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EFFECT OF STRENGTH AND STIFFNESS OF BUCKLING-RESTRAINED BRACES ON SEISMIC RETROFITTING OF REINFORCED CONCRETE BUILDINGS

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Effect of Strength and Stiffness of Buckling-Restrained Braces on Seismic Retrofitting of Reinforced Concrete Buildings

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ABSTRACT

EFFECT OF STRENGTH AND STIFFNESS OF BUCKLING-RESTRAINED BRACES ON SEISMIC RETROFITTING OF REINFORCED CONCRETE BUILDINGS

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M.Sc. in Civil Engineering

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This thesis presents an analytical study aimed at evaluating the effect of using buckling-restrained braces (BRBs) on the seismic retrofitting of 3 and 6 storey reinforced concrete (RC) buildings. In the design of the BRBs with non-prismatic cross-sections, twelve combinations of α and β design parameters that influence the strength and stiffness of the BRBs, respectively, were considered. The response of the RC structures with and without BRBs under earthquake ground accelerations were evaluated through nonlinear dynamic analysis. Two sets of ground motions representative of the design earthquake with 10% and 50% exceedance probability in fifty years were taken into account. By comparing the structural performance of the original and buckling restrained braced structures, it was observed that the use of the BRBs were very effective in mitigating the seismic response as a retrofit scheme. However, the selection of the strength and stiffness parameters of the BRBs had considerable effect on the response characteristics of RC structures such as by increasing the value of α and by decreasing the value of β of the buckling-restrained braces, the maximum deformation demand of the structures increased.

Keywords: Buckling restrained brace, Nonlinear dynamic analysis, Reinforced concrete building, Seismic response

ÖZET

BURKULMASI ÖNLENMİŞ ÇARPRAZLARIN DAYANIM VE RİJİTLİĞİNİN BETONARME BİNALARIN SİSMİK GÜÇLENDİRİLMESİNDEKİ ETKİSİ

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Bu tez çalışmasında, burkulması önlenmiş çelik çarprazların 3 ve 6 katlı betonarme binaların sismik güçlendirilmesindeki etkinliği araştırılmıştır. Burkulması önlenmiş çarprazların prizmatik olmayan tasarımında çarprazların dayanımına ve rijitliğine etki eden sırasıyla 12 çeşit α ve β kombinasyonu dikkate alınmıştır. Mevcut ve güçlendirilmiş yapıların deprem etkisi altındaki tepkileri doğrusal olmayan dinamik analiz yardımıyla değerlendirilmiştir. Dinamik analizlerde 50 yılda aşılma olasılığı %10 ve 50 yılda aşılma olasılığı %50 olan depremi esas alan tasarım ivme spektrumlarıyla uyumlu olacak şekilde ölçeklendirilen iki grup deprem ivmesi kaydı kullanılmıştır. Mevcut ve burkulması önlenmiş çarprazlı binaların yapısal performanslarının karşılaştırılması sonucunda burkulması önlenmiş çarprazların güçlendirme amaçlı kullanımının, yapının sismik tepkilerinin azaltılması için oldukça etkili olduğu gözlenmiştir. Ancak, çarprazların dayanımı ve rijitliği gibi tasarım parametrelerinin yapı performansı üzerinde etkili olduğu; örneğin α değerinin artırılması ve β değerinin azaltılmasının yapının maksimum yer değiştirme taleplerinde artışa neden olduğu görülmüştür.

Anahtar Kelimeler: Burkulması önlenmiş çarpraz, Doğrusal olmayan dinamik analiz, Betonarme bina, Sismik tepki

To my beloved parents, father, mother and sister...

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My special thanks are reserved for my parents, all my family. They have given me an endless enthusiasm and encouragement.

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LIST OF SYMBOLS / ABBREVIATIONS

α	Ratio of reduced area of core to total area of section
β	Ratio of length of core with reduced area to total length of brace
ϕ	Rebar diameter
δ_y	Yield displacement
A	Total area of cross section
BRAD	Buckling restrained axial damper
BRB	Buckling restrained braces
BRBF	Buckling restrained braced frames
E	Modulus of elasticity
E_t	Modulus of elasticity after yielding
FEMA	Federal emergency management agency
M3	Flexural moment hinges
NLLink	Nonlinear link
OF	Original frame
PMM	Axial force-biaxial moment hinges
RC	Reinforced concrete

CHAPTER 1

INTRODUCTION

1.1 General

In the last few decades, there have been several studies on innovative approaches in order to better protect or strengthen the structures under the effect of external dynamic forces. The idea behind these innovative approaches which especially focus on the materials and systems is to limit the inelastic deformations in other structural members by dissipating the energy in itself (Symans et al., 2008; Housner et al., 1997; Soong et al., 1997; Soong et al., 2002). In this study, from these innovative approaches, buckling-restrained braces (BRBs), which have the advantage of low cost, ease of production, and installation (Kanaji et al., 2003; Farhat et al., 2009), were investigated.

In buckling restrained braces, there is a core that provides the axial strength to avoid material failure and there is a restraining section that provides the flexural rigidity to avoid buckling. An inert filler material such as infill concrete, mortar or grout is used to fill the space between the core and restraining section. In order to prevent excessive shear stress transfer that may occur when the brace is under compression, an unbonding material is also placed between the core and inert filler material. In BRBs, the aim is to obtain a yielding core that can deform longitudinally under tension and compression forces independent from buckling. Since lateral and local buckling behavior is restricted, high cyclic ductility is attainable (Sabelli et al., 2003). Based on these design approaches, the buckling-restrained braces provide stable hysteresis behavior having approximately the same axial yield force in tension and compression. In the last few decades, many researchers have been investigated the hysteresis behavior of different designs of buckling restrained braces (Sabelli et al., 2003; Watanabe et al., 1988; Wada et al., 1998; Merritt et al., 2003a, b; Koetaka et al., 2006; Iwata and Murai, 2006; Usami et al., 2008; Tsai et al., 2008).

Although, the use of buckling restrained braces are desirable for seismic design of new buildings for their higher ductile behavior; in the last decade its use for strengthening and rehabilitation of existing structures gain considerable attention (Ash and Bartoletti, 2009; Disarno and Manfredi, 2010; Güneyisi, 2012). Within this context, the aim of this study is to introduce buckling restrained braces designed with different stiffness and yield load as a seismic upgrading scheme for damage prevention capacity and to evaluate the upgrading effectiveness of using them in a reinforced concrete structure.

1.2 Outline of the Thesis

Chapter 1- Introduction: This chapter includes the brief summary of the literature review and background as well as the main objective of the thesis.

Chapter 2- Literature Review: It provides the historical background on empirical and theoretical applications of buckling restrained brace and retrofitting techniques in reinforced concrete structures such as jacketing of structural members, addition of extra structural members, and energy dissipation devices in previous studies.

Chapter 3- Methodology: This chapter involves the description of the two different reinforced concrete buildings and loading condition of the frame systems. Moreover, in this section, the design properties of buckling restrained braces as a retrofitting technique and the characteristics of the earthquake record used in the nonlinear dynamic analysis are given.

Chapter 4- Results and Discussion: In this chapter, the results obtained from nonlinear dynamic analysis for case buildings and retrofitted ones with different configuration of buckling restrained brace models are provided. The structural performance of the buildings is given in terms of maximum roof displacement, roof drift, maximum inter-storey drift ratio, load deformation behavior, etc.

Chapter 5- Conclusions: This chapter presents the conclusions of this study.

CHAPTER 2

LITERATURE REVIEW

2.1 Retrofitting Techniques in Reinforced Concrete Structure

2.1.1 Jacketing of Structural Members

One of the most popularly used method for strengthening of reinforced concrete buildings is jacketing of structural members. Concrete jacketing method, steel jacketing method, fiber reinforced polymer jacketing method are widely used. Increasing concrete confinement and shear strength by transverse fiber reinforcement and increasing flexural strength by longitudinal fiber reinforcement are intended by using jacketing method (Waghmare, 2011).

2.1.1.1 Concrete Jacketing Method

Concrete jacketing method is the most commonly used method for pre-earthquake or post- earthquake retrofitting structure. This method applies with different ways from time to time. This way is probably cast-in-place or shotcrete. It can be applied through the column, this method can also apply beam to column intersections point or beam (Chalioris and Pourzitidis, 2012).

Altun (2004) studied the application of the reinforced-concrete beam jacketing under the bending experimentally. He determined the mechanical properties of reinforced-concrete beams such as load–displacement behavior, ultimate load, ductility, and toughness before and after jacketing by reinforced concrete. As a result, pre-jacketing reinforced-concrete beam and the same beam post- jacketing exhibited similar performance under the bending. The shear stress of the beam under the bending parallel to interface of jacketing, and this interface area was so much. Because of the fact that, the reinforced-concrete beam jacketing restrained successfully in bending situation.

Jülio et al. (2005) studied on the influence of the added concrete on adhesion experimentally. This study was investigated the bond strength between the main concrete and added concrete which had different specimens. New mixed concrete added to existing concrete which increased bond surface. Then, slant shear tests were applied in order to determine the magnitude of bond strength in shear. This study showed the relation between strength of the existing and added concretes and their bond strength or rupture mode. Difference of strength between existing and added concrete in slant shear test, for same level of shear stress, arrived higher magnitude of normal stress.

Vandaros and Dritsos (2006) studied experimentally on the retrofitting of reinforced-concrete column. To strengthen the existing column, concrete jacketing was applied as a poured-concrete or shotcrete concrete. Longitudinal reinforcement of existing column and added reinforcement welded at interface of added concrete and existing concrete. This test included earthquake simulation for displacement controlled cyclic loading. They indicated that this method was significantly useful for retrofitting of column. It was also observed that buckling of column was restrained owing to welded reinforcements.

Tsonos (2008) investigated retrofitting of reinforced concrete columns and intersection of column and beam with cast-in-place method and shotcrete method to improve structure performance. Hence, in his study, reinforced cast-in place concrete and shotcrete jacket were investigated as two and four sided experimentally. Existing beam to column connection exhibited poorly behavior under the cyclic loading. The existing connection failed early when sample subject to seismic loading. The results of the tests showed that for both of two and four sided retrofitted specimens which were occurred flexural hinges in next beam exhibited more ductile behavior. It was also pointed out that energy dissipation capacity of shotcrete jacketing was worse than cast-in-place jacketing. This situation was based on shotcrete could not in full cover the jacketing reinforcement. Both of two and four sided reinforced shotcrete jackets and corresponding reinforced cast-in-place concrete jackets were closed to each other as retrofitting method for concrete columns and intersection of column and beam in existing structures.

Mourad and Shannag (2010) studied on a series of square reinforced-concrete column. This series of column was subjected to axial compression. After that, ferrocement jackets which containing two layer of welded wire mesh used to retrofit these specimens. All of the results were examined in point of load carrying capacity, lateral displacement, axial stress and strain, axial displacement, and ductility. As a result, in comparison to the existing column, the jacketing of reinforced concrete square columns by ferrocement had %26 and %33 higher axial stiffness and axial load capacity, respectively.

Naser and Zonglin (2011) investigated the retrofitting methods in order to exhibit better performance of structural members which was a part of bridge. In their study, concrete shotcrete jacketing method was used to be stiff and increase carrying capacity of the existing arch ring. Firstly, steel bars were placed on the existing building. After that, concrete mixture was sprayed to place steel bar and main bridge, as shown in Figures 2.1-2.4.

Figure 2.1 General view of placing the steel bars in arch rings (Naser and Zonglin, 2011)

Figure 2.2 Close view of arrangement of steel bars in arch rib (Naser and Zonglin, 2011)

Figure 2.3 Spraying the concrete (Naser and Zonglin, 2011)

Figure 2.4 The first layer of sprayed concrete (Naser and Zonglin, 2011)

Kaish et al. (2012) carried out an experimentally study on the influence of ferrocement jacketing on the short columns and corner retrofitting. Ferrocement jacketing was a retrofitting technique which square jacketing with mutable or single layers wire mesh. In their study, various types of ferrocement jacketed reinforced column and main short columns were compared. As a test result, improvement on load carrying capacity and displacement were observed due to the ferrocement jacketing.

Chaliois and Pourzitidis (2012) investigated retrofitting of shear damaged reinforced concrete beams for the purpose of altering their brittle failure mode to a more ductile one. Sample beams were subject to four point loading test, then damaged beams were retrofitted with self-compacting concrete jacket which include reinforcement. Results showed that self-compacting concrete jacketing technique was really useful method in order to restrain shear damage. Jacketed beams also exhibited more ductile behavior than beams which initial undamaged ones.

Raval and Dave (2013) studied on ten beams in which four beams had smooth surface, four beams had rough surface and two beams were not retrofitted. Reinforced-concrete beams were tested using two point loading system. It was observed that sample one which had smooth surface had superior for in case of using combined dowel connectors and bonding agent with micro-concrete. In other cases, sample one which had rough surface exhibited also superior performance.

2.1.1.2 Steel Jacketing

Steel jacketing method is generally used to retrofit reinforced concrete columns and bridges. Steel caging technique with angle and strip is one of the most used technique which in the other steel jacketing techniques. Steel jacketing reinforced concrete member is observed together with fiber reinforced polymer in some study (Giménez et al., 2009).

Kim and Shinozuka (2004) simulated nonlinear dynamic responses of the bridges with steel jacketing of bridge columns before and after retrofitting. This investigation included fragility curve improvement of two bridges which common type in southern California. Effect of retrofitting on fragile behavior of reinforced concrete columns of bridges were quantified by comparing results of after and before retrofitting. Column retrofitted with steel jacketing exhibited less fragile behavior than before retrofit.

Adam et al. (2008) performed a numerical study on the retrofitting of the axial loaded reinforced concrete column with steel caging by using finite element modeling. It was aimed to investigate the behavior of reinforced concrete column which retrofitted by steel caging method. Result of this investigation provided the detailed information about the effect of size of the angles, the yield stress of the steel of the cage, the compressive strength of the concrete in the column, the size of the strips, the addition of an extra strip at the ends of the cage, and the friction coefficient between the layer of mortar and the steel of the cage.

Giménez et al. (2009) investigated the design of steel angle and strip which using for retrofitting of column specimens with steel caging under the axial loading. Steel caging method based on installation steel angles to each corner of reinforced concrete columns joined with welded steel strips was utilized. Axial loading test was applied initially. Failure occurred columns in previous tests were retrofitted by added strips. Significant enhancement on the ultimate load carrying capacity and ductility of the column when strips and angles added was observed.

Su and Wang (2010) studied on strengthening with damaged rectangular reinforced concrete columns using pre-cambered steel plates. In their study, it was investigated that retrofitting approach which using pre-cambered steel planes with purpose of caring pre- compressed axial load. Experimental and theoretical findings were gained through the study. Pre-cambered plates were able to distribute the existing axial load in the existing column. Load carrying capacity, ductility, strength and deformation significantly increased by external steel plates. Moreover, it was suggested that the developed theoretical equilibrium model might be useful for future design.

Hamad et al. (2011) used steel caging for retrofitting reinforced concrete beams. For investigating the flexural behavior of beams which T cross-section, specimens were tested under two points loading in positive bending. Results of tests showed that the ductility due to the proposed strengthening and partial composite effect was increased in the flexural behavior. Spacing and number of the intermittent battens were more effective on behavior of the strengthened specimens.

Choi et al. (2012) observed the effect of steel wrapping jackets on the retrofitting of the circular reinforced concrete columns. Purpose of their study was determination of thickness of the steel wrapping jacket. The experimental study divided into two categories, namely, lateral bending test and column test. There was center line of specimens as a rebar. And there are concrete as a slender around the rebar. After external surface of specimen was covered by steel wrapping jacket, wrapping jacket was welded. It was observed that steel jackets increased the bond strength and ductile capacity. This study resulted in specification of the thickness of the steel wrapping jackets.

2.1.1.3 Fiber Reinforced Polymer Jacketing

Fiber reinforced polymer jacketing is also used to retrofit reinforced concrete structures. The popularity of fiber reinforced polymer jacketing has increasing day to day owing to material properties. For instance, high strength and high stiffness–to-weight ratio as well as light weight and excellent corrosion resistance for ease of application (Luccioni and Rougier, 2004).

Yalçın et al. (2005) retrofitted reinforced concrete columns which using carbon fiber reinforced polymer sheets to have better strength and ductility capacities. It was studied that how the carbon fiber reinforced polymer sheets are used effectively. Then, it was observed that the carbon fiber reinforced polymer sheets really effective for column after the test. Their ductility, energy dissipation capacity and strength characteristics were exhibit better performance. Yet, those tests also indicated that just a slight development for performance was received for the retrofitted lap-spliced column.

Özcan et al. (2007) investigated the use of carbon fiber reinforced polymer wrapping for non-ductile square reinforced concrete columns. Five specimens were selected as specimens in existing buildings. Then, lateral cyclic displacement and constant axial loading were applied to selected reinforced concrete columns. Lateral load carrying capacity and axial load capacity of retrofitted reinforced concrete columns were increased as results.

Sasmal et al. (2010) conducted a study to develop retrofitting procedure for gravity load designed reinforced concrete beams and columns. As a retrofitting approach, glass fiber reinforced polymer, carbon fiber reinforced polymer, and steel plate were used in different ways. Generally, glass fiber reinforced polymer was used to restrain the shear, carbon fiber reinforced polymer was used for the flexural upgradation, and in order to upgrade intersection point of beam-column, it was used steel plates. Upgraded specimens exhibited good performance in seismic performance under the cycling load.

Dai et al. (2011) applied polyethylene terephthalate fiber reinforced polymer composites in order to retrofit reinforced concrete square columns. Terephthalate fiber reinforced polymer composites had a larger tensile capacity compared to conventional fiber reinforced polymers. The experimental tests were planned to compare the performance of polyethylene terephthalate fiber reinforced polymer and aramid fiber reinforced polymer. It was confirmed that polyethylene terephthalate fiber reinforced polymer was an alternative to conventional fiber reinforced polymers for the seismic retrofit of reinforced concrete columns.

Chen et al. (2011) studied the seismic retrofitting of reinforced concrete beams by means of complete wrapping, U- jacketing or side bonding application of externally bonded fiber reinforced polymer composites. It was found that shear failure generally occurred side bonding or U-jacketing fiber reinforced polymer composite because of de-bonding of the fiber reinforced polymer. Improved performance of new shear strength model was observed. The consideration of contrary shear interaction resulted in an important enhancement in the structural performance.

Zhou et al. (2012) proposed a pin mechanical fastened technique as shown in Figure 2.5 and bolt mechanical fastened technique as shown in Figure 2.6 in order to restrict sliding displacement and improve bonding of reinforced concrete beams experimentally. Four point bending test was applied on six simple-span reinforced concrete specimens. Fiber reinforced polymer de-bonding problem was coped with hybrid bonded technique. Hybrid bonded technique provided improving cracking and yielding loads. Moreover, distribution and width of cracks were limited by hybrid bonded technique.

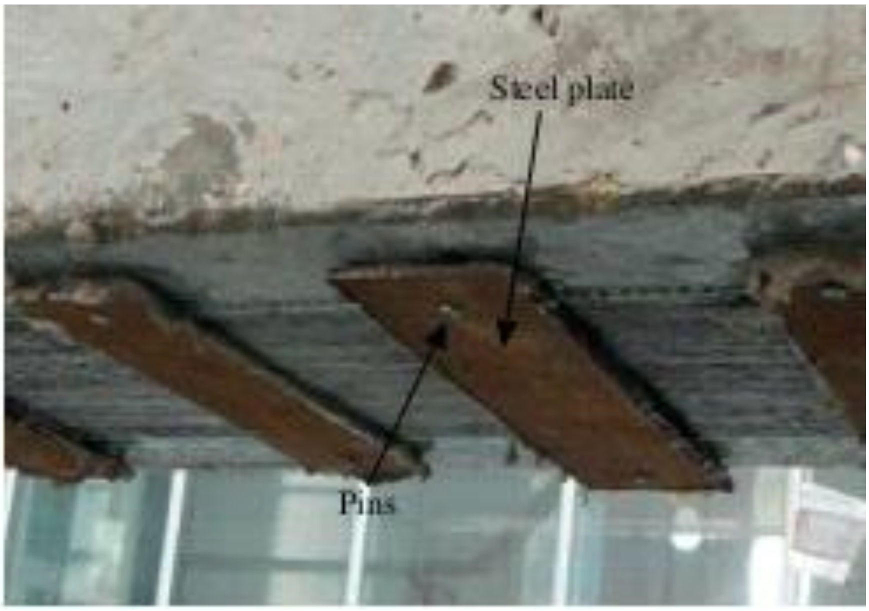


Figure 2.5 Pin mechanical fastened technique (Zhou et al., 2012)

Figure 2.6 Hybrid bonded technique (Zhou et al., 2012)

2.1.2 Addition of Extra Structural Members

2.1.2.1 Shear Walls

A great number of tall buildings have been retrofitted with shear walls to resist lateral forces which generated by wind and earthquake. The advantages of shear wall are increasing ductility, strength and stiffness, a large capacity for plastic energy absorption, and stable hysteretic characteristics in seismic resistant structures (Roberts, 1995).

Roberts (1995) studied experimentally about the use of steel plate shear wall in order to resist lateral forces which were affected by wind and earthquakes. It was investigated that the effect of steel plate shear wall based on ductility, strength and stiffness, a large capacity for plastic energy absorption, and stable hysteretic characteristics. Then, modeled specimen and stiffened modeled specimen were tested in dynamic analysis and drawn hysteretic behavior. The performance of the original and retrofitted specimens was evaluated.

Topkaya and Kurban (2007) carried out an investigation to calculate fundamental natural period of the reinforced concrete building which retrofitted with steel plate shear wall based on the numerical analysis. Comparisons of the fundamental natural periods of structures were found out with using linear three dimensional finite element analysis and estimated fundamental natural period which provided by seismic design specifications. It was shown that a simple hand method was very effective to predict the fundamental period of a steel plate shear wall. Geometrical and mechanical properties of the specimens were investigated based on that how much those properties effected on the fundamental natural period.

Topkaya and Atasoy (2008) investigated lateral stiffness of steel plate shear wall systems using finite element method and strip method. They also made comparisons with the experimental results. Advanced computer models of two systems were prepared. Experimental findings were compared with stiffness prediction which calculated with two prepared systems. As a result, approximation in suitable correctness was simpler than the traditional analysis techniques were found.

Jahanpour et al. (2009) performed semi-supported steel shear wall in order to alter to the conventional type of steel shear wall in ultimate capacity. It was compared finite element method with ultimate shear capacity when subjected to it the bending moments. Maximum shear capacity based on the lower bound theory of plastic design was determined by deciding ultimate shear capacity when it underwent the bending moments. It was reported that parallel results from the finite element analysis were found in a rapid and easy way.

Bhowmick et al. (2009) showed that periods which calculated from current finite element analysis of sequence of steel plate shear walls with different geometries were more confused than the code formula predicts periods. Period data which found from the analysis was used in the regression analysis. In this way, fundamental period of steel plate shear walls were improved by means of the regression analysis. Support conditions of the columns on the fundamental periods and perforations in the infill plate effects were also investigated.

Dan et al. (2010) used composite concrete-steel shear walls with steel covered profiles to horizontal resist for a structure which wanted great lateral load capacity in the theoretical study and the experimental tests. Specimens were selected as different configurations of the cross-section type of the steel covered element and steel shapes emplaced in the cross-section of the wall. Estimation of the dissipated energy, the deformation and the maximum load capacity based on nonlinear behavior was investigated for the case study specimens.

Rafiei et al. (2012) studied on new composite shear wall system consisting of two skins of profiled steel sheeting and an infill of concrete under in-plane loadings to resist lateral loads in steel framed building. The performance of surface tie constrains in the finite element model was compared with the contact surface model. Experimental results of the composite wall better approximated to the finite element model using contact surface in stresses, failure mode, and buckling. Then, finite element model using contact surface was investigated considering influence of material and steel–concrete interaction parameters, steel and concrete strength, interface connector spacing on the structural behavior of composite shear walls. Evidence about how to design of shear wall system consisting of two skins of profiled steel sheeting and an infill of concrete under in-plane loadings was found.

2.1.2.2 Steel Braces

Steel bracing of reinforced concrete frames is an applicable option to shear walls. Steel bracing method is applied with two ways which external or internal bracing. As with the external bracing technique, existing buildings are retrofitted by the way of binding a local or global steel bracing system to the exterior frames. When it comes to the internal bracing method, the retrofitting of the buildings could be accomplished by positioning a bracing system inside the bays of the reinforced concrete frames. External bracing method has advantages for internal architectural design. Different configuration of steel bracing system are commonly used such as concentrically braced frame, eccentrically braced frames, moment restrained braced frames. Several concentrically braced frame configurations are shown in Figure 2.7 (Gray, 2012).

