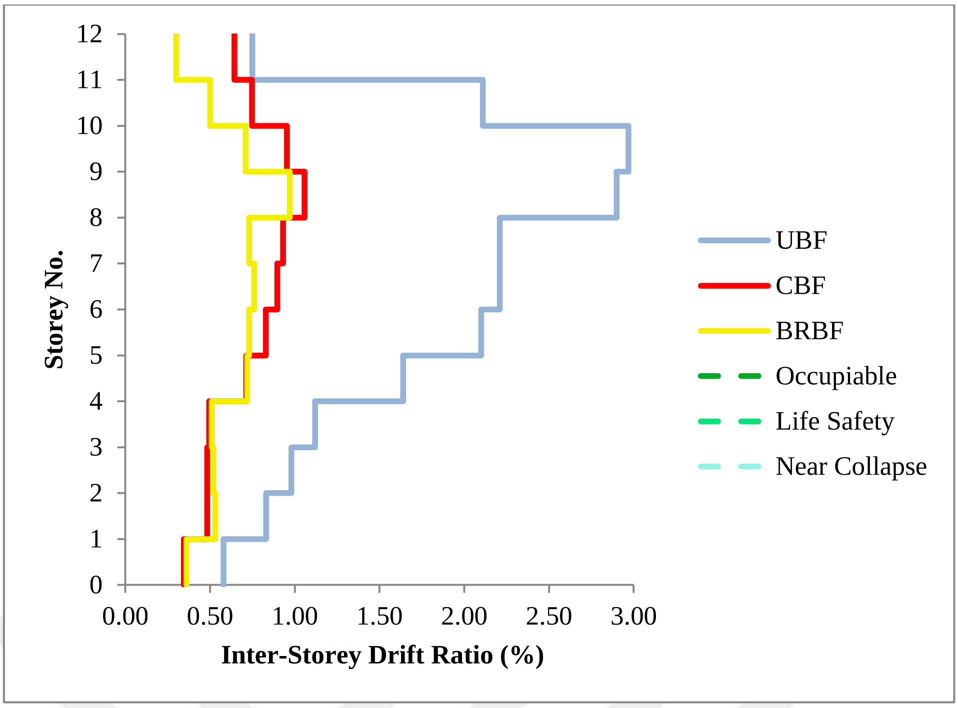


(a)

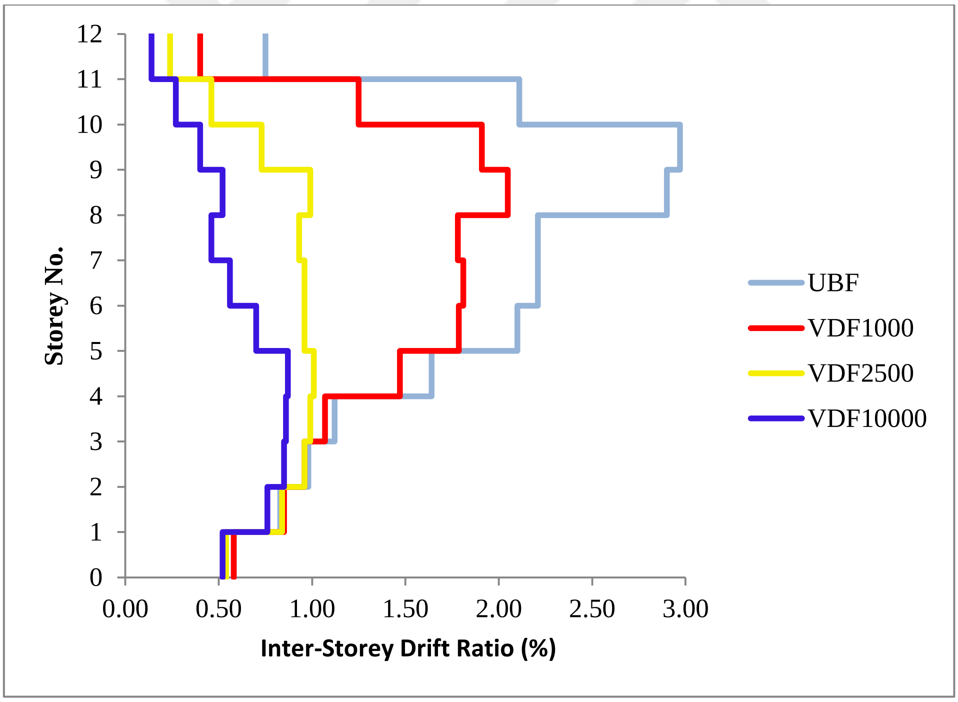


(b)