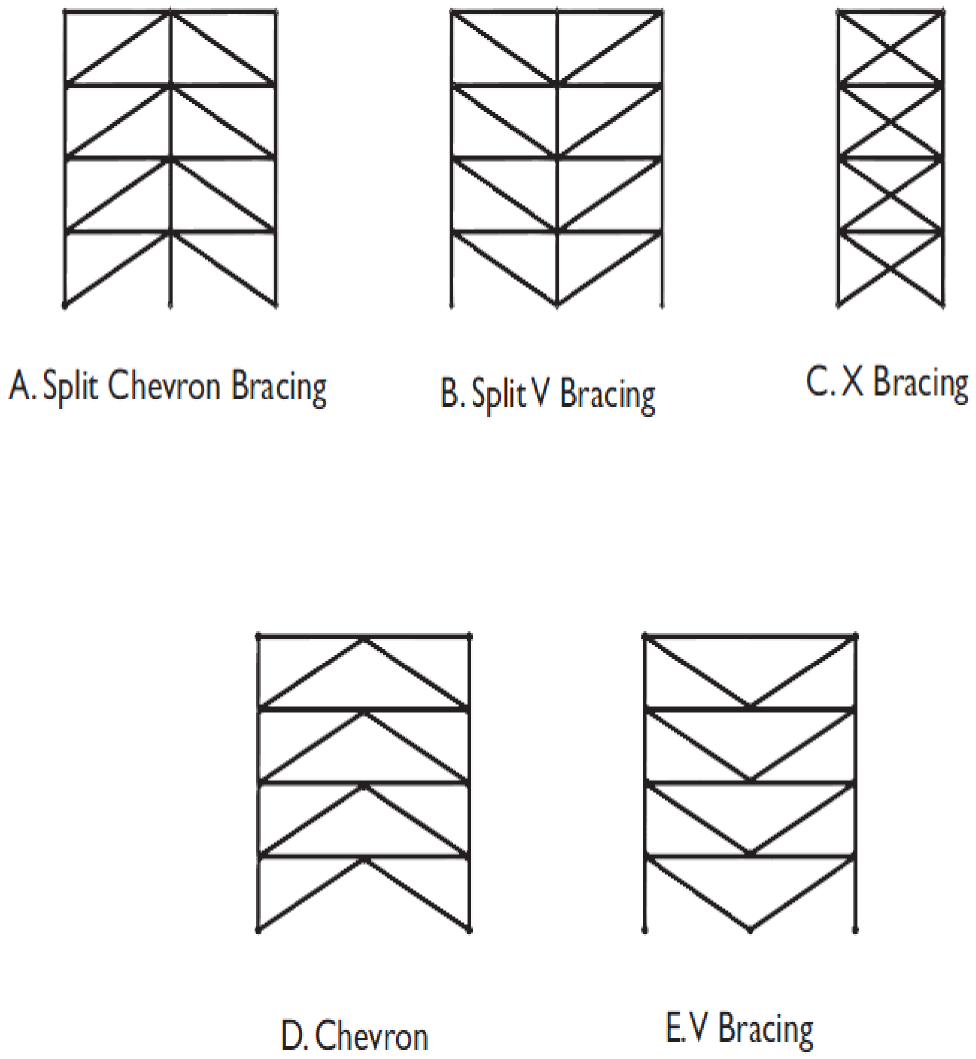


Figure 2.7 Several concentrically braced frame configurations (Gray, 2012)

Nateghi (1995) strengthened eight-storey reinforced concrete building which designed without seismic protection in Tehran. This building actually five stories but building owners decided to add three more stories added before retrofitting of this structure. The existing situation of the building became too weak because of the additional weight. It was reported that the retrofitting of the existing building using steel braces were very influential.

Maheri and Hadjipour (2003) studied about empirical examination and design of steel brace connection to reinforced concrete frame. It was investigated three type connections of brace which suitable for design of frames under construction and existing reinforced concrete frame which suitable for the seismic retrofitting. Brace members and connections of brace and reinforced concrete frame were subjected to direct tensile load and recorded until specimen were occurred ultimate failure. Tests results demonstrated the well-matched with prediction the individual elements in each connection.

Davaran and Far (2007) studied on an inelastic nonlinear model to resist cyclic load with investigating influence of low cycle fatigue of steel brace elements. Therefore, brace element was designed as pinned to a plastic hinge at mid-span and ends of brace. Low cycle fatigue impairment of the brace element was explained by simplification of the linear cumulative damage theory during the inelastic cyclic behavior. Experimental results indicated that it could predict the low cycle fatigue of brace elements by this model. Moreover, the rupture point of the brace element was anticipated satisfactorily by the model in one example.

In the study of Hagishita and Osaki (2007), heuristic method called scatter exploration was utilized in order to optimize the position of braces for steel frames with semi-rigid joints. For structural analysis, nonlinear analysis was used. The relation bending between moment and rotation of semi-rigid joints was evaluated comparatively.

Durucan and Dicleli (2009) presented a recommended seismic retrofitting system to increase the performance of reinforced concrete buildings which were weak for lateral load. Proposed seismic retrofitting system was a relatively new retrofitting method which was occurred with a yielding shear link connected between the frame and braces and also rectangular steel housing frame with chevron braces. It was applied on reinforced concrete building in order to increase strength, ductility, and stiffness properties of the building. In this study, the proposed seismic retrofitting system was compared with a traditional retrofitting system which using squat infill shear panels on a present office building and school. The original and 17 retrofitted buildings were subjected to nonlinear time history analyses considering three seismic performance levels. Result of analysis showed that the building which retrofitted with proposed seismic retrofitting system exhibited well energy dissipation with more constant hysteretic behavior than retrofitted with squat infill shear panels.

Özel and Güneyisi (2011) investigated the seismic resistance of a mid-rise reinforced concrete building retrofitted with different type and configuration of eccentric steel braces. The six storey mid-rise reinforced concrete structure was designed according to 1975 version of Turkish Seismic Code. Both of influence of distributing the steel bracing throughout the height of the reinforced concrete frame and influence of the type of eccentric steel braces in building was examined. The fragility curves were developed in terms of peak ground accelerations for four limit states. It was concluded that the retrofitted reinforced concrete building exhibited less fragile than the existing one, depending mainly on type and spatial distributions of steel braces.

Tisai (2011) purposed adequate supply of steel braces to increase developing collapse endurance of existing building with aim of resist sudden column loss. It was regarded that pseudo-static response analysis of single degree of freedom system with an idealized plastic-elastic when approach of design retrofit was improved. The stiffness and strength of added braces were determined from the connection between the structural characteristic and enhancement of collapse resistance. Incremental dynamic analysis was used to confirm exactness of the recommended approach. Nonlinear dynamic analysis results demonstrated that the proposed approach for the retrofitting was applicable for practical implementations based on the column loss reaction of the braced frames.

Salawdeh and Goggins (2011) found a numerical model for rectangular structural hollow sections and cold-formed steel-square as axial loaded members for restraining earthquake. Cold-formed steel brace models were subjected to axial loading for pseudo- static cyclic physical tests in order to improve and measure a robust numerical model that imitated the results from tests. Spread of plasticity along the element was regarded by designing a nonlinear fiber based beam-column element model. Energy dissipation, total energy dissipation, number of cycles to fracture, initial buckling load, and maximum tensile force were considered as a design parameter in this study. It was observed that the numerical model provided a good suggestion of the maximum measured test parameters.

2.1.3 Addition of Energy Dissipation Devices

2.1.3.1 Viscous Fluid Damper

Viscous fluid dampers consist of closed steel tube which filled with viscous fluid such as silicone oil, non-toxic, non-flammable, and stable for extremely long periods of time. And, a piston rod is connected to a piston head in closed steel tube as demonstrated in Figure 2.8. The piston can move in the cylinder. The damping action is provided by the flow of fluid from inside the piston head. The shape of the piston head determines the damping characteristics of viscous damper. The friction converts some of the earthquake energy to heat energy when the friction damper is established in a structure. The viscous fluid damper is generally installed as part of a building's bracing system using single diagonals (Soong and Dargush, 1997).

Sorace and Terenzi (2001) studied on fluid viscous spring-dampers as base-isolation systems of a building. Designs of the specimens were developed on the single degree of freedom system. Then, the specimens were tested. The damping coefficient values were appraised at the first level in the design problem. Simple building with very stiff superstructure was investigated in their study. The investigation was involved using different types of fluid viscous damper under different record of ground motions for retrofitting the building. Some of the case study buildings were discussed.

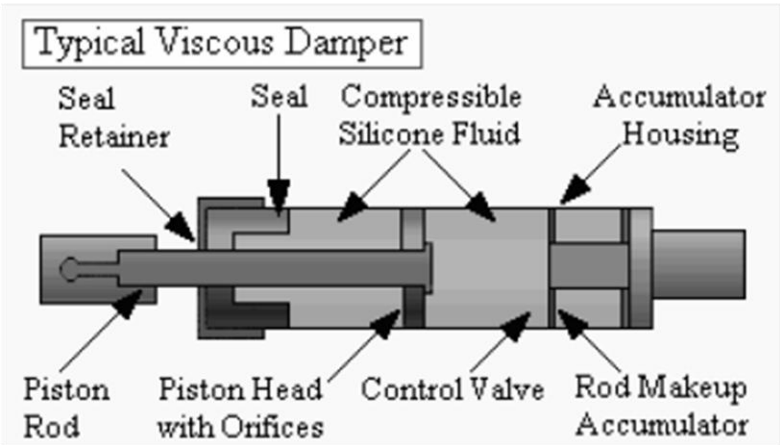


Figure 2.8 Typical viscous damper (Soong and Dargush, 1997)

Rodrigo and Romero (2002) investigated numerically dynamic response of a multi-story steel moment resisting frame upgraded with fluid viscous dampers under the lateral loads. Linear and nonlinear fluid viscous dampers were used for seismic retrofitting. Aim of their study was to apply and implement fluid viscous dampers in the structure. Results of this investigation indicated a suitable retrofitting choice which applied on six storey steel structure. It also demonstrated that nonlinear fluid viscous dampers could be decreased more than 35% linear viscous dampers in comparison.

Museros and Rodrigo (2005) evaluated a novel choice for diminishing resonant vibration of simply cantilevered beams under live loads. Linear fluid viscous damper was placed to connect the main beams and a supporting beam positioned under the existing one. Existing beam with the damping system was exposed to a sinusoidal excitation and analyzed in order to explore the dampers which minimized the dynamic response when resonance occurred. Results indicated that the resonant response of the existing beam decreased successfully with viscous fluid damper. Then, the proposed method was practiced to the existing bridges which were under rail traffic loads. Thus, their suitability was demonstrated for a wide range of circulating velocities.

Güneyisi and Altay (2006) developed fragility curves for comparative assessment of a number of retrofitting measures from analytical results of a multistory reinforced concrete building which built in Istanbul with adding three different fluid viscous dampers. Comparison of the fragility curves demonstrated that the fluid viscous damper was a smart device which was very effective against various earthquake ground motions and twice decreased in possibility of exceeding damage states could be succeeded by the installation of the passive fluid viscous damper systems into the reinforced concrete building.

Dicleli and Mehta (2006) investigated seismic performance of steel frames with viscous fluid dampers which placed as chevron braces and only chevron braced steel frames with regard to a function of ground motion and damper characteristics. Ground motions which had various frequency characteristics were categorized as large, moderate, and small intensity earthquakes. They were used to compare the results of nonlinear time history analyses of multiple and single storey chevron braced steel frames with viscous fluid dampers and only chevron braced steel frames. In addition, influence of the velocity exponent and damping ratio of the viscous fluid dampers on the seismic performance of the frames were investigated for this purpose. Because of brace buckling effects, seismic performance of the steel frames with chevron braces and without viscous fluid dampers was very poor and sensitive. Elastic behavior was supplied with installing viscous fluid dampers into the chevron braced steel frames. Thus, seismic performance of retrofitting could be improved. It was concluded that the influence of viscous fluid damper improved with larger damping ratio and smaller velocity exponents. But, if viscous fluid dampers were applied with damping ratios larger than 50%, it would not exhibit important extra development in the seismic performance of the chevron braced steel frames.

Rodrigo and Museros (2009) studied on the fluid viscous damper to implementation of dynamic performance of railway bridges which subjected to high-speed traffic. The main purpose of the study was optimizing the parametric quantity of retrofitting system and proving its capability under the movement of railway vehicles. Orthotropic plate model under harmonic excitement was firstly used for these objectives. Intensive sensitivity analysis of the plate response was applied and the governing parameters are extracted analytically. Three-dimensional finite element code was in particular programmed by the authors for this implementation. It was applied to an existing bridge which was a piece of the Spanish Railway network.

Aydın (2011) evaluated optimal damper positioning concerned with base moment in the steel frames. The equation of the transfer function and motion in terms of the fundamental natural frequency of the structures was evaluated with fourier transform. It was compared optimal damper design with other optimal damper methods which based on base shear, top absolute acceleration, and top displacement. A ten-storey steel planar frame was retrofitted by optimal dampers. As a result, this optimization supplied to reduce the base moment and inter-storey drift ratios in some frequency regions.

2.1.3.2 Tuned Mass Dampers

A device mounted in structures in order to decrease the amplitude of mechanical vibrations is called as tuned mass damper or harmonic absorber. This attachment occurs from a spring, a mass and a damping device which dissipates the energy created by the motion of the mass fastened to a vibrating main system to prevent discomfort, damage, or outright structural failure. If the structure begins to oscillate, the mass in tuned mass damper which has hydraulic or frictional constituent attached between mass and structure converts the kinetic energy to oscillate in opposite direction. The influence of a tuned mass damper is dependent on the mass ratio of the tuned mass damper to the structure itself. The value of the frequency of the structure should be equal to the frequency of the tuned mass damper in order to exceed ideal situation (Kwok and Samali, 1995).

Marano et al. (2007) compared conventional deterministic optimum design with different approaches for structural optimization which were multi-objective and robust single-objective optimum design methods. They used for this investigation tune mass damper in a single degree of freedom system to optimize the mechanical characteristics of the tune mass damper. As a result, preferable examination of the design resolution option was improved by using multi-objective robust design.

Sgobba and Marano (2009) investigated the design of tune mass damper system for inelastic structures. BouchWen hysteretic model was a single linear tune mass damper which was treated. Then, it was supposed to be affected to a single nonlinear degree of freedom system. Optimization problem was described previously considering the maximum standard deviation of displacement, average dismantled energy of retrofitted building with regards to un-retrofitted one and functional damage that covered the two indexes. Different parametric analysis and numerical examples based on these optimization criteria were investigated to find out the most suitable damping ratio and tuning frequency of the tune mass damper system. Consequently, it was proved that the tune mass damper was very useful for the protection of buildings under severe dynamic loadings.

Mohtat and Niri (2009) aimed at developing passive vibration controllers that would generalize robust design of tuned mass damper systems. The main idea of this study was based on the use of the linear fractional transformation formulation as well as the concept of balanced multi-output and multi-input norms for defining performance and a worst-case performance evaluation method. The robust design framework strategy was studied with care of this main idea. This strategy was transacted on many design multi-story building samples involving different combinations under the seismic loads. Random vibration analysis, time-domain simulations, uncertain frequency response plots, and visualization of Pareto fronts were used for the evaluation of the results.

Farshidianfar and Soheili (2011) studied on the optimization parameters for tuned mass dampers with the time domain analysis based on newmark method in order to decrease the earthquake vibrations of tall buildings. Tabas and Kobe earthquakes data and ant colony optimization method were used to provide the best parameters for the tuned mass dampers. To decrease the acceleration of stories and the maximum displacement, they were used different magnitude of damping coefficient, spring stiffness, and mass. Effect of soil type on the time response of structures and optimized parameters were reported. Moreover, how the ant colony optimization could be effectively utilized to design the optimum tuned mass dampers was shown.

Almazán et al. (2011) investigated the response of non-linear and asymmetrical linear structures retrofitted with one or two tuned mass dampers exposed to bidirectional and unidirectional seismic excitation. Tuned mass dampers decreased the edge deformation values differing from 20% to 50%. Where the deformation was greater, the highest reductions were obtained at the edges. It was found that as a general rule, if uncontrolled response was higher in any place of structure, tuned mass dampers would locate towards of this place. If the excitation was a narrow-band process, the tuned mass dampers would approximate to synchronize with the characteristic frequency of the excitation else, the tuned mass dampers approximated to synchronize with the frequencies related to the two translationally dominant modes of the structure. There was not important development resulting from connecting a second tuned mass damper because of the similarity of one and two tuned mass dampers.

2.1.3.3 Friction Dampers

Frictional dampers use the mechanism of friction for internalizing or dispersing the energy imparted to the dynamic systems. Frictional dampers are extensively used in mechanical systems in different industries in order to reduce the collision and vibration influences. Frictional dampers are also used in the structures as passive control devices to enhance the seismic behavior the structures. The damper consists of several parts of steel plates with slotted holes in them and they are screwed together. The plates can slide over one another, thus making friction at forces high enough. Friction dampers are designed to have moving parts that are to slip over one another during a powerful earthquake. The parts create friction that uses a certain amount of the energy from the earthquake that heads into the building when they slide over each other (Mirtaheri et al., 2010).

Bhaskararao and Jangid (2005) modeled two adjoining structures as single-degree-of-freedom system binded with a friction damper. They were derived from closed-form expressions to investigate analytical seismic responses during non-slip and slip forms. But, It was too complex that the origin of analytical equations for seismic responses of multi- degree-of-freedom structures in short times. Two numerical models of friction dampers of multi-degree-of-freedom structures were validated with the results of an example of single-degree-of-freedom structures. Decreasing the amount of structural responses such as displacement, shear forces and acceleration of connected adjacent structures was investigated as effectiveness of dampers. Suitable location and placement of the dampers were found to minimize the number of dampers. As a result, if the slip force of the dampers is properly chosen, adjoining structures of distinctive vital frequencies could influentially decrease earthquake-inflicted responses of the structures.

Ng and Xu (2006) performed a study on using variable friction dampers semi-active coupling control for a building to diminish seismic responses. It was established the numerical model of a building complex under earthquake stimulation with variable friction dampers. Then, a clipped control strategy was developed using linear quadratic gaussian control algorithms to work effectively. This developed control strategy involved viscous and reid friction controllers. The regulated homogeneous friction controllers and non-adhesive friction controllers were also developed. Each acceleration and storey drift reactions of control performance of each controller for the building complex with either single or multiple friction dampers was analyzed under different seismic loads. Semi-active coupling control was shown to reduce the seismic reactions of both buildings.

Lee et al. (2006) proposed a methodology for the design of the friction dampers based on the storey shear force distribution of an elastic building structure under the seismic load. Optimal rigidity ratios of the brace versus primary structure were found and the effect of the slip-load and brace stiffness was investigated from the numerical analysis of various single degree-of-freedom systems. This was applied by using two normalization methods for the slip- load of a friction damper. Five multi-storey building structures of different numbers of storey and natural periods were numerically analyzed. Proposed method compared with conventional design method based on the results of the numerical analysis.

Park et al. (2006) put forward a new equivalent linearization technique including a friction damper–brace system as an elasto-plastic system. Dispersed energy based on the probable distribution of the extremal displacement of the friction damper–brace system and secant stiffness were utilized by the proposed equivalent linearization technique. An existing model modernization technique was modified for comparative study. A small scale three-storey shear building model in which a rotational friction damper–brace system was installed and tested by shaking table for the purpose of verification. After that, the suggested technique and the model modernization technique were used to obtain equivalent linear systems. Results from the equivalent linear systems were compared with emprically acquired ones. Thus, the response quantities, modal damping ratios, and natural frequencies were estimated. Impacts of the maximum friction force on the suggested technique were analyzed by the mathematical analysis of single degree of freedom systems retrofitted with a friction damper–brace system.

Mirtaheri et al. (2010) investigated a novel type of frictional damper which contained the outer cylinder and the inner shaft. Properties and dimensions of these parts were selected according to the seismic demand of the structures. Considerable dissipation of mechanical energy was provided by the friction of these two parts of the damper. The results which obtained from tests of hysteretic behavior of cylindrical friction damper by mathematical and empirical methods demonstrated that desirable damper owned an important energy absorption by stable hysteretic loops subjected to earthquake loads.

Monir and Zeynali (2012) demonstrated the dynamic behavior of a lately developed friction damper which was occurred from nine high strength steel bolts and nine steel stripes. This square device was utilized in the diagonal bracing of structures. Firstly, prototype of the qualified friction damper was analyzed by using universal machine. Then, the friction damper was located to a single degree of freedom steel frame.

After that, this structure was tested with shaking table under a number of earthquake excitations. Thereafter, this structure was modeled by SAP2000 software for numerical assessment of the system analyzed with using seismic records which had been used during the shaking table tests. The existing model and model modified with a damper for a four story frames was analyzed under several seismic records in order to determine the response of damper in multi-storey buildings. The results of this study demonstrated that the energy which dispersed by the damping system and addition of this modified energy absorber decreased base shears and horizontal displacements of the multi-storey building.

2.1.3.4 Viscoelastic Dampers

Viscoelastic dampers involve layers of special high damping material between the steel plates and structure in order to decrease the acceleration occurring due to earthquake or wind forces and the motion amplitude by transforming same of the mechanical energy of earthquake or wind to heat. Typical viscoelastic damper is shown in Figure 2.9. These are passive dampers which have been established with success in several tall buildings and other structures (Samali and Kwok, 1995).

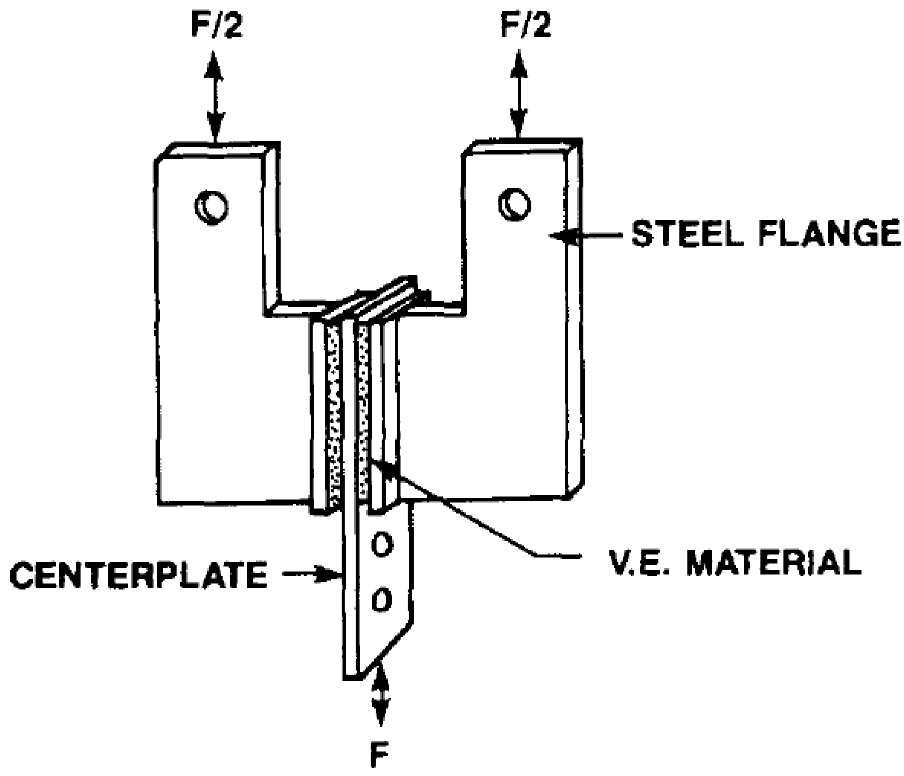


Figure 2.9 Typical viscoelastic damper (Samali and Kwok, 1995)

There are essential three approaches developed with a viscoelastic material as a damping device. First type is the direct utilization of a viscoelastic layer to the vibrating part (Figure 2.10a). The second type (Figure 2.10b) is developed from the first with adding new layer of a rigid material on top of the viscoelastic part, a constraint layer is formed. The third type and most used typical viscoelastic damper which is closely all of the deformation is in shear (Figure 2.10c). Each of these configurations has some advantage and disadvantages. However the third type is more adequate and is more influential when a lot of energies are occurred (Samali and Kwok, 1995).

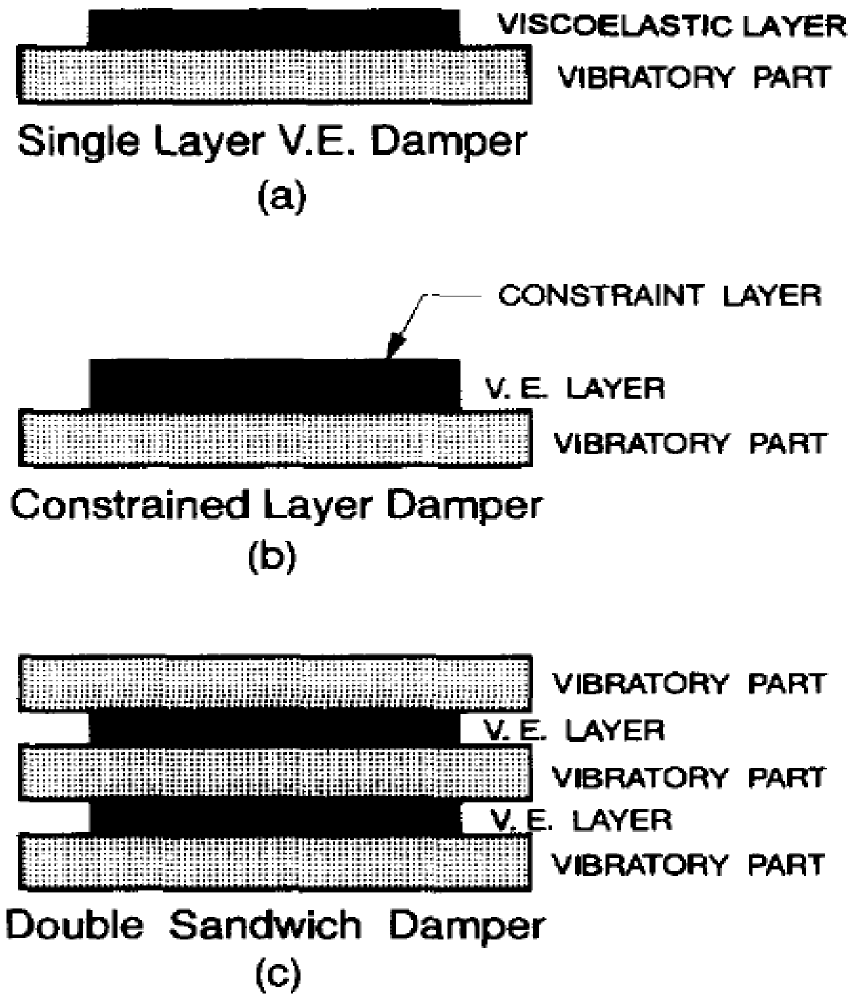


Figure 2.10 Different viscoelastic dampers configurations (Samali and Kwok, 1995)

Munshi (1996) presented the influence of viscoelastic dampers used as retrofitting of reinforced concrete elements in energy dissipation, ductility, and hysteretic response.

Different pinching characteristics and reduction was investigated on the reinforced concrete elements in this parametric study. DRAIN-2DX software was used to simulate pinching effects, strength decay and stiffness degradation of the reinforced concrete elements. Significant increase in the total energy dissipation of the system was provided by viscoelastic dampers. It was shown that optimum viscoelastic damping ratio depend upon the design ductility ratio and the period of the system.

Kim et al. (2004) studied the structural responses of the seismic joints or building–sky-bridge connections. Single-degree-of-freedom systems were retrofitted by 29 viscoelastic dampers and subjected to white noise and earthquake ground excitations. Then, parametric studies were conducted. It was demonstrated that viscoelastic dampers in definitive size minimized the dynamic responses of the structures. In addition, if the natural frequencies were so enough, such a scheme would be very effective. Then, dynamic analyses were conducted on 25-storey and 5-storey rigid frames connected to braced-frames. As a result, it was reported that the utilization of viscoelastic dampers in sky- bridges or in seismic joints could be suitable way for decreasing earthquake-induced responses.

García et al. (2006) investigated the torsional balance of asymmetric structures equipped with viscoelastic dampers. The horizontal deformation demand between structural members and unbalanced designs related to asymmetric behavior under seismic load was evaluated. Mass-eccentric structures were subjected to several seismic loads. Then, analytical and experimental response of stiffness was investigated. Controlling the lateral–torsional coupling was obtained by viscoelastic dampers. Results demonstrated that a small capacity damper could provide response reduction factors ranging from 1.5 to 3. Increasing stiffness or mass eccentricities affected linearly to optimal damper eccentricity values.

Ou et al. (2006) analyzed the response of structures with viscoelastic and viscous dampers. Models were connected to compose with viscous and viscoelastic dampers in series. Influencing several key parameters on the energy dissipation was investigated. Relationships of efficiency of the dampers and the parameters were determined by this investigation. It was developed that equivalent model for the passive energy dissipation system with the velocity-dependent dampers which could importantly simplify the dynamic analysis of structures. Finally, the analysis results of a multi-storey structure and single-storey structure with viscoelastic and viscous dampers were used to determine the effectiveness of the passive energy dissipation devices.

Pawlak and Lewandowski (2012) studied on the dynamic analysis of structures with viscoelastic dampers. A few models of viscoelastic damper was described for a single structure. Fractional derivatives were used to define the dynamic force deformation characteristics of damper. The structure was supposed as an elastic linear system. However, the equations of motion could include fractional derivatives as attachment to existing ones. Frequency domain equations of motion were determined by using continuation method. Formulation of a few results regarding the adequacy of the suggested method and fractional models of viscoelastic dampers was derived with using results of a numerical analysis.

2.1.3.5 Hysteretic Dampers

Hysteretic dampers are generally used for decreasing damage and collapse occurrence under seismic induced actions. They are also utilized seismic protection of new structures as well as seismic retrofitting of existing structure. As an advantage, they can be adopted with the aim of absorbing plastic deformation. And they can also be replaced if highly damaged after the destructive seismic event. After retrofitting interventions of existing structures, huge reduction of ductility demand in terms of inter-storey drifts or plastic rotations of existing members can be achieved through hysteretic dampers. All the various kind of hysteretic devices are generally based on the thermodynamic transformation of mechanical work into heat with the aid of plasticity of metallic members, viscosity of liquid components, and so on (Faella and Mazzolani, 2009).

Climent (2004) investigated ultimate energy absorption capacity and seismic performance up to collapse of existing non-ductile reinforced concrete frames which upgraded with brace-type hysteretic dampers. To test in shaking table, interior, and exterior connections without and with brace dampers were constructed. Dampers were designed to limit maximum inter-storey drift and to raise the horizontal stiffness of the connections. The model subjected to a sequence of seismic simulations was prepared. Brace- type hysteretic dampers provided excellently to reduce the maximum inter-storey drifts by 60%–80%. Moreover, it was observed that brace dampers raised ultimate energy dissipation capacity of the interior and exterior connections by 4 and 12 times, respectively.

Koetaka et al. (2004) presented a new beam-to-column moment connection which applicable for the column’s weak axis. The purpose in here was escaping complication encountered in situ welding. Therefore, they utilized bolts with the number of welds decreased. A wide-flange beam and column were connected each other by novel designed hysteretic dampers at the bottom flange and bolted splices at the top flange. They described the whole connection system and mechanical behavior of the damper. It was pointed out that the proposed system is capable of achieving stable hysteresis behavior in a large deformation range.

Basili and Angelis (2006) studied on a reduced order model for optimal design of two multi-degree-of-freedom structures connected by hysteretic dampers. A stochastic linearization technique was applied in order to simplify the problem since the passive connection was modeled as a nonlinear hysteretic element. Design procedure was described using the principle of virtual displacements for replacing the two multi-degree-of-freedom system, with a generalized two single-degree-of-freedom system. Each structure was represented by a hysteretic device interconnected by an elementary oscillator. Examples confirmed the entire methodology and also verified the effectiveness of hysteretic connection on seismic response.

Ok et al. (2007) proposed an optimal design method for nonlinear hysteretic dampers that increased the seismic performance of two adjacent structures. It was shown that the stochastic responses of coupled buildings without performing numerous nonlinear time-history analyses estimated efficiently the behavior of the structures. Therefore, nonlinear random vibration analyses were used with stochastic linearization method. The purpose was also to mitigate the total cost of the damper system. A multi-objective genetic algorithm was adopted in order to deal with such conflicting objectives. This approach systematically obtained several Pareto optimal solutions that were non-inferior or non-superior to each other. The process of Pareto solutions was also discussed to choose a reasonable design from the optimal surface. They considered passive-type magneto-rheological dampers with fixed input voltages as an example of a nonlinear hysteretic damping device. Furthermore, they determined optimal voltages and numbers of installed dampers. The robustness was examined with optimal design against uncertain characteristics of ground motions by extensive nonlinear random vibration analyses.

Oviedo et al. (2009) presented the constant yield storey-drift ratio as a deformation controlling scheme for the definition of the yield deformation of hysteretic dampers. And, this schema also supplied to observe its influence on the performance of the reinforced concrete structure which installed hysteretic dampers. A parametric study was performed through non-linear time-history analyses to evaluate the structural performance on a series of ten-storey reinforced concrete frames with hysteretic dampers. The results demonstrated that the scheme presented relation between constant distribution along the height of the reduction of floor displacements and seismic damage in the reinforced concrete main structure.

2.2 Buckling Restrained Braces

2.2.1 Definition

The idea of buckling restrained brace frames was born from the need to produce a symmetric hysteretic response. Hence, the compressive capacity of braces is increased while not affecting the tensile capacity of braces (Abraham, 2005).

A buckling restrained brace is also called as unbonded brace consists of two basic components. First one is a steel core element that carries the entire brace axial force, and second one is a perpendicularly restraining from axial load exterior element that prevents the core from buckling in compression and allowing it to fully yield in both tension and compression symmetrically, as shown in Figure 2.11. The concrete fill is effectively debonded from the brace. If buckling restrained braces have a stable hysteretic behavior, it would provide significant energy dissipation and large ductile capacity under the cyclic load (PEER, 2002).

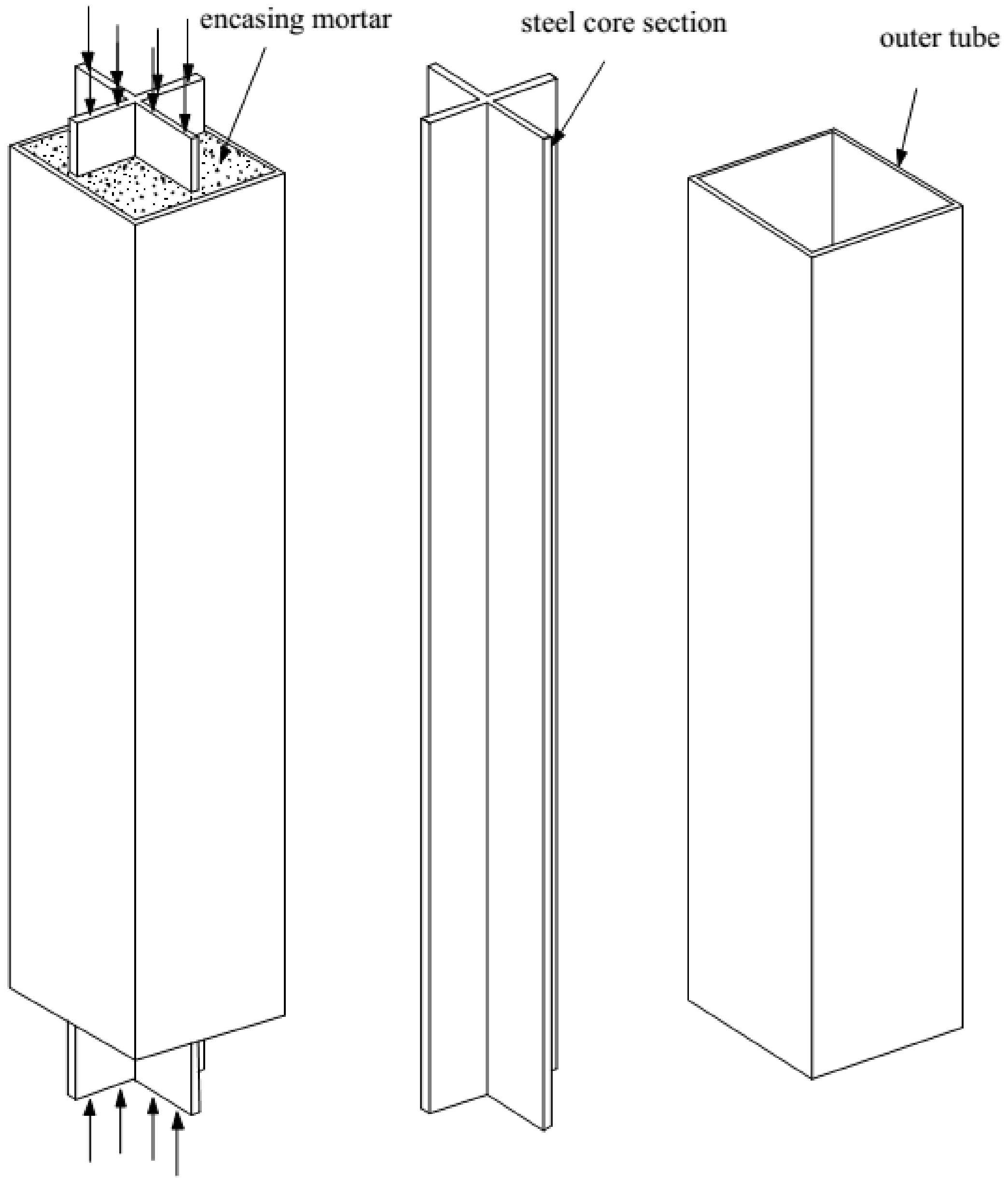


Figure 2.11 Schematic of the unbonded brace (left), the steel core (center), and the outer tube (right) (PEER, 2002)

D'Aniello (2007) reported that the buckling restrained brace was good protecting system for the reinforced concrete structure which was suffered from severe earthquake damages due to the fact that the buckling restrained brace provided stable energy dissipation capacity under seismic excitation.

2.2.2 Property /Behavior

Buckling restrained braces generally are occurred with composing a steel core which undergoing significant inelastic deformation when subjected to strong earthquake loads and a casing for restraining global and local buckling of the core element. Buckling restrained braces exhibit a stable hysteretic behavior with superb energy dissipation capacity (Kim and Choi, 2003).

Buckling restrained braces demonstrate well performance under the cyclic load with changing energy occurred by the earthquake. It is arrived to yield point of steel core which is independent of the encasing system. It thanks to the design of buckling restrained braces that provide to restrain buckling. And, this situation supplies to spend more energy by buckling restrained braces. Comparison of buckling restrained braces and traditionally braces are shown in Figure 2.12 (Eghtesadi, 2010).

These devices are greatly dissipated to energy input by a strong earthquake. They can make the rehabilitation easy if they are damaged after the earthquake. Because, buckling restrained braces are designed to be replaceable. Buckling restrained braces provide effective alternative to conventional braces for earthquake resisting systems. They can be used for both new and existing buildings. They have more specifications such as substance, energy absorption capability, etc. While the frames with conventional braces present irregular deformation under earthquake excitations, buckling restrained braces exhibit a more stable and symmetrical behavior in one or more stories. Response of buckling restrained braced system is more uniform along the height of the structure (Karimi and Arbabi, 2008).

Main structural members are protected under a strong earthquake, while damage due to yielding is occurred in the buckling restrained braces. These highly yielded devices can easily replaceable, thus replaced the resisting capacity of the structure remains constant. Energy dissipation of buckling restrained braces depends primarily on relative displacement of the hysteretic devices which have force-displacement response. Desired features of buckling restrained braces such as long term reliability, stable hysteretic behaviour, low-cycle fatigue property and insensitivity to environmental temperature are significant for retrofitting strategy (Calado et al., 2008).

The buckling restrained braces succeed symmetric and stable hysteretic behavior with suitable ductile compression yielding. Thus, It could be achieved the full theoretical capacity of the brace in both tension and compression by buckling restrained braces. Ductile behaviour of these devices could be predicted. Then, they have large plastic deformation capacity in both tension and compression. Thus, large amounts of seismic energy could be absorbed by buckling restrained braces (Eckert, 2009).

The use of buckling restrained braces supplies stable energy dissipation without strength or stiffness degradation thanks to restrain global and local buckling by suitable design. The inelastic deformations do not occur in the main frame under the earthquake. It can easily be replaced after the case that a destructive earthquake occurred. This is another significant advantage of the buckling restrained braces (Palazzo and Crisafulli, 2009).

The disadvantages of traditionally braced frames are the occurrence of buckling on the braces in compression. Occurrence of buckling causes limited energy dissipation capacity of a traditional brace. Hence, stiffness reduction and strength degradation can occur. As a result, in comparison to the traditional one, buckling restrained braces perform wide and stable hysteretic dissipation at large amplitudes (Sarno and Manfredi, 2010).

Since buckling restrained braces have the similar behaviour in both tension and compression, it can provide stable energy dissipation capacity even under severe seismic excitations. Damage occurred in buckling restrained braces is being large plasticization. This is expected as occurred within these added energy-dissipating members while other structural members are protected. In addition, buckling restrained braces avoid from global compression buckling in order to provide the effective solution to the problem of limited ductility of classic concentric bracing. Buckling restrained braces have the ability of bracing elements to yield inelastically in compression as well as in tension. Thus, this device changed more seismic energy to deformation energy (Eghtesadi, 2010).

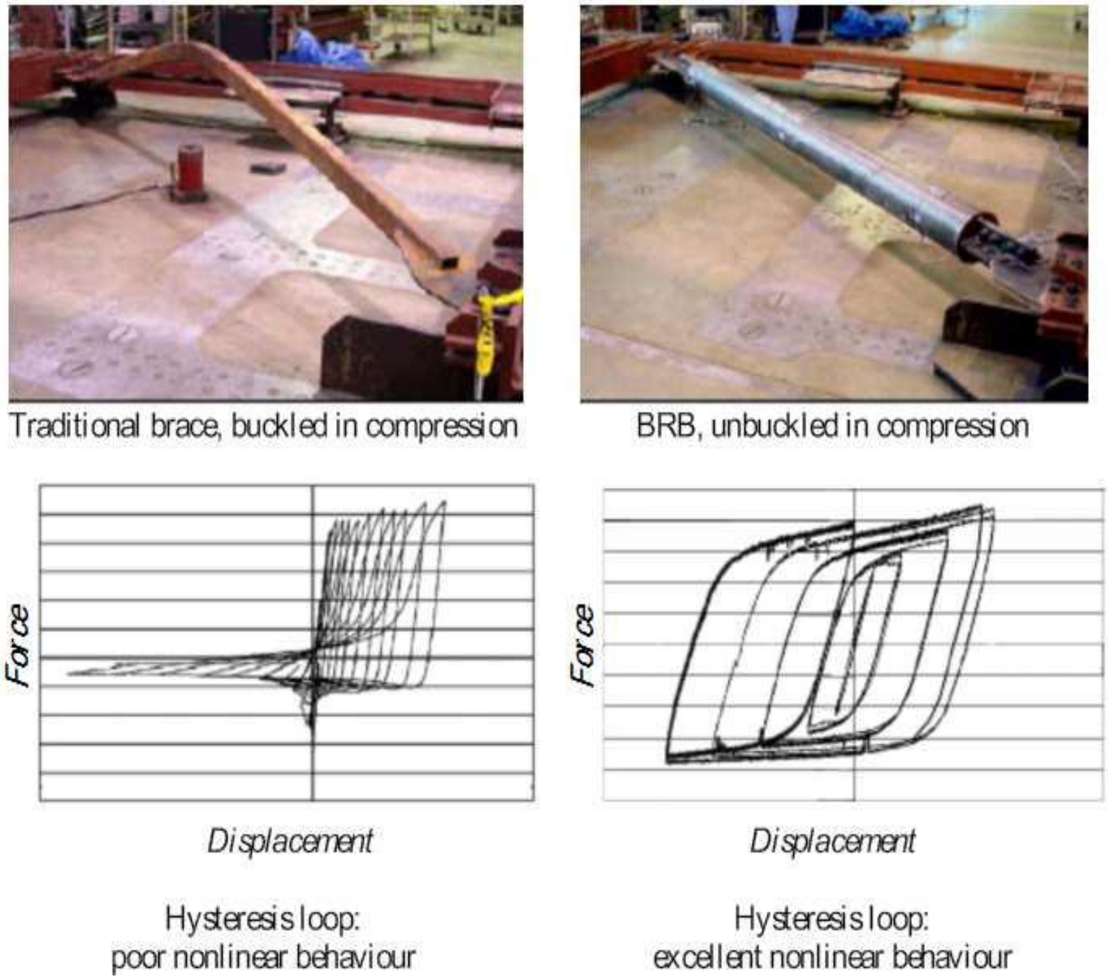


Figure 2.12 Traditional brace vs. buckling restrained brace (Eghtesadi, 2010)

Güneyisi (2012) studied on buckling restrained braces for seismic retrofitting of steel moment resisting framed buildings through fragility analysis. Samples of regular three-storey and eight-storey steel moment resisting frames were designed in earthquake-prone regions, then they were retrofitted with concentrically chevron conventional braces and buckling restrained braces. Nonlinear time history analyses were applied to analyze the structures with subjected to earthquake ground motion records with markedly different characteristics records. The original building with both conventional braces and especially buckling restrained braces exhibited significantly the good seismic behavior by increasing the median values of the structural fragility curves and reducing the probabilities of exceedance of each damage state.

2.2.3 Type of Buckling Restrained Braces

Tsai (2004) mentioned that cross section of the steel core member was usually bi-axially symmetric, could be a cruciform, an H or a flat bar shape. Restraining part could be constructed from reinforced concrete, mortar filled in the tube, reinforced concrete covered with fiber reinforced polymer or all-metallic steel tubes. Typical types of the buckling restrained braces are shown in Figure 2.13 (Eghtesadi, 2010).

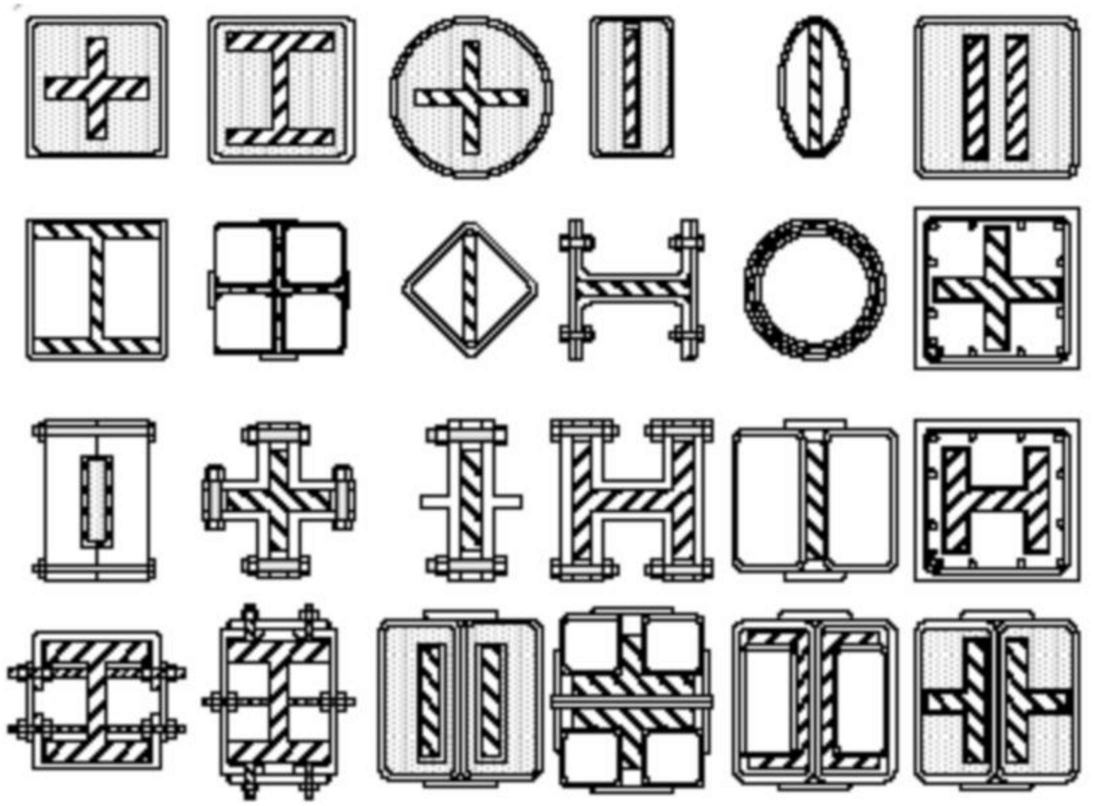


Figure 2.13 Typical types of buckling restrained braces (Eghtesadi, 2010)

The brace core can slide freely inside the restraint mechanism when axial loading is applied. Thus, there is not strain compatibility across the brace core-confining material interface. This situation requires the use of a debonding material such as epoxy resin, silicon resin, vinyl tapes, and combinations of these. It is also considered gap between the brace member and encasing material allow for transverse expansion of the brace (Abraham, 2005).