Figure D3 Height – wise distribution of peak storey drift of the 3 storey RC a) UBF, CBF and BRBF, b) UBF and VDFs under Erzincan earthquake.

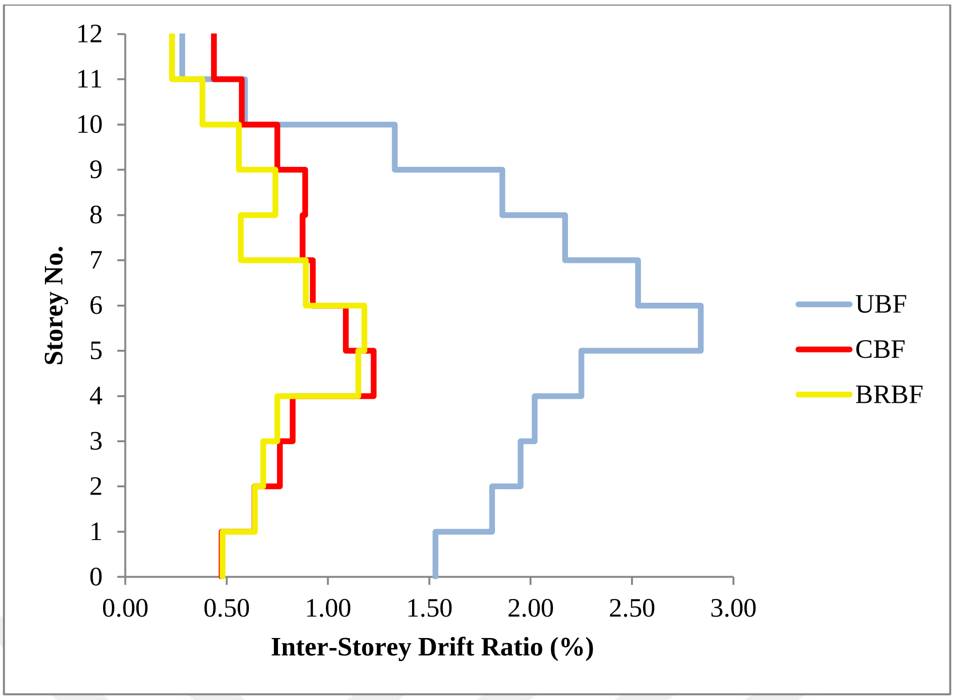


(a)



(b)

Figure D4 Height – wise distribution of peak storey drift of the 12 storey RC a) UBF, CBF and BRBF, b) UBF and VDFs under Hector Mine earthquake.



(a)