Many types of buckling restrained braces sections have been developed and currently used in engineering practice such as a rectangular or cruciform shaped yielding steel core members and a concrete filled tube section which is used in order to restrain outer steel casing. Other no filler material is used for the buckling restrained brace configurations, providing no unbonding material (Calado et al., 2008).

Moreover, the restraining tube is filled with concrete, and an unbonding layer is placed at the contact surface between the core plates and the filling concrete in the most classical form of unbonding braces. It has been proposed with two or more steel tubes in direct contact with the yielding steel plates. An adequate gap size between the brace and the restraining tubes is also required in case of "only-steel" buckling restrained braces in order to provide the necessary space for relative deformation between both members (Eghtesadi, 2010).

2.2.4 Application in New Building

Application of buckling restrained braces in new building is spread all around world due to their superior specification. These devices are used not only retrofitting structure but also they are used for new building in order to resist lateral seismic load. Some examples for the use of buckling restrained braces in new building are given in Figures 2.14-2.25.

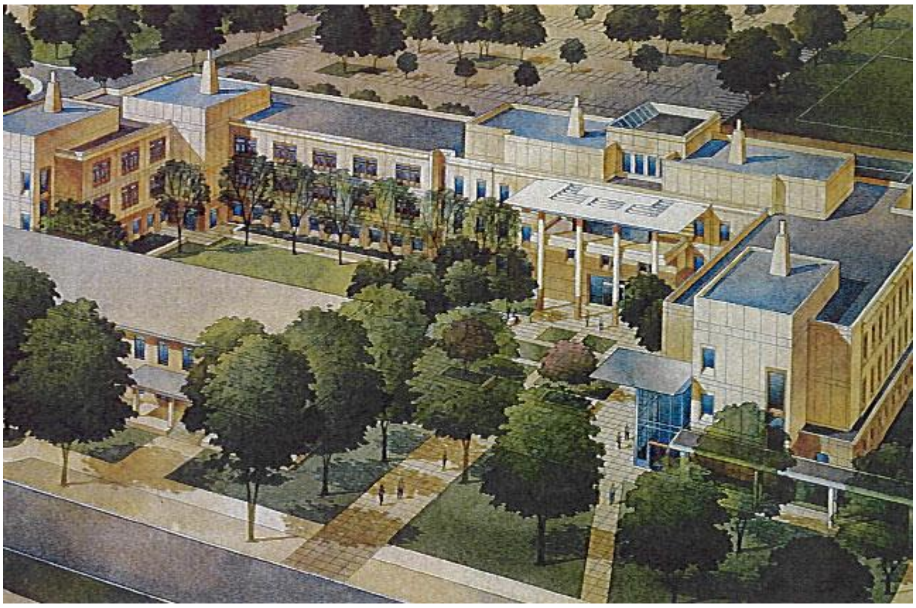


Figure 2.14 Plant and environmental science building at University of California, Davis, California (PEER, 2002)

Figure 2.15 Broad center for the biological sciences-California Institute of Technology, California (PEER, 2002)

Figure 2.16 Centralized dining and student services building-University of California, Berkeley, California (PEER, 2002)

Figure 2.17 Genome and biomedical sciences building-University of California, Davis, California (PEER, 2002)

Figure 2.18 Physical sciences building in University of California at Santa Cruz, California (PEER, 2002)

Figure 2.19 Second research building (Building 19B) at University of California, San Francisco, California (PEER, 2002)

Figure 2.20 Kaiser Santa Clara medical center hospital building: Phase I, Kaiser Permanente, Santa Clara, California (PEER, 2002)

Figure 2.21 Construction site of gymnasium (Chinese Culture University) (Tsai et al., 2004)

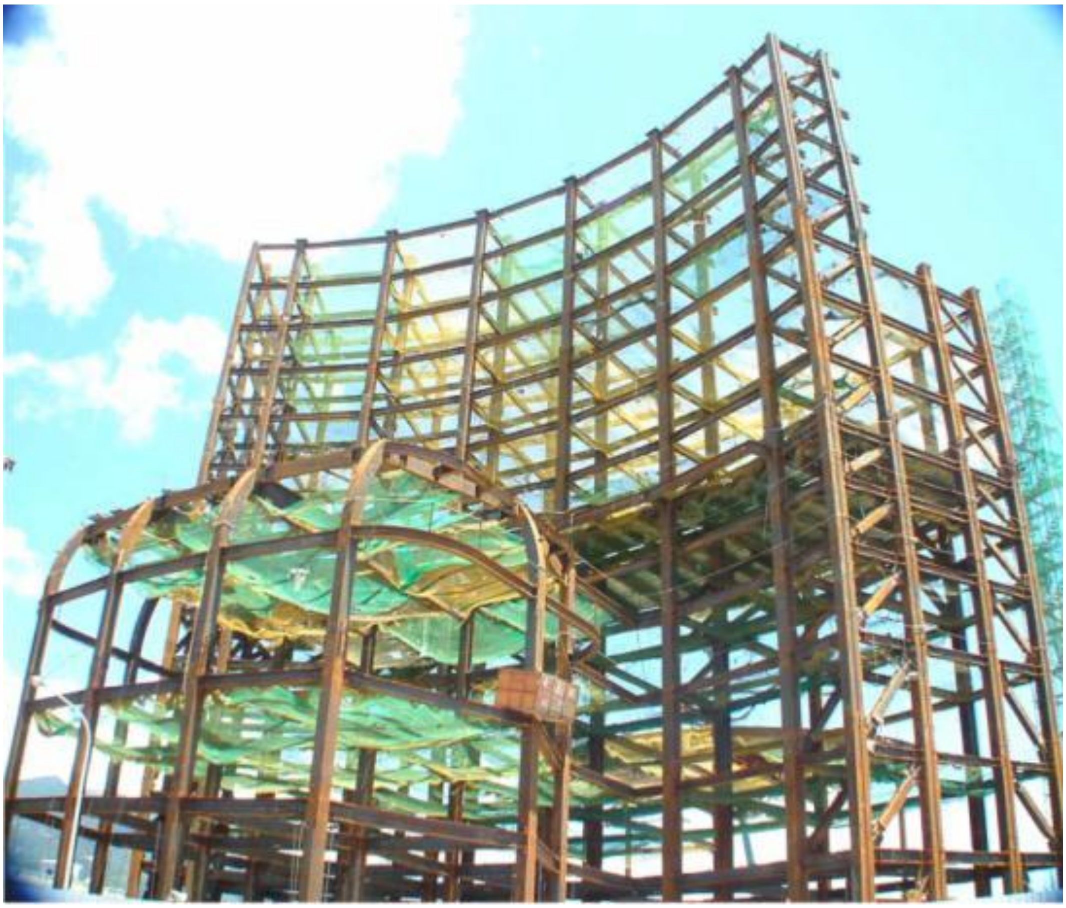


Figure 2.22 Construction site of Tzu-Chi TV station (in Taipei) (Tsai et al., 2004)

Figure 2.23 Buckling restrained braced frame detail of Tzu-Chi TV Station (in Taipei) (Tsai et al., 2004)

Figure 2.24 The 3D model of Shee-Hwa United World Tower and BRB details (in Taichung) (Tsai et al., 2004)

Figure 2.25 Two BRADs installed in the new building of University of Ancona (Eghtesadi, 2010)

2.2.5 Application in Existing Buildings

There are same applications for the purpose of the seismic retrofit of the existing buildings. Some of the related retrofitting applications are given in Figures 2.26-2.33.

Figure 2.26 Wallace F. Bennett Federal building (Brown et al., 2001)

Figure 2.27 Unbonded braces at fabrication plant, Chino, Japan and views of shear transfer assembly as well as brace connection during construction (Brown et al., 2001).

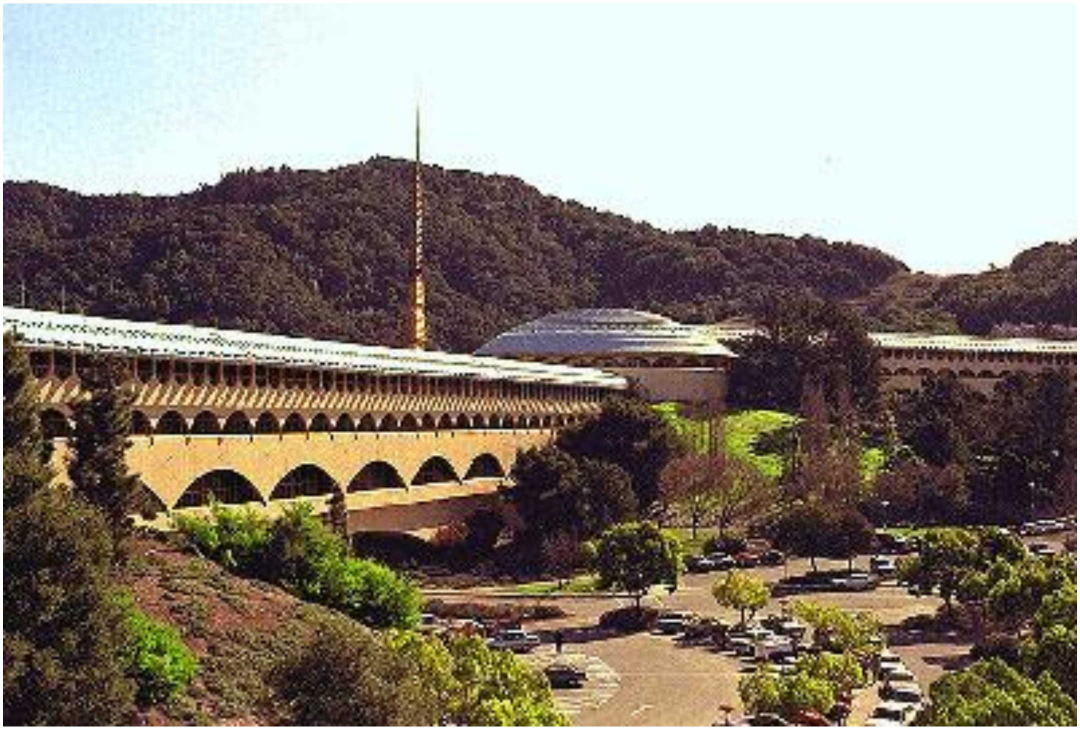


Figure 2.28 Marin county civic center hall of justice-County of Marin, California (PEER, 2002)

Figure 2.29 Hildebrand hall-University of California, Berkeley, California (PEER, 2002)

Figure 2.30 Building Corvallis campus— Hewlett-Packard, Corvallis, Oregon (PEER, 2002)

Figure 2.31 King county courthouse king county, Seattle, Washington (PEER, 2002)

Figure 2.32 The tested reinforced concrete structure in its original condition (Bagnoli Naples, Italy) (D'Aniello et al., 2009)

Figure 2.33 Damage pattern: +3% of inter-story drift ratio (D’Aniello et al., 2009)

CHAPTER 3

METHODOLOGY

3.1 Description of Sample Structures

3.1.1 Original Frame

Three and six storey reinforced concrete (RC) buildings were selected in order to compare the seismic response of the original structures and retrofitted structures considering the addition of buckling restrained braces with different design properties. Both of original buildings which consist of four bays on each direction and which is regular in shape and symmetric in plane was chosen in order to carry out the analysis on two-dimensional models which ease the interpretation of the results of analysis. Typical floor plan and elevation of the selected buildings are given in Figures 3.1 and 3.2.

The material properties such as the uniaxial compressive strength of concrete and the yield strength of both longitudinal and lateral reinforcement were assumed to be 16 MPa and 220 MPa, respectively. The dimensions of the columns varied with the storey height, as shown in Tables 3.1 and 3.2 while all the beams had the same cross-sectional properties as 250x500 mm. For these structural members, the reinforcement details are also given in Tables 3.1 and 3.2. Moreover, for the potential plastic hinge zones, the spacing of the lateral reinforcement was taken as 150 mm. In modeling of the frame, as gravity loads, dead load, and live load were taken into consideration. In the calculation of the dead load, weight of the structural members and weight of the masonry infill walls were included. The live load was taken as 2.0 kN/m^2, which is typical for residential buildings.

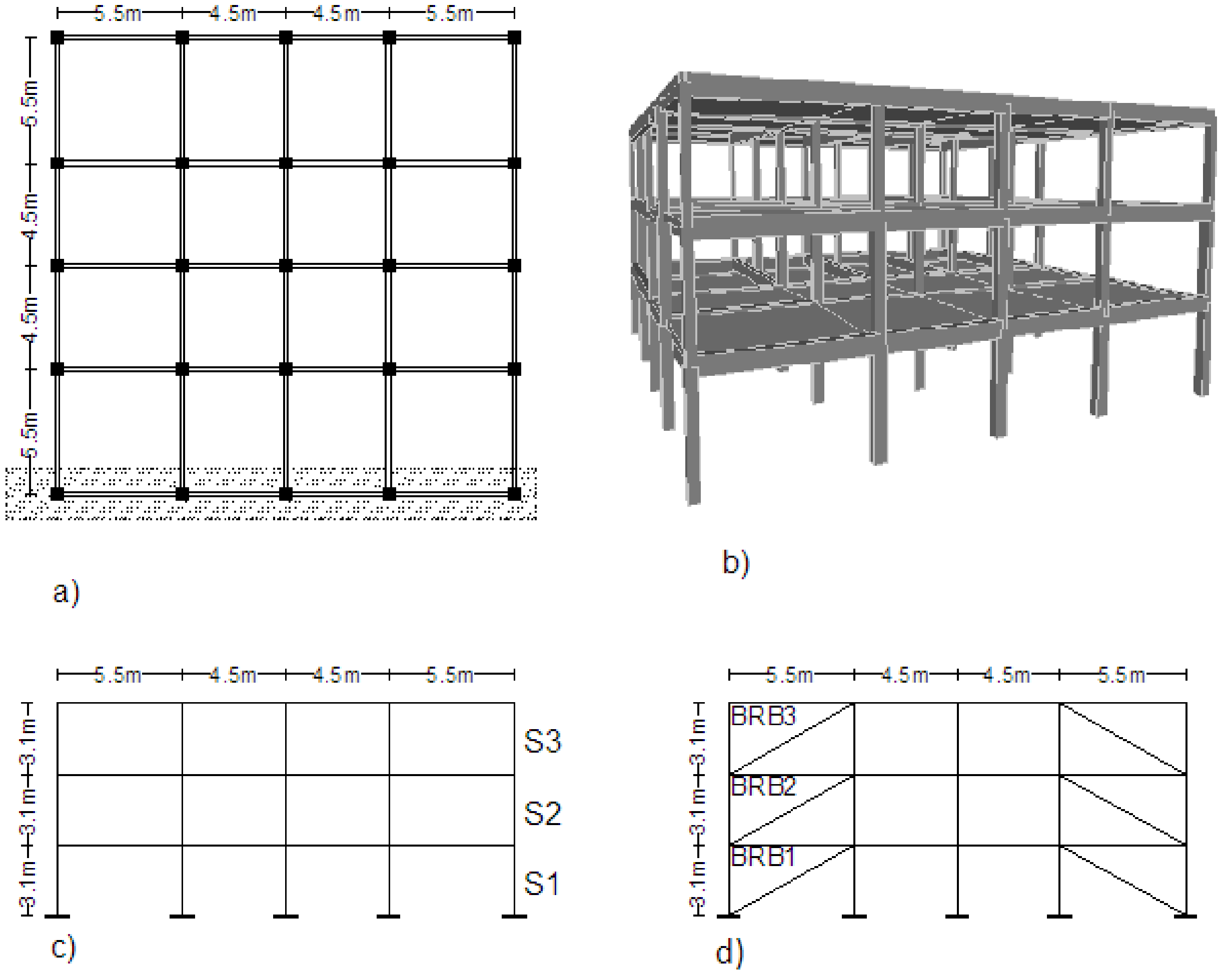


Figure 3.1 A layout for a) a floor plan, b) 3-dimensional view, c) elevation of the original frame, and d) elevation of the 3 storey buckling restrained braced frames

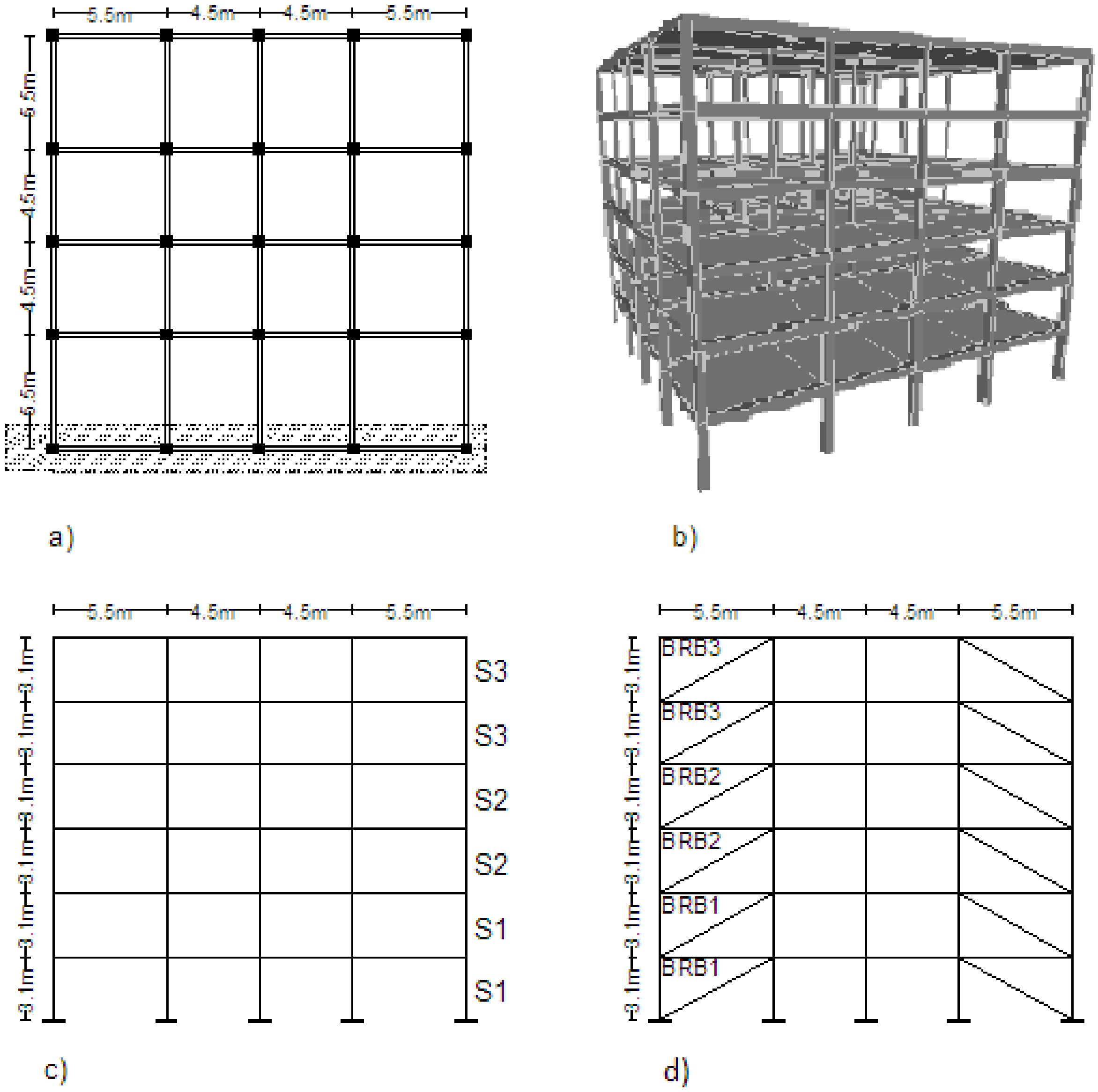


Figure 3.2 A layout for a) a floor plan, b) 3-dimensional view, c) elevation of the original frame, and d) elevation of the 6 storey buckling restrained braced frames

Table 3.1 Properties of columns in the 3 storey original frame and the core area for buckling restrained braces in BRBFs

Storey Level	Original Frame			Buckling Restrained Braced Frame	
	Column	Dimensions (mm)	Reinforcement	Buckling Restrained Brace	Core Area (mm^2)
1	S1	400x400	8ϕ16	BRB1	600
2	S2	350x350	8ϕ14	BRB2	450
3	S3	300x300	6ϕ16	BRB3	200

Table 3.2 Properties of columns in the 6 storey original frame and the core area for buckling restrained braces in BRBFs

Storey Level	Original Frame			Buckling Restrained Braced Frame	
	Column	Dimensions (mm)	Reinforcement	Buckling Restrained Brace	Core Area (mm^2)
1	S1	400x400	8ϕ16	BRB1	600
2	S1	400x400	8ϕ16	BRB1	600
3	S2	350x350	8ϕ14	BRB2	450
4	S2	350x350	8ϕ14	BRB2	450
5	S3	300x300	4ϕ16	BRB3	200
6	S3	300x300	4ϕ16	BRB3	200

The analytical model of the frames including nonlinear properties of the structural members was obtained by using SAP 2000 Nonlinear version 14.0 which is a general purpose structural analysis program (SAP 2000). Lumped plasticity approach was utilized and the nonlinearity was taken into account by adopting plastic hinges with hysteretic relationships based on FEMA-356 (FEMA 356) to each end of the beam and column members. For the column members, axial force and biaxial moment hinges (PMM) and for the beams, flexural moment hinges (M3) were considered.

3.1.2 Buckling Restrained Braced Frames

In this study, buckling restrained braces with different design properties were utilized as a means to upgrade the seismic response of the original structure, and the effectiveness of the buckling restrained braces were investigated comparatively for the purpose of seismic retrofitting. In the study of Kalyanaraman et al. (1998), it was shown that for the buckling restrained braces having non-prismatic cores, by changing the reduced area of the core and by changing the length of the core with reduced area; it was possible to obtain wide range of stiffness and strength for the buckling restrained braces. They also showed that by using this property, it was possible to attain the desired inter-storey drift and energy absorption in the structures. Furthermore, in the study of Kumar et al. (2007), the parameters related to hysteretic behavior of the buckling restrained braces such as the stiffness in the elastic and post-elastic range were given in terms of the reduced area of the core and the length of the core with reduced area. The ratio of the reduced area of the core to the total area of the section was denoted with α ($\alpha = A_{reduced}/A_{total}$), and the ratio of the length of the core with reduced area to the total length of the brace was denoted with β. Thus, the hysteretic behavior of the buckling restrained brace was plotted in Figure 3.3 is defined in terms of α and β by using Equations 3.1 and 3.2:

$$k_1 = \frac{AE}{l\left(\frac{\beta}{\alpha} + (1-\beta)\right)} \; for\ \delta < \delta_y \tag{3.1}$$

$$k_2 = \frac{AE}{l\left(\alpha\frac{E_t}{E} + (1-\beta)\right)} \; for\ \delta \geq \delta_y \tag{3.2}$$

In these equations, A is the total area of the cross section, E is the modulus of elasticity whereas E_t is the modulus of elasticity after yielding and $\delta_y = f_y \alpha \frac{A}{k_1}$ is the yield displacement.

In the current study, buckling restrained braces with different hysteretic behavior defined following the approach stated by Kumar et al. (2007) were introduced into the exterior bays of the original reinforced concrete frame, as shown in Figures 3.1 and 3.2. The total area of the core with respect to storey level was kept constant for the frames with non-buckling braces, as given in Tables 3.1 and 3.2. In the analytical model, the hysteretic behavior of the buckling restrained braces was modeled with nonlinear link (NLLink) members.

For the ratio of α which was the reduced area of the core to total area, 0.25, 0.50, 0.75 and for the value of β which is the ratio of the length of the core with reduced section to the total length, 0.25, 0.50, 0.75, 1.0 were considered. Therefore, 12 combinations of these α and β values were utilized in buckling restrained braced frames (BRBFs). The set of BRBFs assessed in this study including their modal properties are summarized in Table 3.3 for 3 storey building and Table 3.4 for 6 storey building.