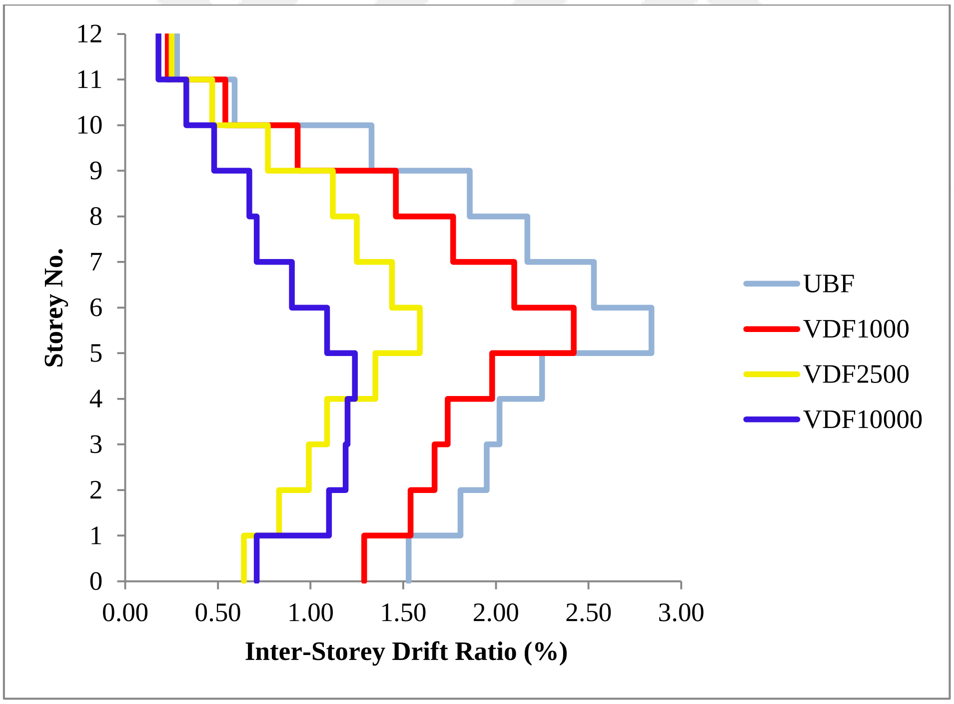


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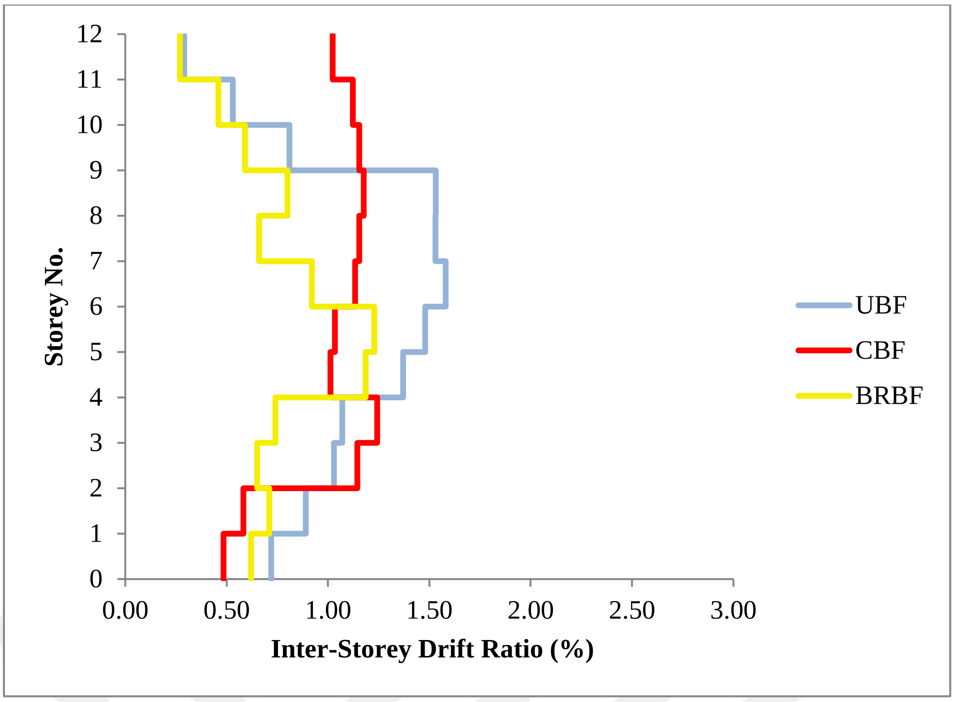
Figure D5 Height – wise distribution of peak storey drift of the 12 storey RC a) UBF, CBF and BRBF, b) UBF and VDFs under Morgan Hill earthquake.