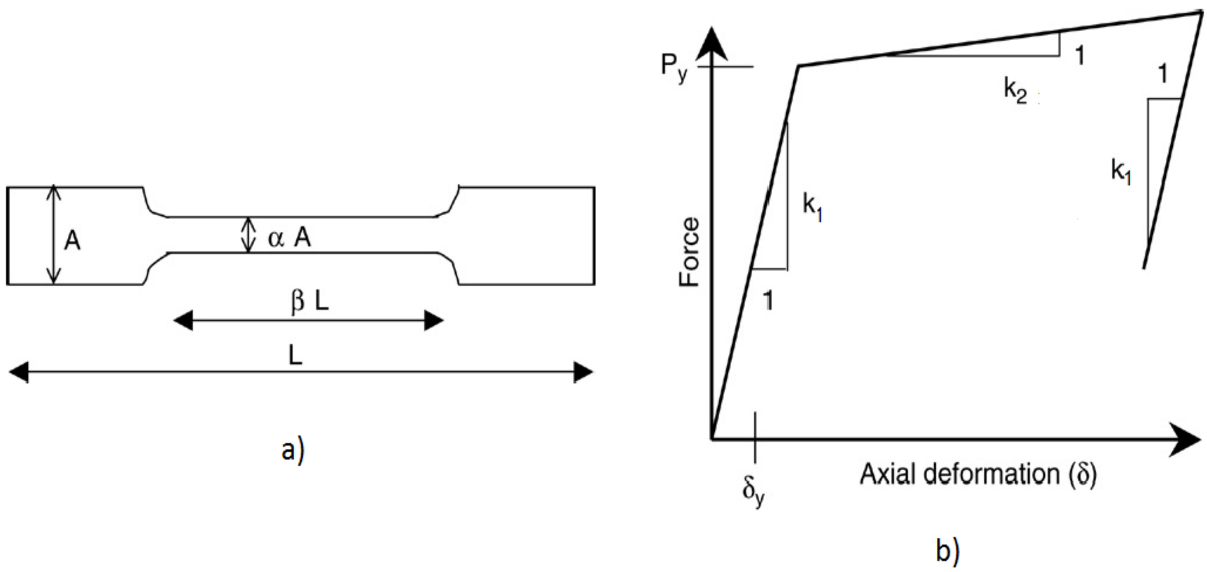


Figure 3.3 Views of a) Core element and b) hysteretic behavior of buckling restrained braces (Kumar et al., 2007)

Table 3.3 Free vibration periods of the original and buckling restrained braced frames for 3 storey building

Frame System	α	β	Period (s)		
			1st	2nd	3rd
OF	-	-	0.36	0.14	0.08
BRBF100100	1	1	0.30	0.12	0.07
BRBF075100	0.75	1	0.31	0.12	0.07
BRBF050100	0.5	1	0.32	0.13	0.07
BRBF025100	0.25	1	0.34	0.13	0.08
BRBF075075	0.75	0.75	0.31	0.12	0.07
BRBF050075	0.5	0.75	0.32	0.12	0.07
BRBF025075	0.25	0.75	0.33	0.13	0.08
BRBF075050	0.75	0.5	0.30	0.12	0.07
BRBF050050	0.5	0.5	0.31	0.12	0.07
BRBF025050	0.25	0.5	0.33	0.13	0.08
BRBF075025	0.75	0.25	0.30	0.12	0.07
BRBF050025	0.5	0.25	0.30	0.12	0.07
BRBF025025	0.25	0.25	0.32	0.12	0.07

Table 3.4 Free vibration periods of the original and buckling restrained braced frames for 6 storey building

Frame System	α	β	Period (s)		
			1st	2nd	3rd
OF	-	-	0.70	0.26	0.15
BRBF100100	1	1	0.57	0.22	0.13
BRBF075100	0.75	1	0.59	0.23	0.13
BRBF050100	0.5	1	0.62	0.23	0.14
BRBF025100	0.25	1	0.66	0.25	0.14
BRBF075075	0.75	0.75	0.59	0.22	0.13
BRBF050075	0.5	0.75	0.61	0.23	0.14
BRBF025075	0.25	0.75	0.65	0.24	0.14
BRBF075050	0.75	0.5	0.58	0.22	0.13
BRBF050050	0.5	0.5	0.60	0.23	0.13
BRBF025050	0.25	0.5	0.63	0.24	0.14
BRBF075025	0.75	0.25	0.57	0.22	0.13
BRBF050025	0.5	0.25	0.59	0.22	0.13
BRBF025025	0.25	0.25	0.61	0.23	0.14

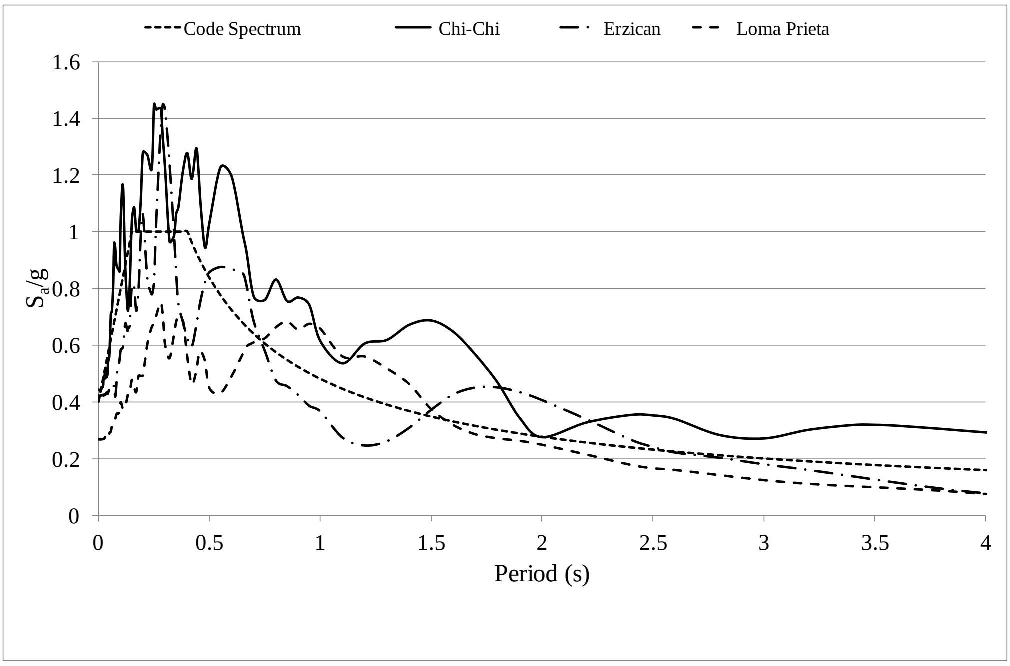
3.2 Nonlinear Dynamic Analysis

In order to examine the effects of different structural characteristics of buckling restrained braces on seismic response of the structures, the seismic behavior of the unbraced and buckling restrained braced frames were investigated under different earthquake ground accelerations. For this, nonlinear dynamic analysis was performed. In the nonlinear time history analysis, analytical models consisting the nonlinear behavior of the structural members as described in the previous sections were subjected to earthquake ground accelerations.

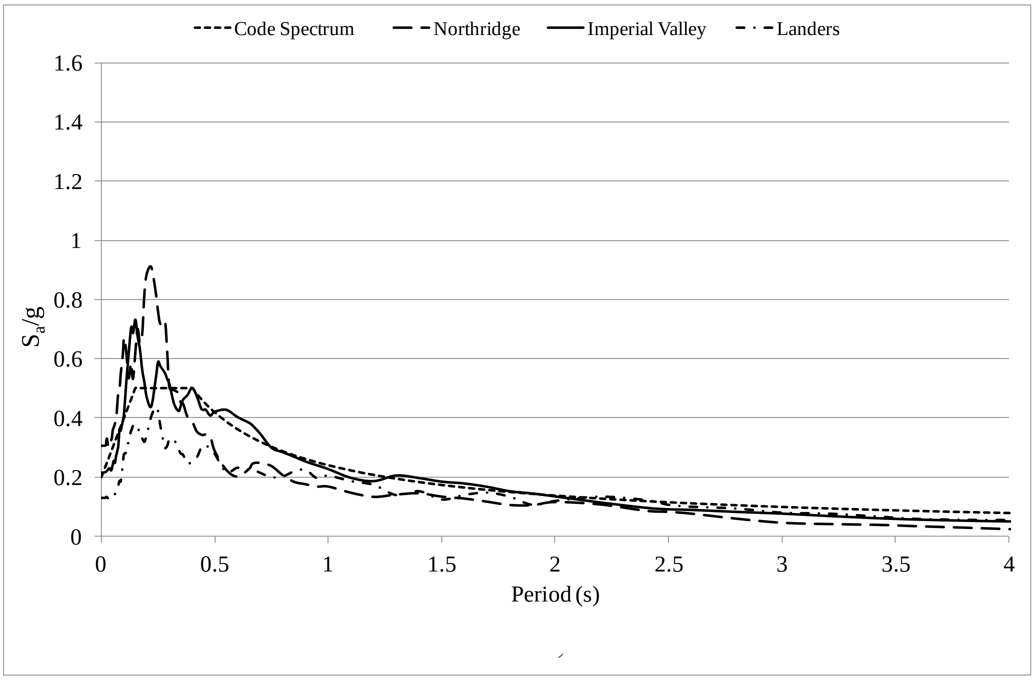
For the nonlinear dynamic analysis of the frames, a set of natural ground accelerations were generated as spectrum compatible were utilized (PEER, 2011). Two levels of seismic hazard for the design code spectrum were considered such as: 10% and 50% probability of exceedance in 50 years period. The comparison between the design code spectrum and elastic spectra of the scaled natural ground accelerations are given in Figure 3.4. Moreover, the characteristic properties of the natural ground motions such as the magnitude (M_w), the peak ground acceleration (PGA), the peak ground velocity (PGV), peak ground displacement (PGD), and characteristics of the site where acceleration recorded are listed in Table 3.5.

Table 3.5 Characteristics of the selected ground accelerations

Seismic Hazard Level	Earthquake Record	Year	Magnitude (M_w)	V_{s30} (m/s)	PGA (g)	PGV (cm/s)	PGD (cm)	Scale Factor
10% probability of exceedance in 50 years	Chi Chi	1999	7.62	504.4	0.443	139.20	146.42	2.0
	Erzincan	1992	6.69	274.5	0.420	45.29	16.52	1.0
	Loma Prieta	1989	6.93	594.5	0.267	52.96	13.95	2.1
50% probability of exceedance in 50 years	Northridge	1994	6.69	336.2	0.600	31.07	10.46	0.5
	Imperial Valley	1905	6.53	202.3	0.430	55.33	32.82	0.5
	Landers	1992	7.28	271.4	0.128	19.00	9.25	1.0



a)



b)

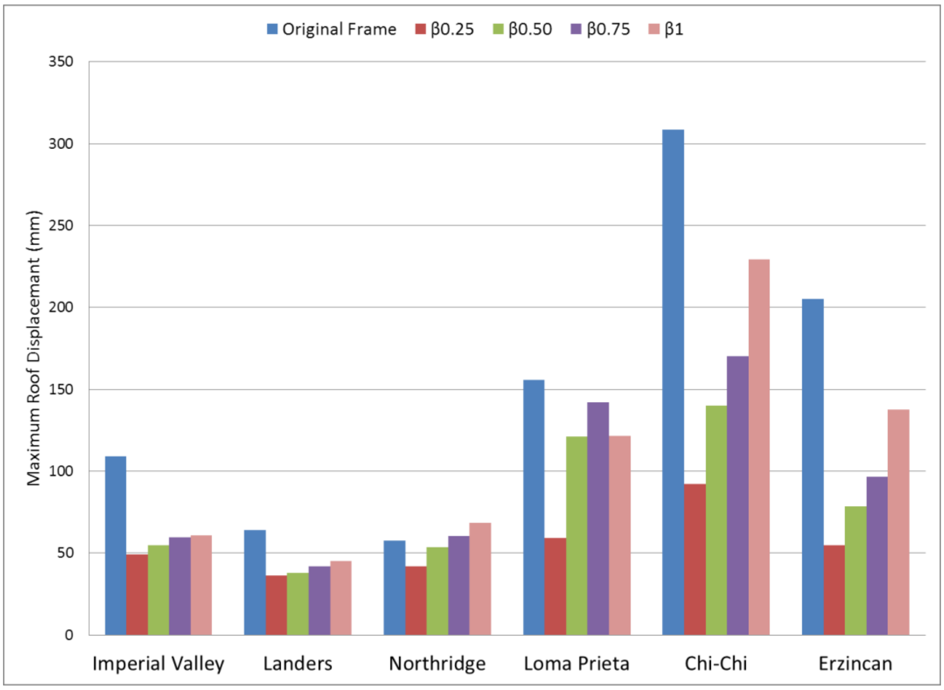
Figure 3.4 Elastic spectral accelerations of the ground motions scaled for seismic hazard of a) 10% probability of exceedance in 50 years and b) 50% probability of exceedance in 50 years

CHAPTER 4

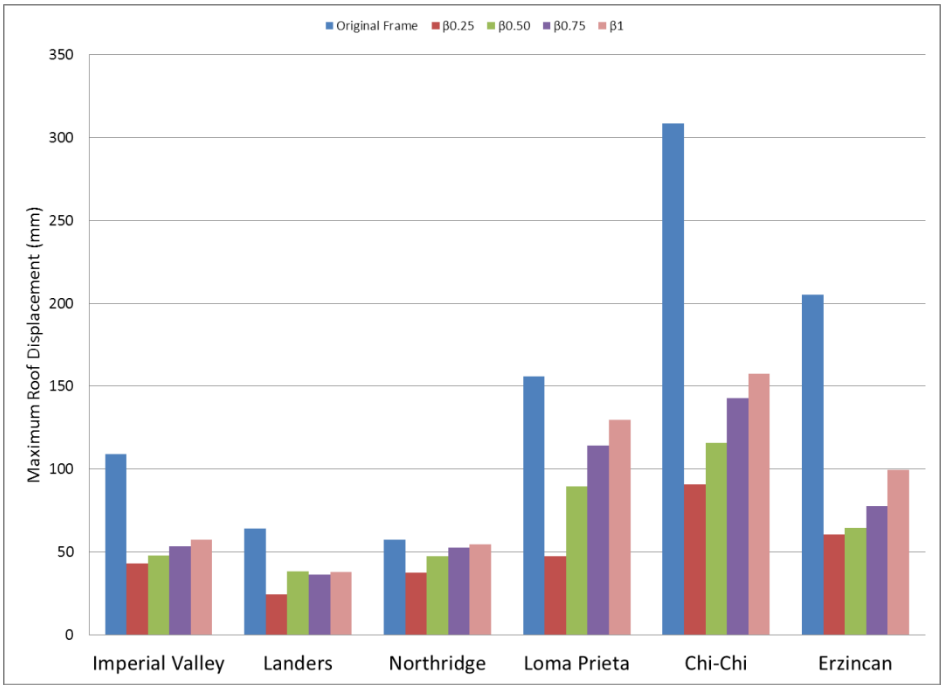
RESULTS AND DISCUSSION

As one of the global seismic performance parameter, maximum roof displacement demands for the original and retrofitted frames were obtained and the variation of maximum roof displacement demand of the two original and buckling restrained braced frames are demonstrated in Figures 4.1 and 4.2. As seen in Figures 4.1 and 4.2, for all BRBFs, the maximum roof displacement demand obtained was less than that of the original frame under all earthquake ground motions.

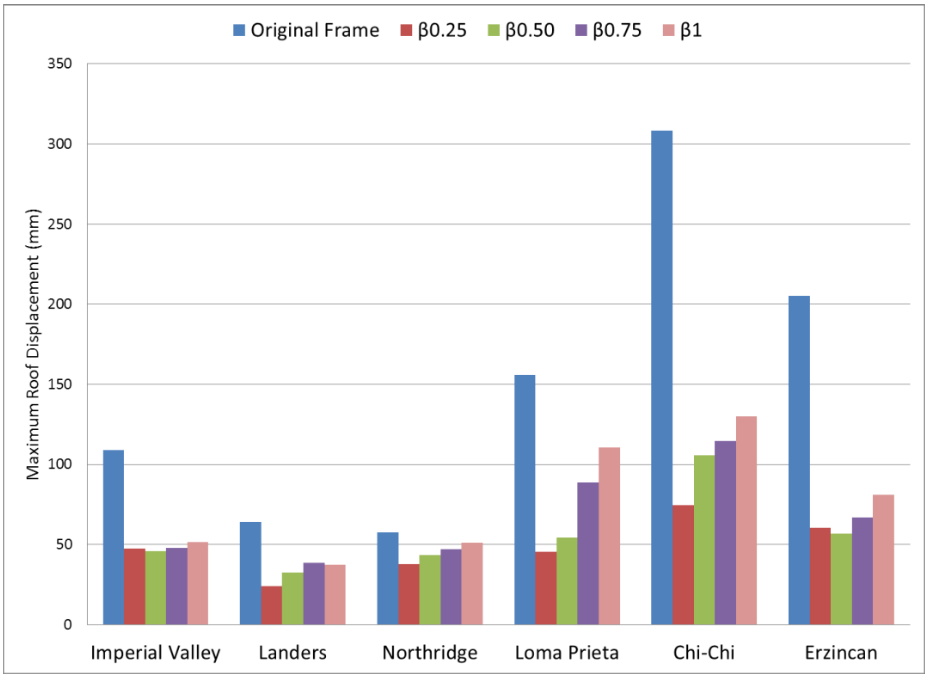
The conducted comparative analyses illustrated that the variation of the α and β values had significant effect on the maximum roof displacement demand such as the maximum roof displacement of the BRBFs decreased with the increase in α value, generally increased with the increase in β values. For example in 6 storey one, under Chi-Chi earthquake, the maximum roof displacement of the original frame was 3.4, 2.7, 2.2, and 2.0 times the maximum roof displacement demand of the BRBFs with α=0.50 and β=0.25, 0.50, 0.75 and 1.0, respectively. The ratio of the maximum roof displacement demand of the original frame to that of BRBFs with a constant β=0.50 and varying α values of 0.25, 0.75 and 1.0 became 2.2, 2.7 and 2.9, respectively. However, when subjected to earthquakes which have 50% probability of exceedance, these ratios reduced such as under Imperial-Valley earthquake, the maximum roof displacement of the original frame was 2.5, 2.3, 2.0 and 1.9 times the maximum roof displacement demand of the BRBF050025, BRBF050050, BRBF050075, and BRBF050100, respectively.


Original Frame
β0.25
β0.50
β0.75
β1
Maximum Roof Displacemant (mm)
350
300
250
200
150
100
50
0
Imperial Valley
Landers
Northridge
Loma Prieta
Chi-Chi
Erzincan

a)

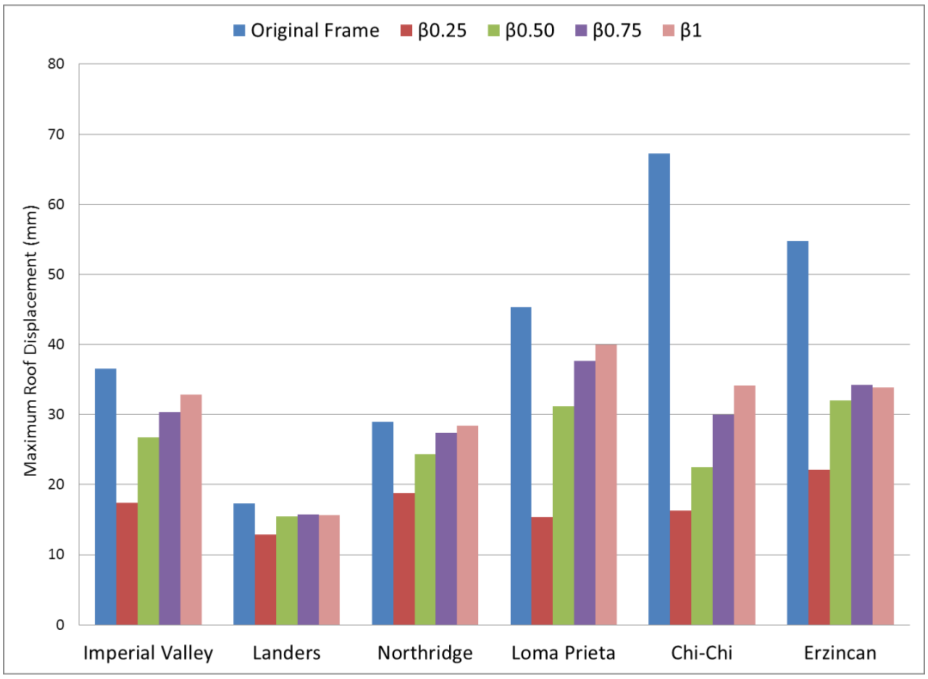

Original Frame
β0.25
β0.50
β0.75
β1
Maximum Roof Displacement (mm)
350
300
250
200
150
100
50
0
Imperial Valley
Landers
Northridge
Loma Prieta
Chi-Chi
Erzincan

b)

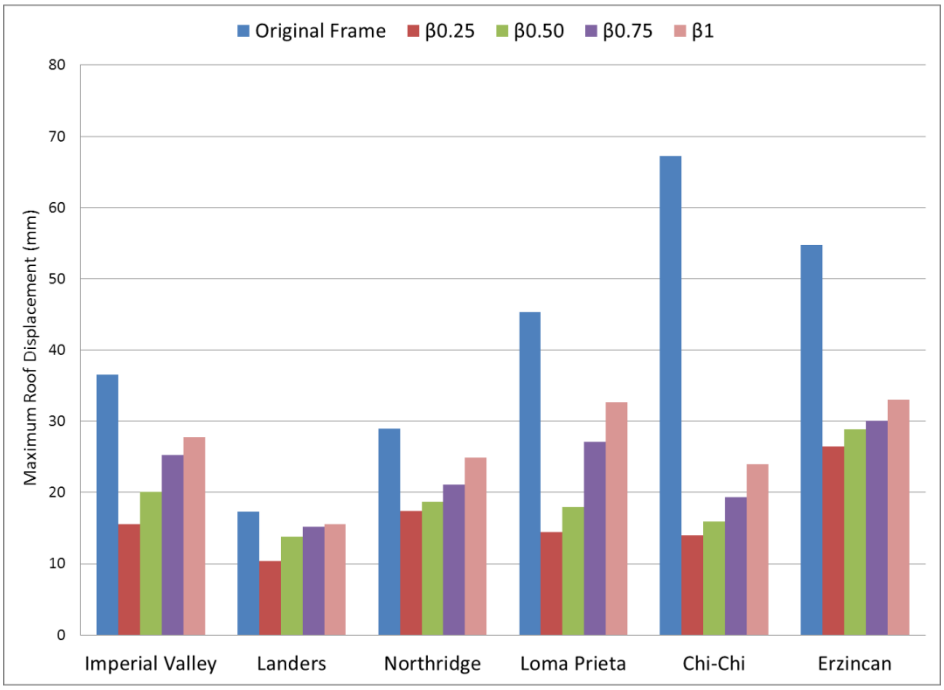


c)

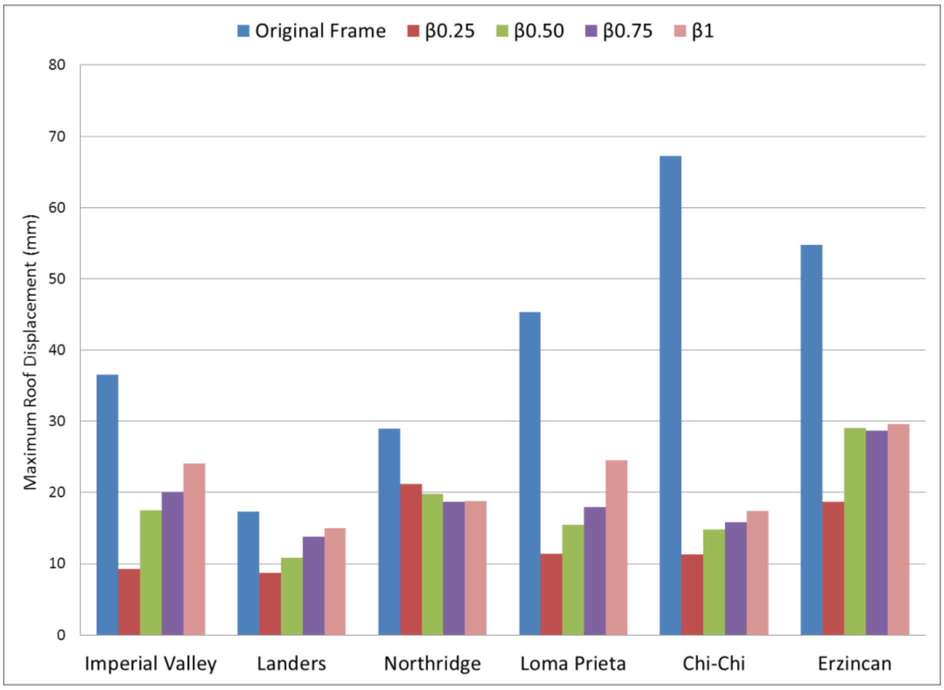
Figure 4.1 Variation of maximum roof displacement demand for the original and BRBFs for 6 storey with α value of a) 0.25, b) 0.50, and c) 0.75



a)



b)