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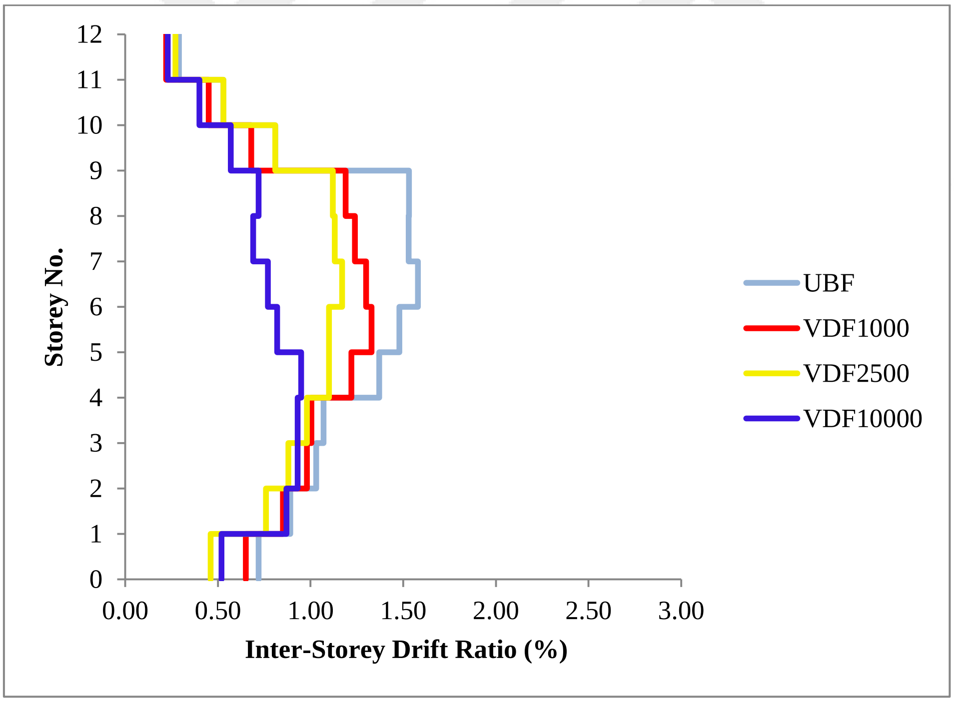

(b)

Figure D6 Height – wise distribution of peak storey drift of the 12 storey RC a) UBF, CBF and BRBF, b) UBF and VDFs under Erzincan earthquake.

PERSONAL INFORMATION

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Marital status : Married

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Education

Degree	Graduate School	Year
Master	University of Salahaddin - Erbil	1994
Bachelor	University of Salahaddin - Erbil	1992
High school	Erbil Secondary School	1988

Work experience

Year	Post	Place of work
1992-2000	Teaching Staff	Civil Department, University of Salahaddin - Erbil
2000-2003	Office Engineer	United Nation / UN-HABITAT
2003- up to date	CEO	Alwand Company

Publications

a- International Journals

a- AZEZ, Ibrahim Hassan (1997). A comparison Between Flexural Strength and Behavior of Ferrocement and Fibrous Thin Concrete Ribbons. Zanco Journal , Vol. 9, No. 1.

b- AZEZ, Ibrahim Hassan (1997). Behavior and Flexural Strength of Ferrocement One-Way Slabs with Opening. *Journal of Zankoy Sulaimani*, **1**, No. 1.

c- AZEZ, Ibrahim Hassan (1999). Effect of Partial Replacement of Steel Wire Meshes by Steel Fibers on Behavior and Strength of Ferrocement Beams. *Zanco Journal*, **11**, No. 1.

d- Güneyisi, E. M., Osman T. and Ibrahim H. Azez, (2015). Nonlinear Dynamic Respone of Reinforced Concrete Building Retrofitted with Buckling Restrained Braces. *Journal of Earthquakes and Structures*, **8**, No. 6, pp 1349-1362.

e- Güneyisi, E. M. and Ibrahim H. Azez, (2016). Seismic Upgrading of Structures with Different Retrofitting Methods. . *Journal of Earthquakes and Structures*, *10*, No. 3, pp 589-611.

b- <u>International conferences</u>

a- Azez, Ibrahim Hassan and Saeed, J. A. (1999). Behavior and Flexural Strength of Ferrocement slabs. First Scientific Conference of University of Dohuk.

b- Azez, Ibrahim Hassan (1997). A comparison Between Behavior of Ferrocement, Fibrous and Conventionally Reinforced Concrete Slabs. Third Scientific Conference of University of Salahaddin-Erbil, 1997.

Languages : English, Arabic and Kurdish

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