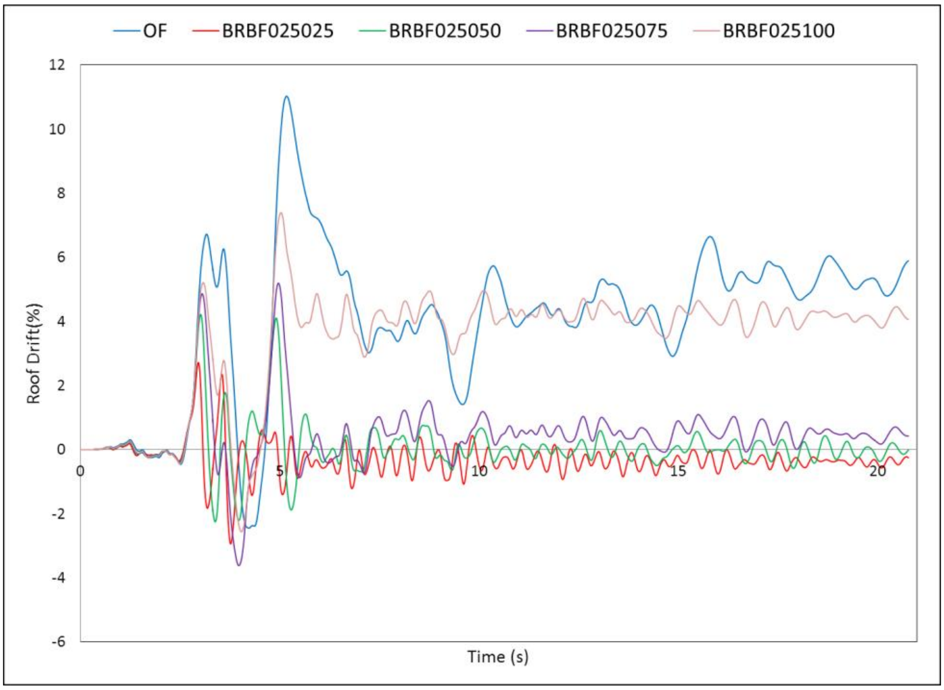


c)

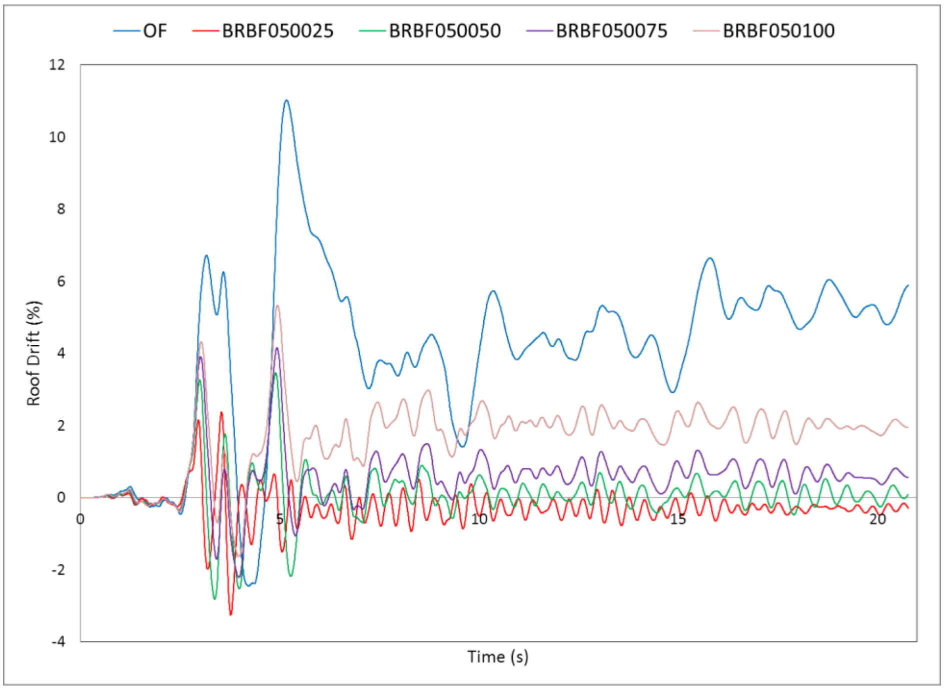
Figure 4.2 Variation of maximum roof displacement demand for the original and BRBFs for 3 storey with α value of a) 0.25, b) 0.50, and c) 0.75

The time history response of the roof drifts of the two original and BRBFs subjected to Erzincan and Landers earthquakes are presented in Figures 4.3, 4.4, 4.5 and 4.6, respectively. When the response histories obtained under Erzincan and Landers earthquakes were compared, it was observed that the buckling restrained braces were more effective in reducing roof drifts when subjected to Erzincan earthquake acceleration which has 10% probability of exceedance. Similarly, significant residual drifts were examined in the original frame and in some of the buckling restrained braced frames when the earthquake accelerations with 10% probability of exceedance in 50 years were considered.

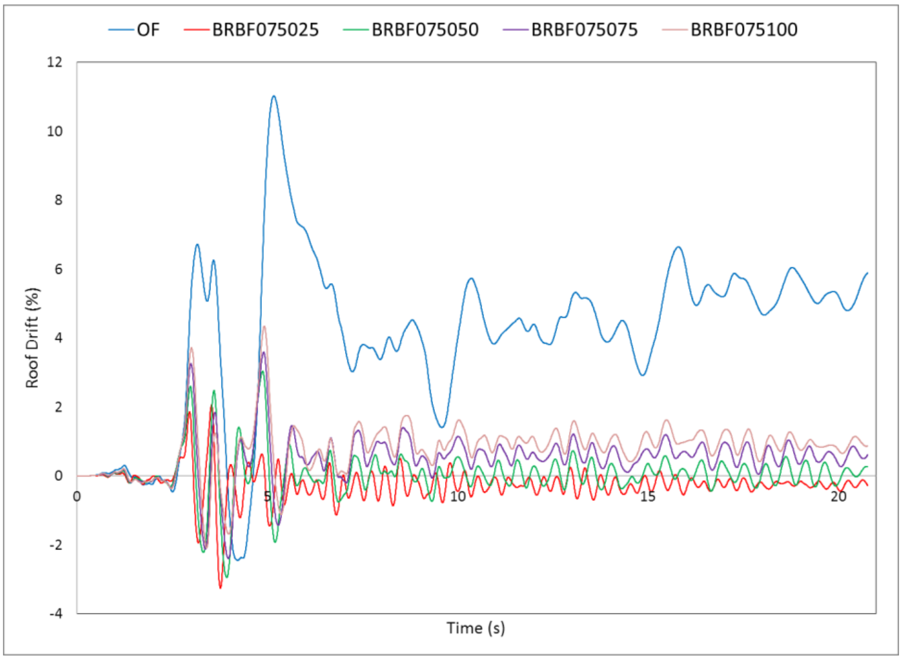
Furthermore, with the increase in α value, more reduction in the roof drift history of the BRBFs was observed and it was much more pronounced especially for the case of β=1.0. For instance in 6-storey building, under Erzincan earthquake, the reduction in the maximum roof displacement demand of the BRBF075100 with respect to the original frame was about 60%, whereas 33% to 51% reduction was obtained for BRBF025100 and BRBF050100, respectively. In addition to this, it was noticed that for a constant α value, the decrease in β value resulted in a reduction of the roof drift response history of the BRBFs. For BRBF025025, BRBF025050, BRBF025075, and BRBF025100, approximately 73%, 62%, 53%, and 33% reduction in the maximum roof drift with respect to the original frame was obtained.



a)

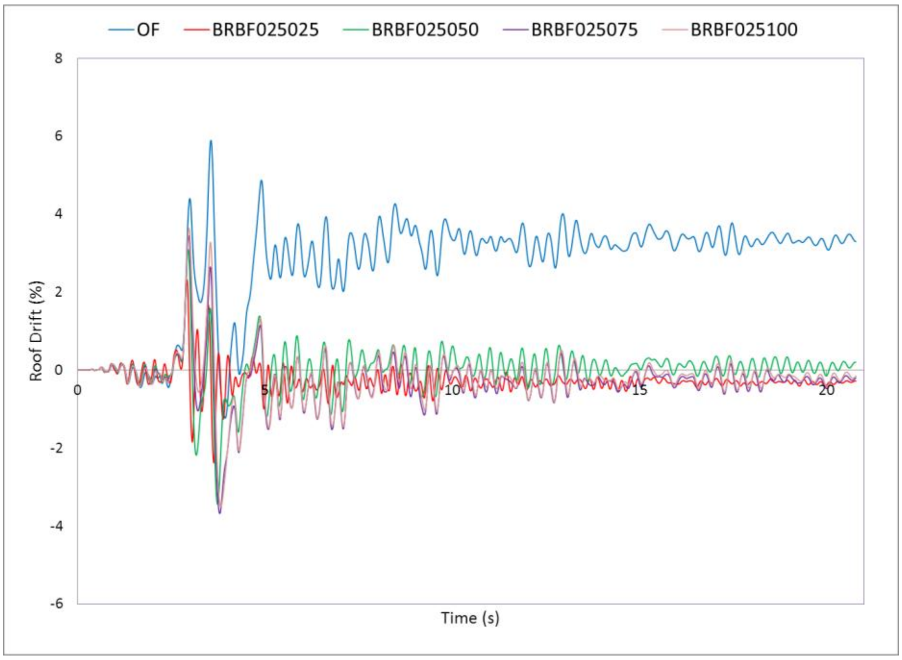


b)

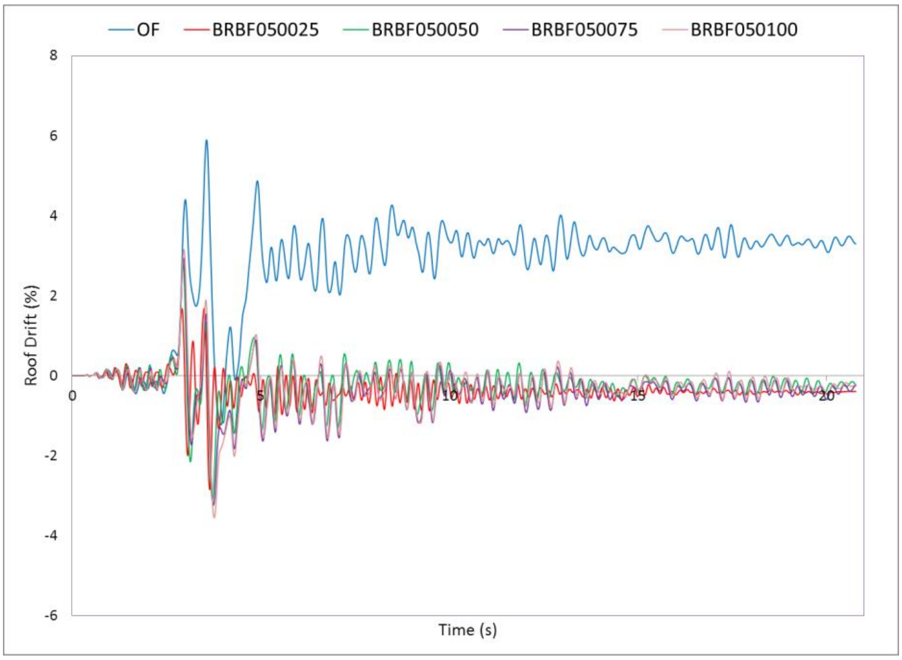


c)

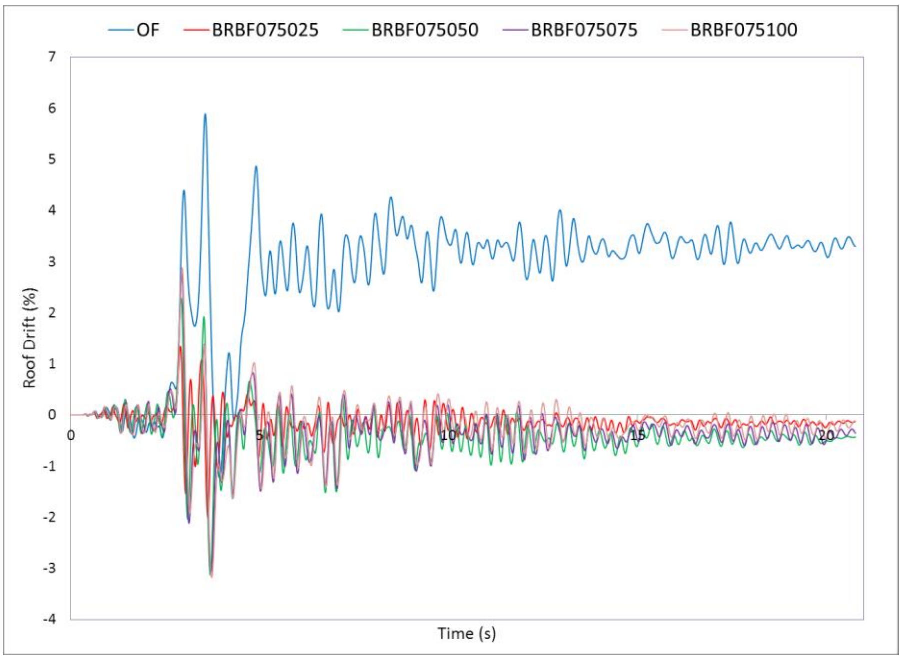
Figure 4.3 Roof drifts obtained for the original and BRBFs for 6 storey with α value of a) 0.25, b) 0.50, and c) 0.75 under Erzincan earthquake


OF
BRBF025025
BRBF025050
BRBF025075
BRBF025100
Roof Drift (%)
Time (s)
8
6
4
2
0
-2
-4
-6
0
5
10
15
20

a)

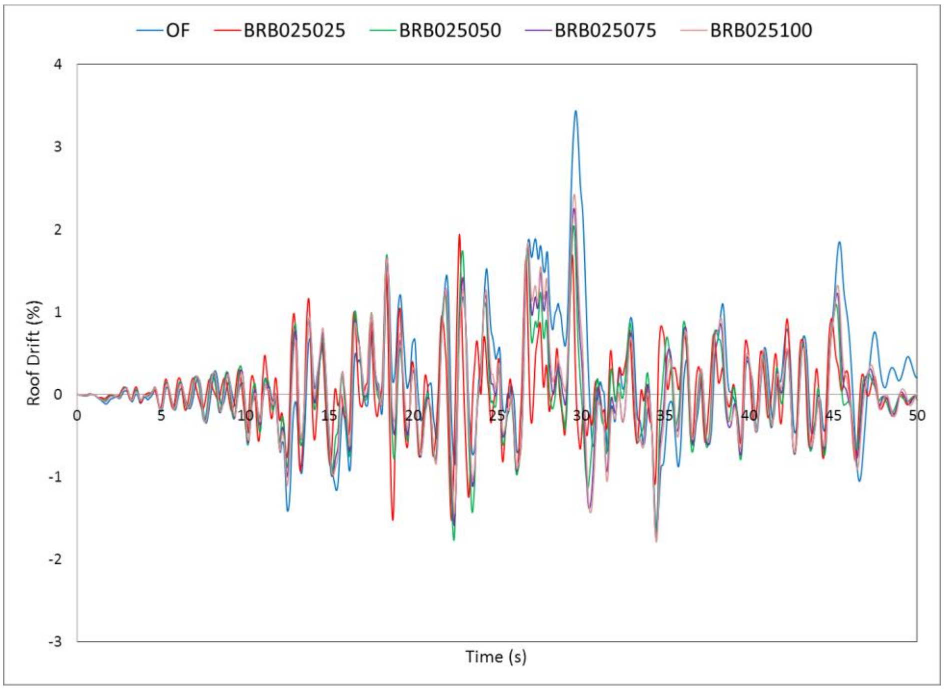

OF
BRBF050025
BRBF050050
BRBF050075
BRBF050100
Roof Drift (%)
Time (s)
8
6
4
2
0
-2
-4
-6
0
5
10
15
20

b)

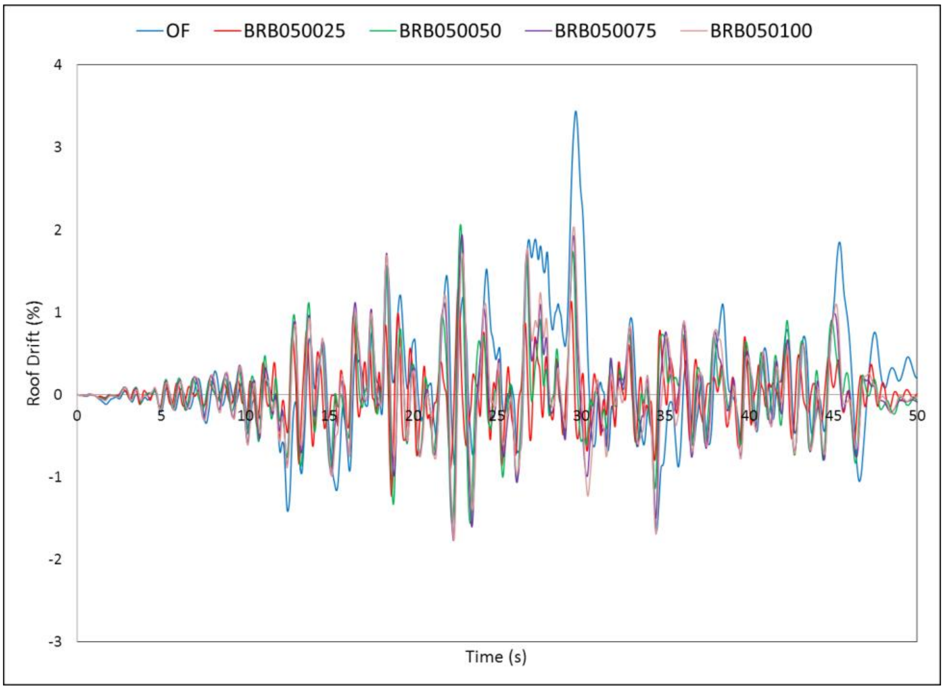


c)

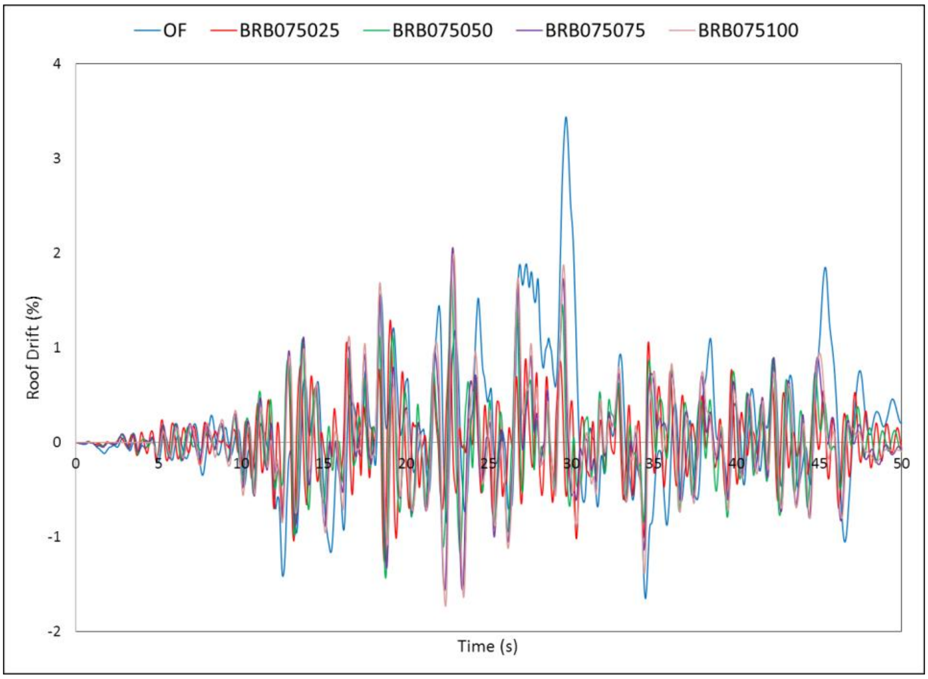
Figure 4.4 Roof drifts obtained for the original and BRBFs for 3 storey with α value of a) 0.25, b) 0.50, and c) 0.75 under Erzincan earthquake



a)

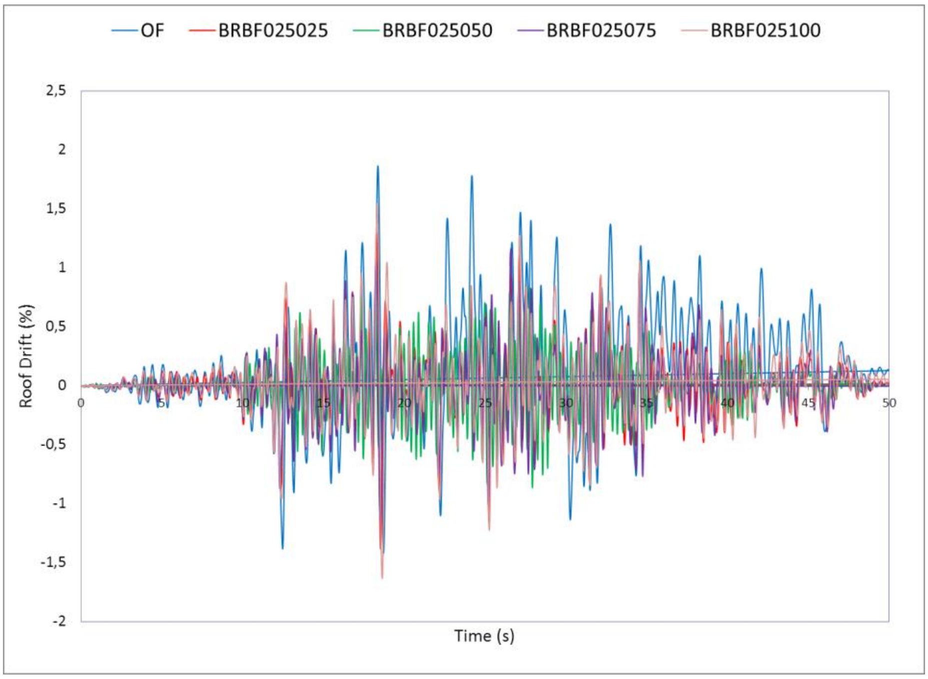


b)



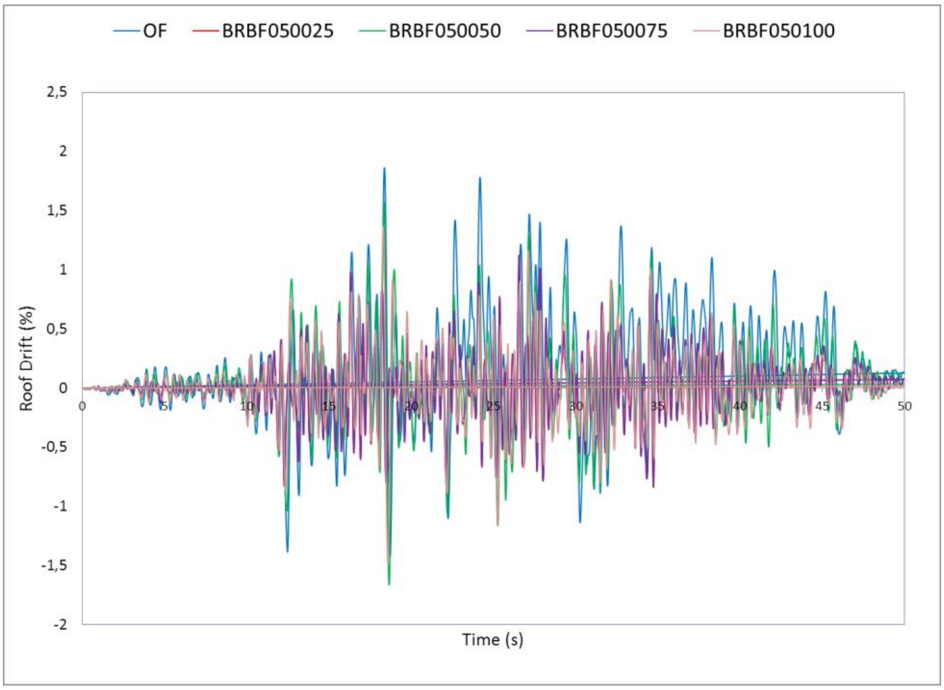
c)

Figure 4.5 Roof drifts obtained for the original and BRBFs for 6 storey with α value of a) 0.25, b) 0.50, and c) 0.75 under Landers earthquake



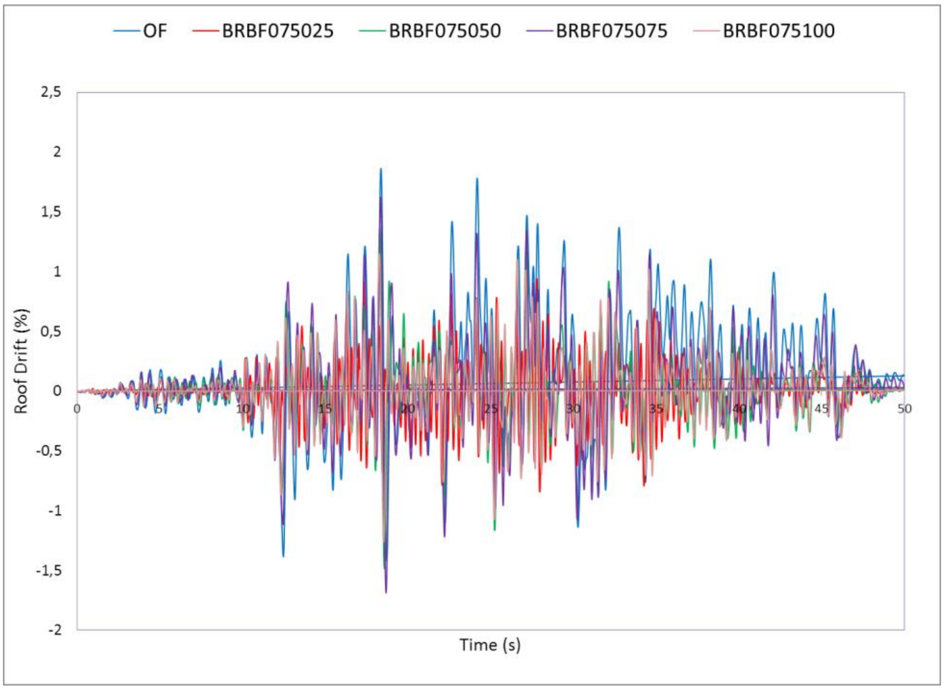
OF
BRBF025025
BRBF025050
BRBF025075
BRBF025100
Roof Drift (%)
Time (s)

a)



OF
BRBF050025
BRBF050050
BRBF050075
BRBF050100
Roof Drift (%)
Time (s)

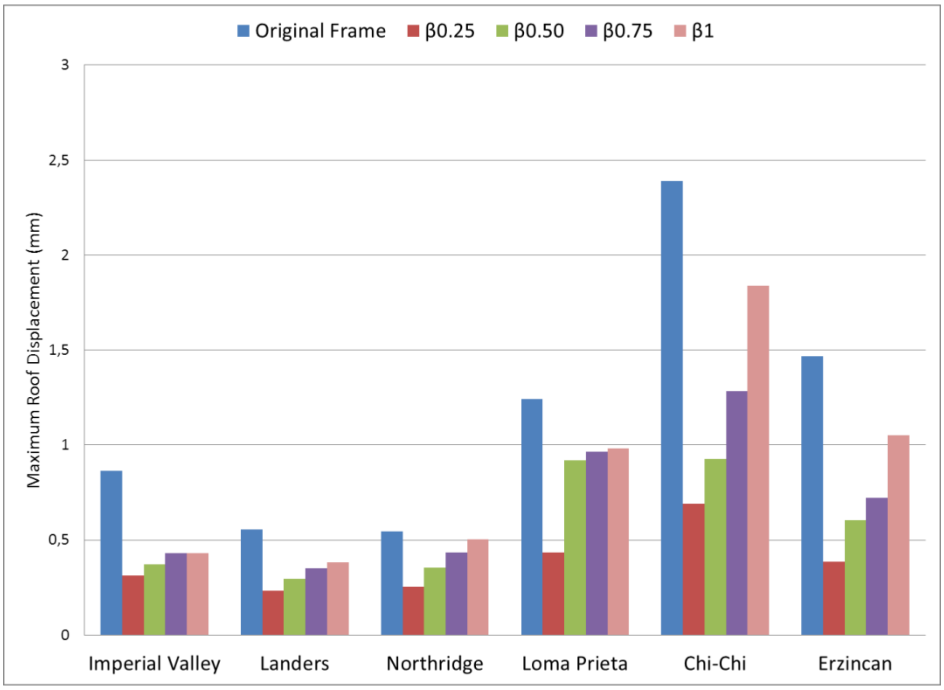
b)



c)

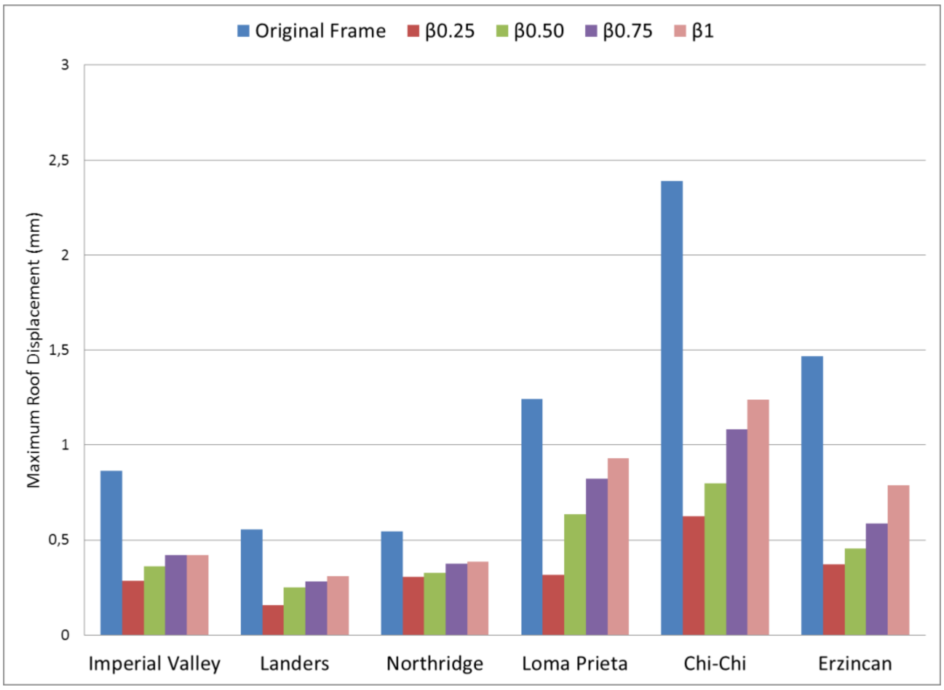
Figure 4.6 Roof drifts obtained for the original and BRBFs for 3 storey with α value of a) 0.25, b) 0.50, and c) 0.75 under Landers earthquake

Maximum inter-storey drift ratio obtained at each storey level for the original and BRBFs with α value of 0.25, 0.50, and 0.75 are given in Figures 4.7 and 4.8. As seen from Figures 4.7 and 4.8, the maximum inter-storey drift ratio considerably decreased with the increase in the α value. The maximum inter-storey drift ratio decreased with the use of buckling restrained braces for all α and β values, and this reduction became more evident for the earthquakes with 10% probability of exceedance in 50 years period. Therefore, the results verified the beneficial effects of buckling restrained braces in reducing the seismic deformation demand.



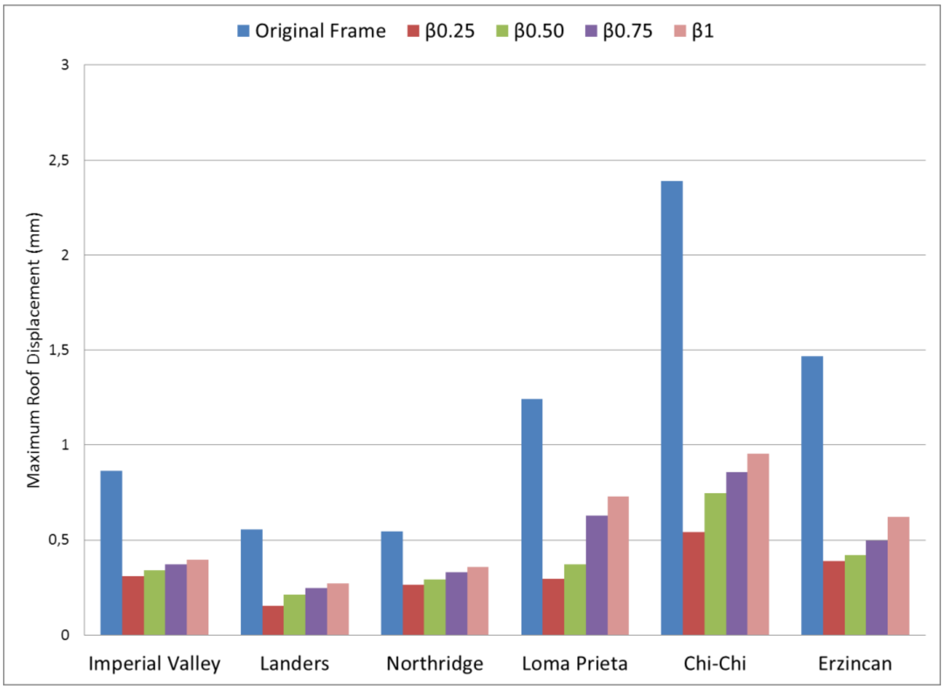
Original Frame
β0.25
β0.50
β0.75
β1
Maximum Roof Displacement (mm)
3
2,5
2
1,5
1
0,5
0
Imperial Valley
Landers
Northridge
Loma Prieta
Chi-Chi
Erzincan

a)



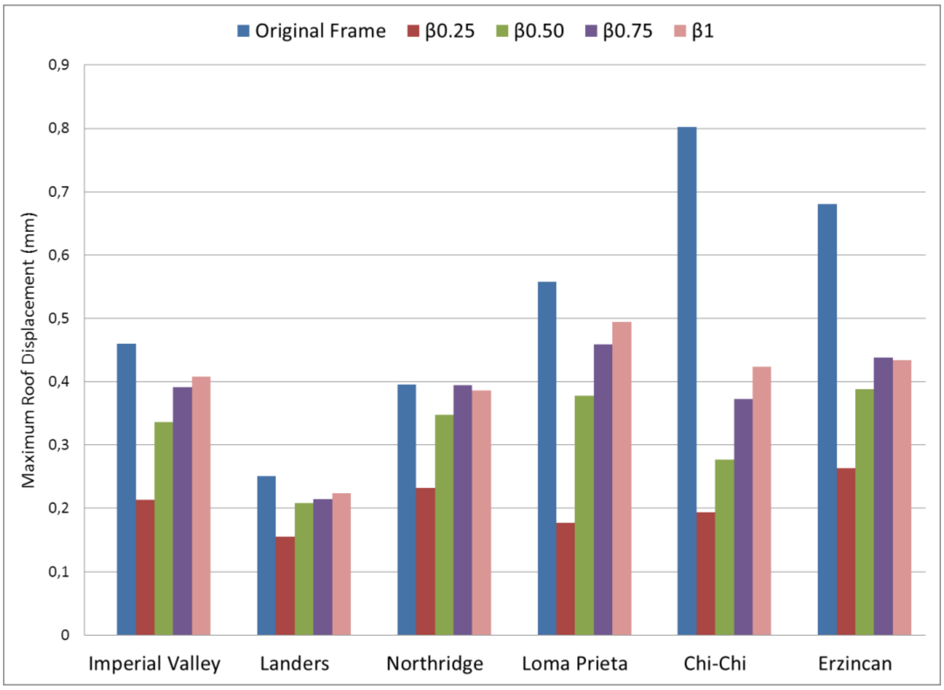
Original Frame
β0.25
β0.50
β0.75
β1
Maximum Roof Displacement (mm)
3
2,5
2
1,5
1
0,5
0
Imperial Valley
Landers
Northridge
Loma Prieta
Chi-Chi
Erzincan

b)

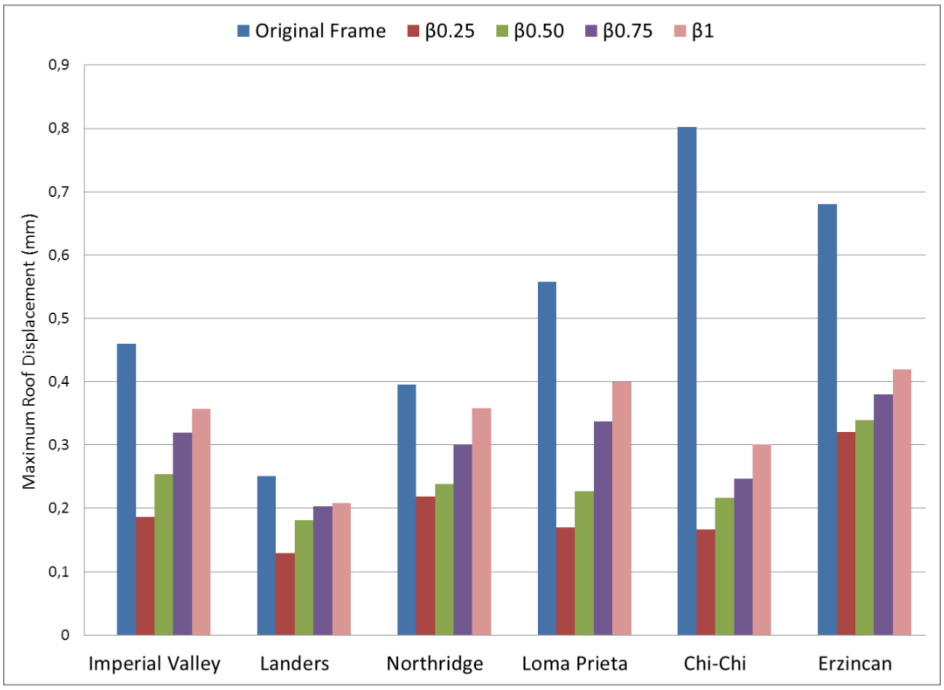


c)

Figure 4.7 Variation of maximum inter-storey drift ratio for the original and BRBFs for 6 storey with α value of a) 0.25, b) 0.50, and c) 0.75



a)



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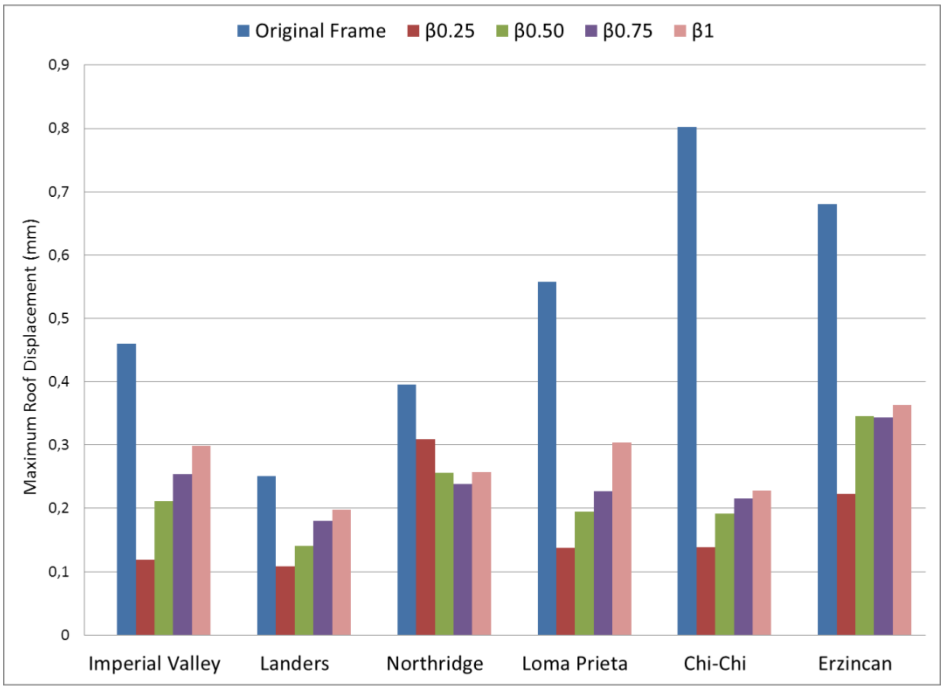
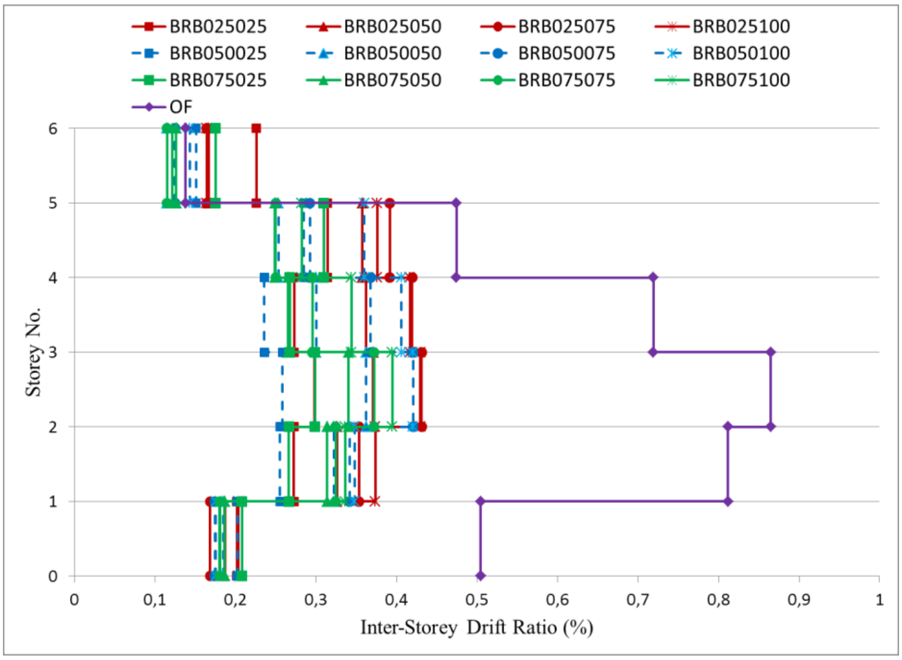


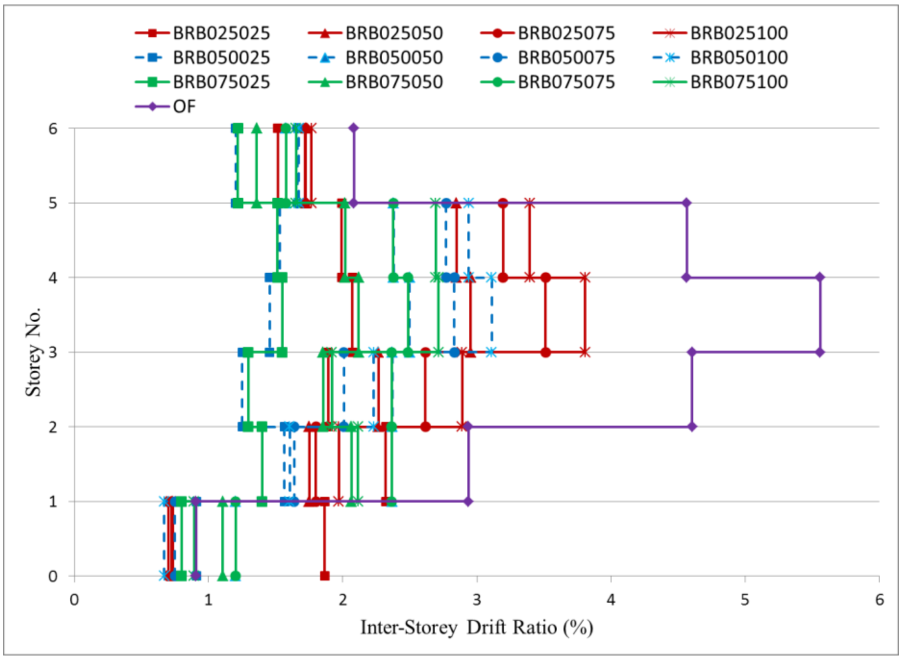
c)

Figure 4.8 Variation of maximum inter-storey drift ratio for the original and BRBFs for 3 storey with α value of a) 0.25, b) 0.50, and c) 0.75

The results of inter-storey drift ratios presented in Figures 4.9 and 4.10 illustrated that buckling restrained braces were more effective for the earthquakes with 10% probability of exceedance, and thus they became more efficient in the inelastic range. Moreover, with the use of buckling restrained braces, generally more uniform distribution of inter-storey drifts along the height of the structure was attained, but the maximum inter-storey drift distribution through the height of the structure was changing, depending mainly upon the α and β values and also the ground acceleration used. The design values of α and β parameters of the buckling restrained braces significantly influenced the level of seismic response of the BRBFs. Moreover, it was observed that for a constant α value, the maximum inter-storey drift ratio at each storey level increased with the increase in β value, and for a constant β value, the maximum inter-storey drift ratio decreased with the increase in α value.



a)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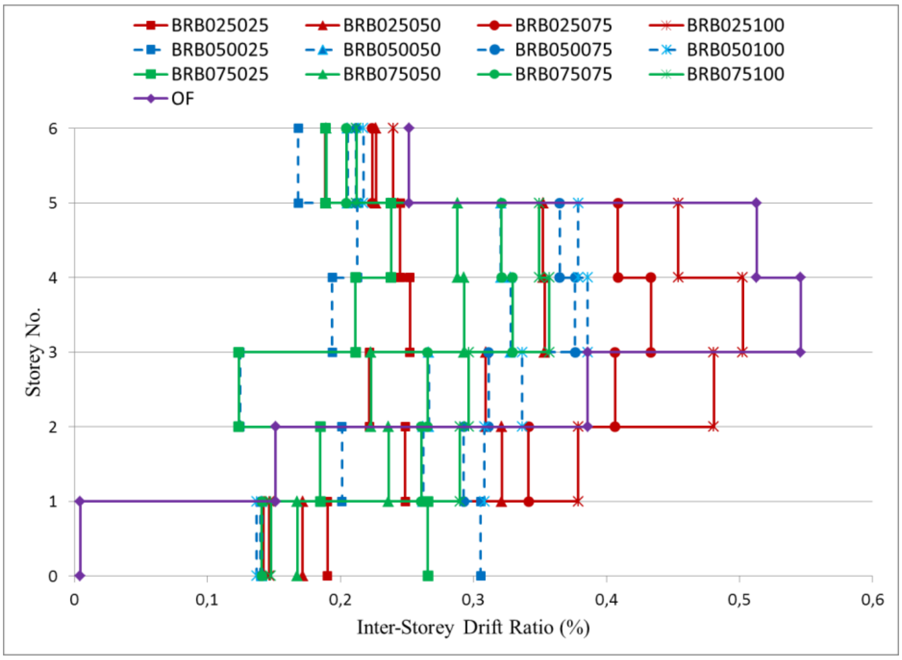
BRB075075
BRB075100
OF
Storey No.
Inter-Storey Drift Ratio (%)

b)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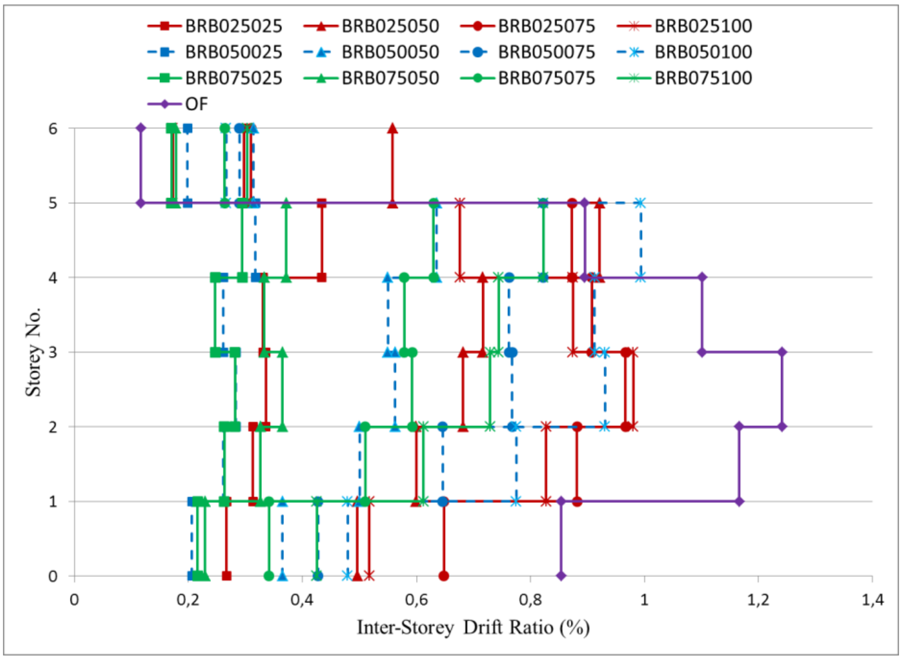
BRB075100
OF
Storey No.
Inter-Storey Drift Ratio (%)

c)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BRB050100
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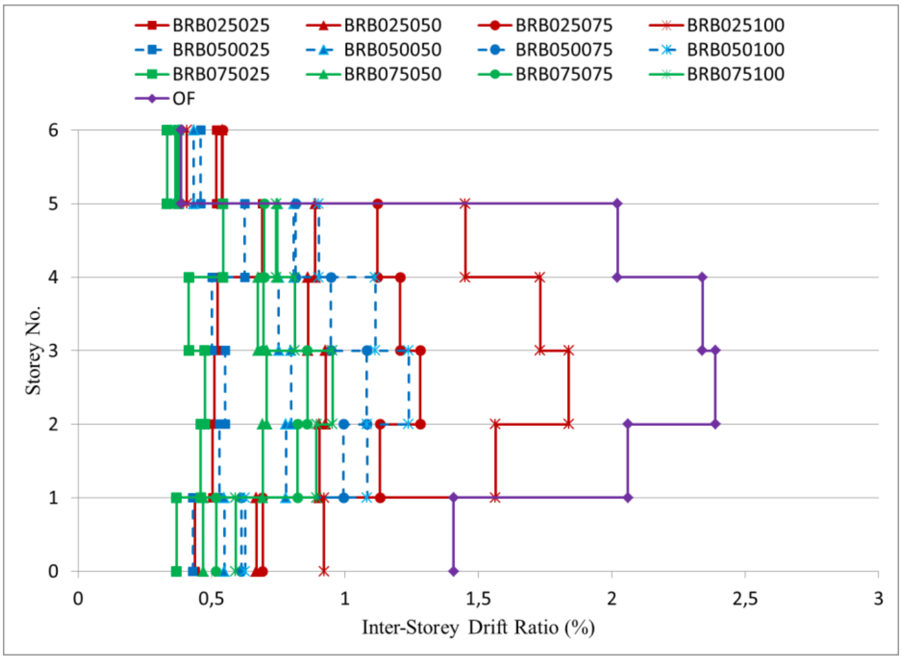
BRB075050
BRB075075
BRB075100
OF
Storey No.
Inter-Storey Drift Ratio (%)
0
0,2
0,4
0,6
0,8
1
1,2
1,4
1
2
3
4
5
6

d)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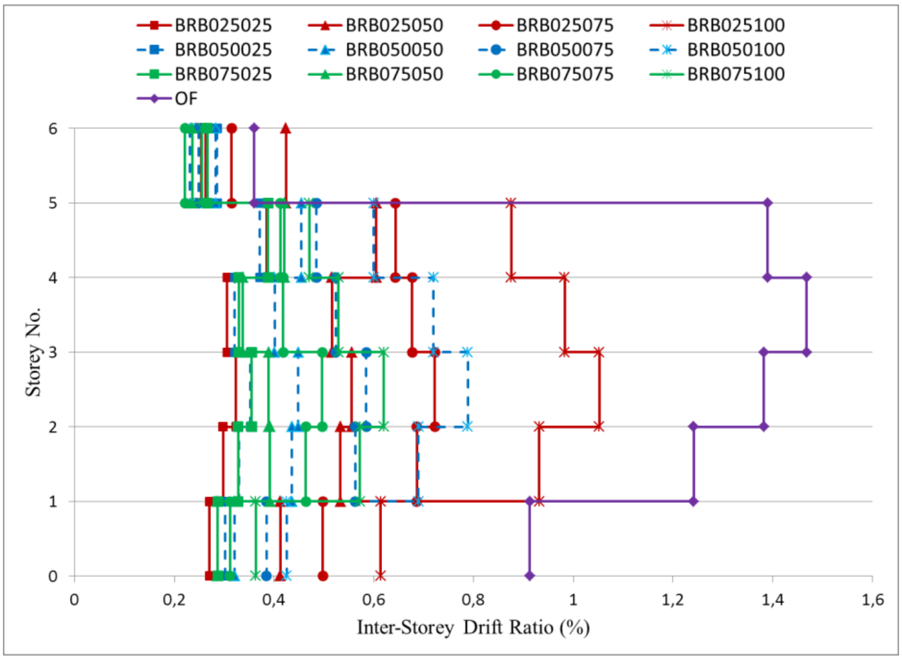
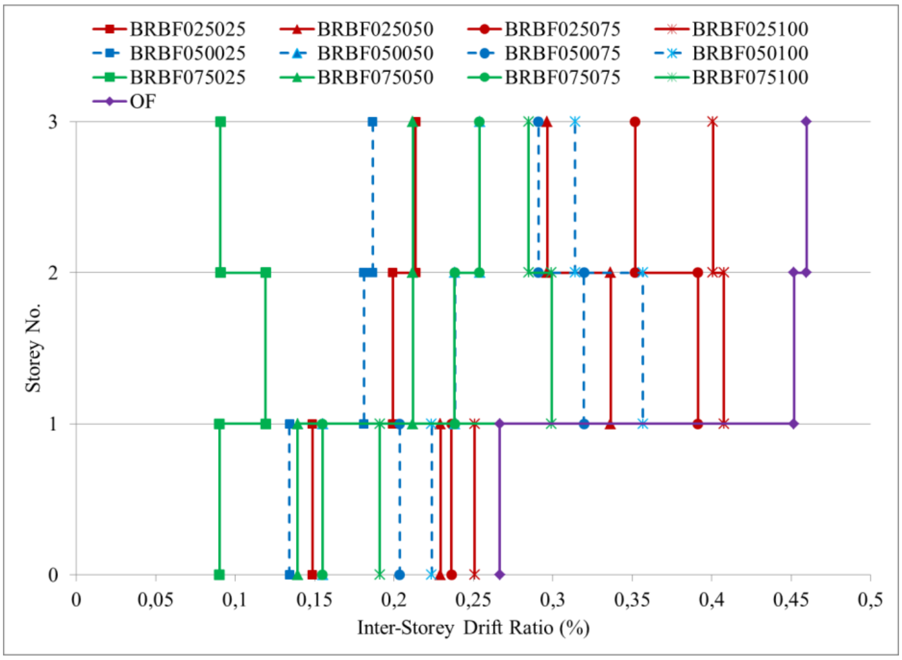
BRB075100
OF
Storey No.
Inter-Storey Drift Ratio (%)
0
0,5
1
1,5
2
2,5
3
1
2
3
4
5
6

e)

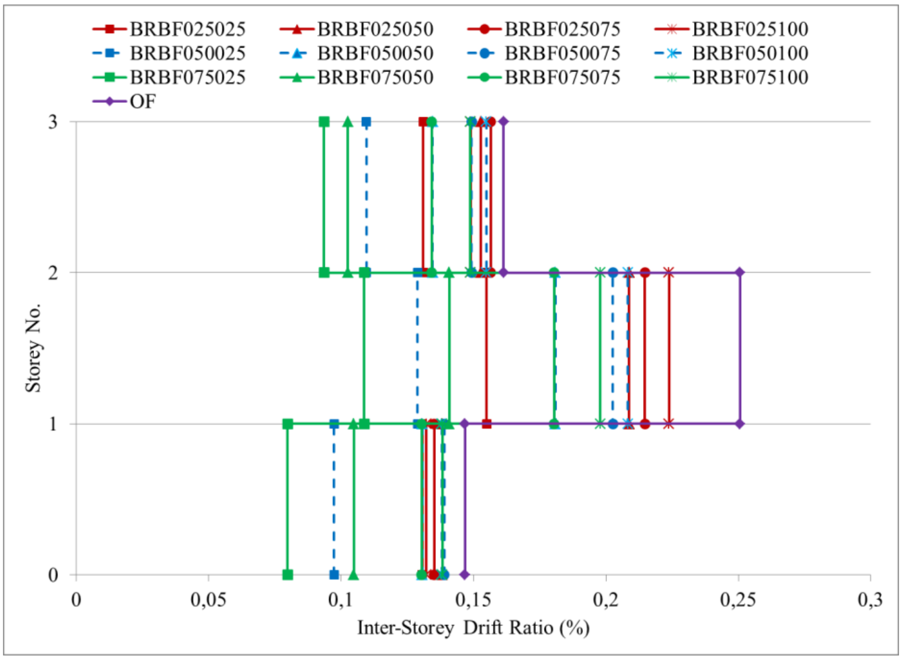


f)

Figure 4.9 Maximum inter-storey drift ratios for the original frame and BRBFs for 6 storey subjected to a) Imperial Valley, b) Landers, c) Northridge, d) Loma Prieta, e) Chi-Chi, and f) Erzincan earthquake accelerations



a)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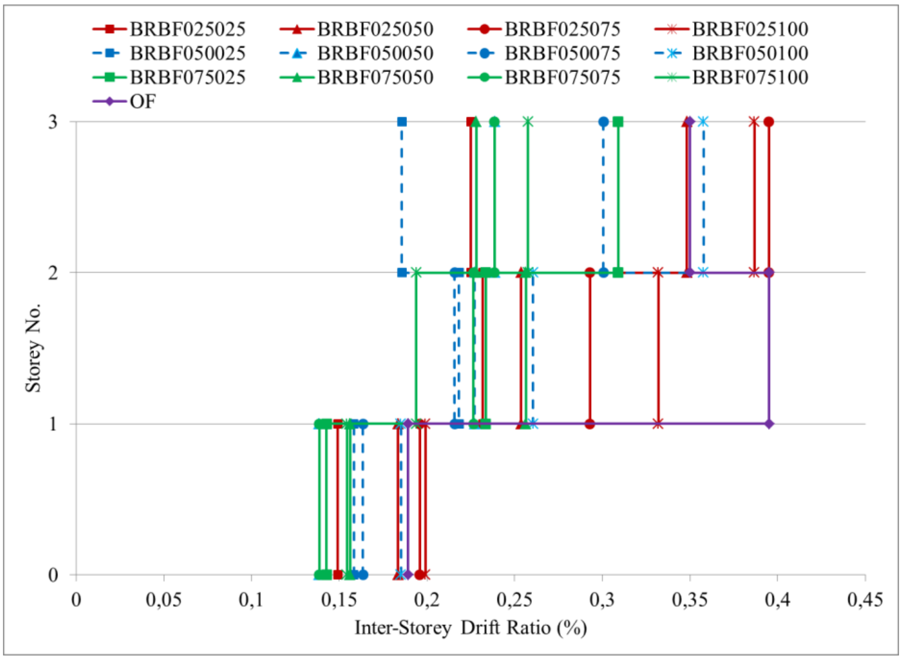
BRBF050100
BRBF075025
BRBF075050
BRBF075075
BRBF075100
OF
Storey No.
Inter-Storey Drift Ratio (%)
0
1
2
3
0
0,05
0,1
0,15
0,2
0,25
0,3

b)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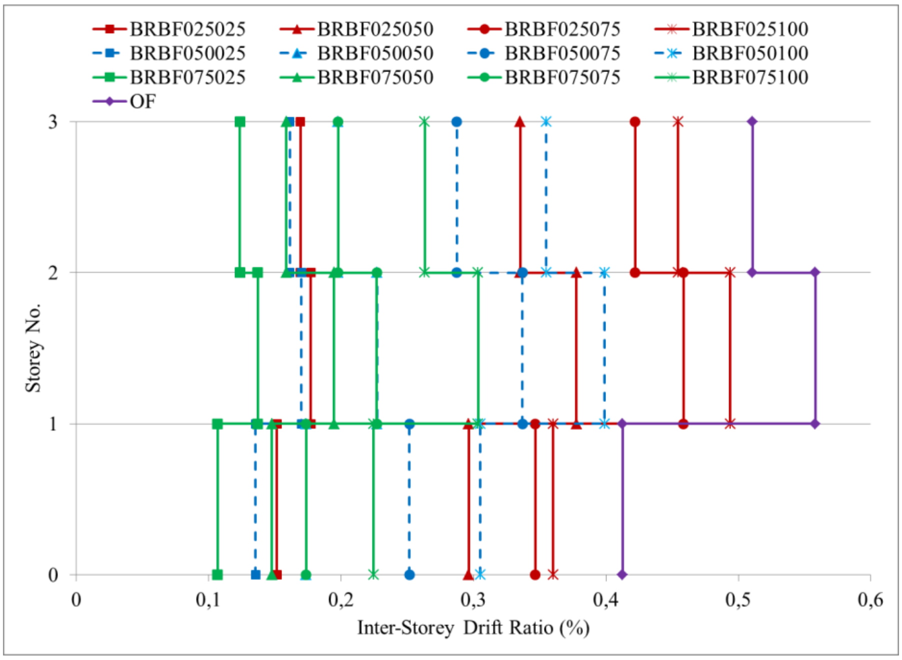
BRBF050100
BRBF075025
BRBF075050
BRBF075075
BRBF075100
OF
Storey No.
Inter-Storey Drift Ratio (%)
0
1
2
3
0
0,05
0,1
0,15
0,2
0,25
0,3
0,35
0,4
0,45

c)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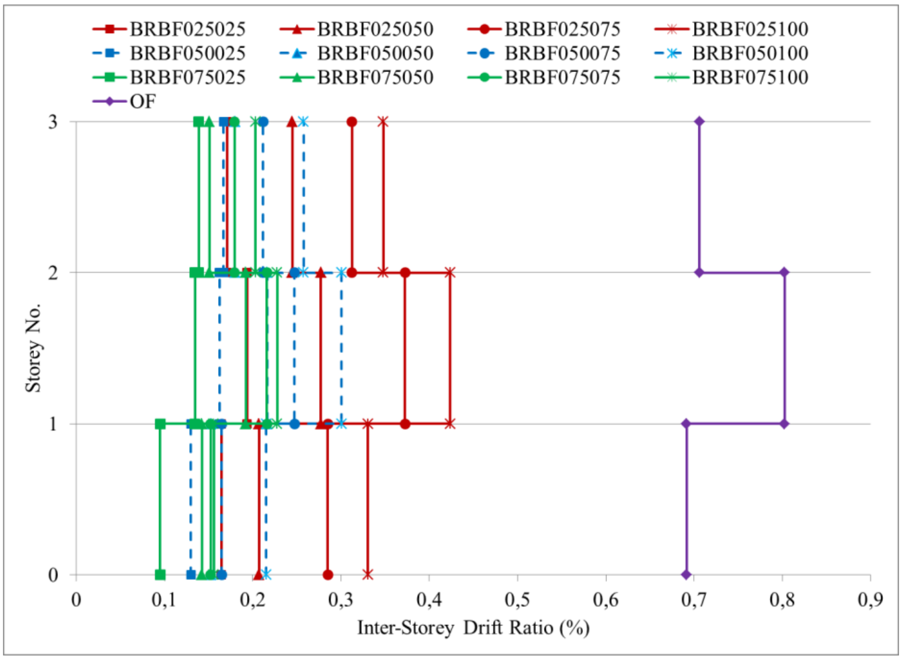
BRBF050025
BRBF050050
BRBF050075
BRBF050100
BRBF075025
BRBF075050
BRBF075075
BRBF075100
OF
Storey No.
Inter-Storey Drift Ratio (%)
0
0,1
0,2
0,3
0,4
0,5
0,6
1
2
3

d)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BRBF050025
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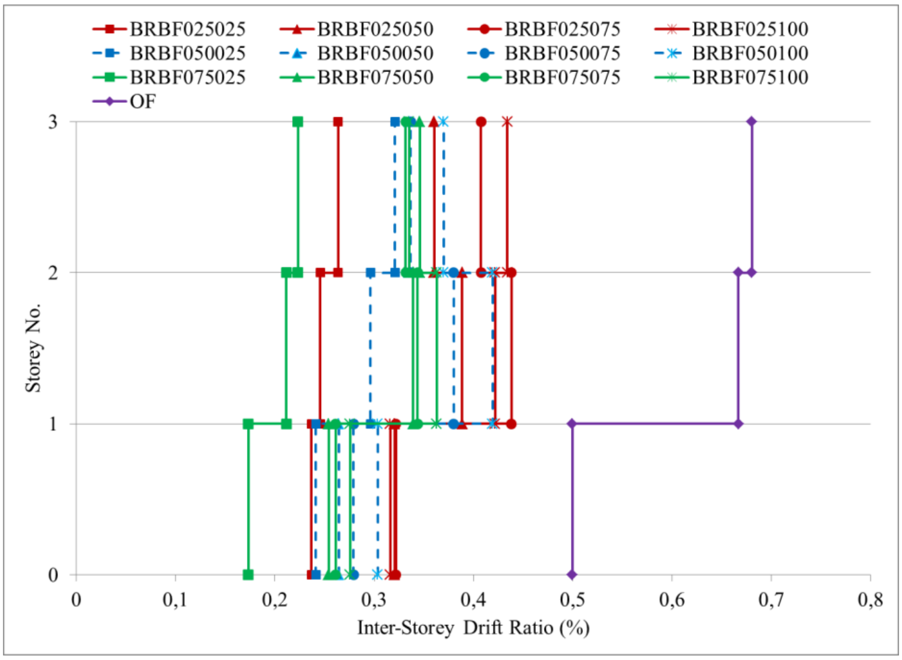
BRBF050075
BRBF050100
BRBF075025
BRBF075050
BRBF075075
BRBF075100
OF
Storey No.
Inter-Storey Drift Ratio (%)
0
0,1
0,2
0,3
0,4
0,5
0,6
0,7
0,8
0,9
1
2
3

e)

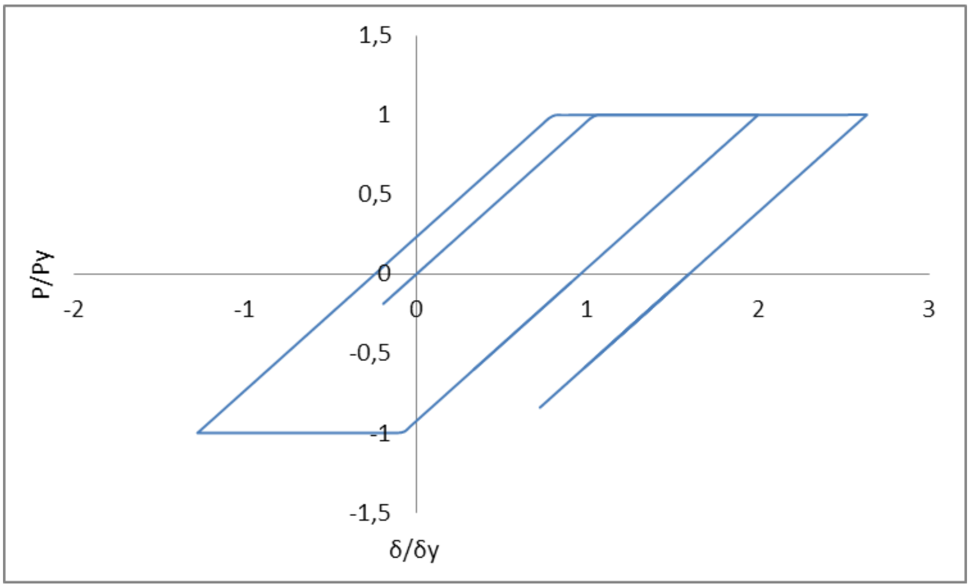


f)

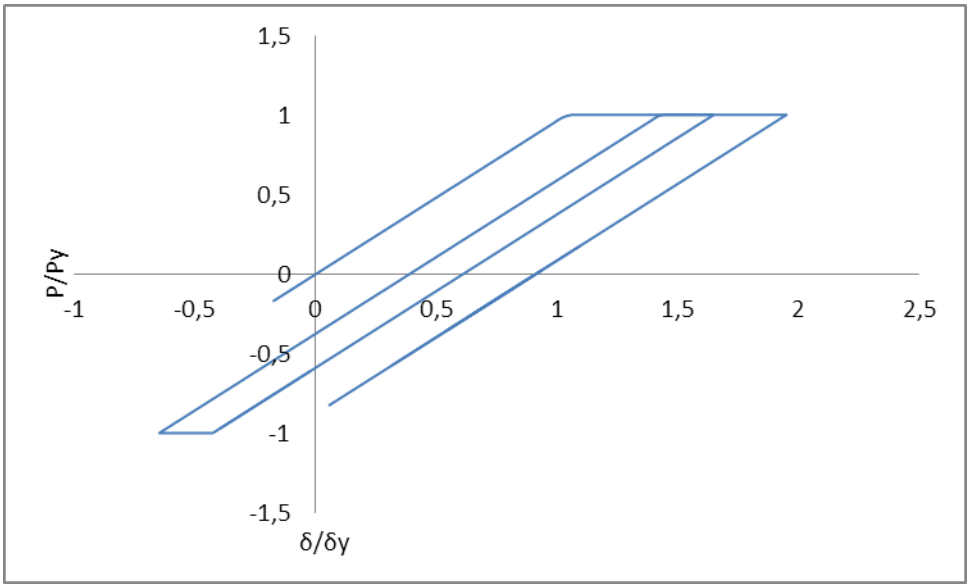
Figure 4.10 Maximum inter-storey drift ratios for the original frame and BRBFs for 3 storey subjected to a) Imperial Valley, b) Landers, c) Northridge, d) Loma Prieta, e) Chi-Chi, and f) Erzincan earthquake accelerations

It was reported that a properly designed buckling restrained braces could accommodate inelastic deformations without permitting undesirable modes of failure (Sabelli and López, 2004) and under strong ground accelerations; they could experience axial strains which are approximately 20 times their yield strain (Usami et al., 2005).

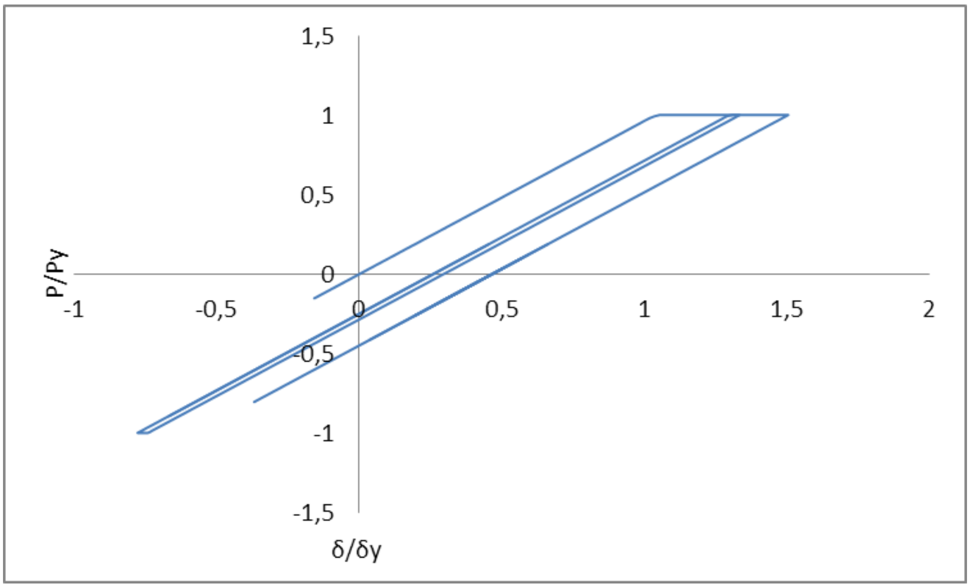
In this study, conservatively, 12 times the yield strain was considered as the strain capacity of the buckling restrained braces and the maximum axial strain demand of the buckling restrained braces were checked. As an example, the normalized load deformation behavior of buckling restrained braces obtained under Erzincan earthquake acceleration are given in Figures 4.11 and 4.12. From this cyclic behavior of buckling restrained braces, it was observed that with the decrease in α and β values, axial strain demand of the buckling restrained braces had a tendency to increase.


1,5
1
0,5
0
-0,5
-1
-1,5
-2
-1
0
1
2
3
P/Py
δ/δy

a)

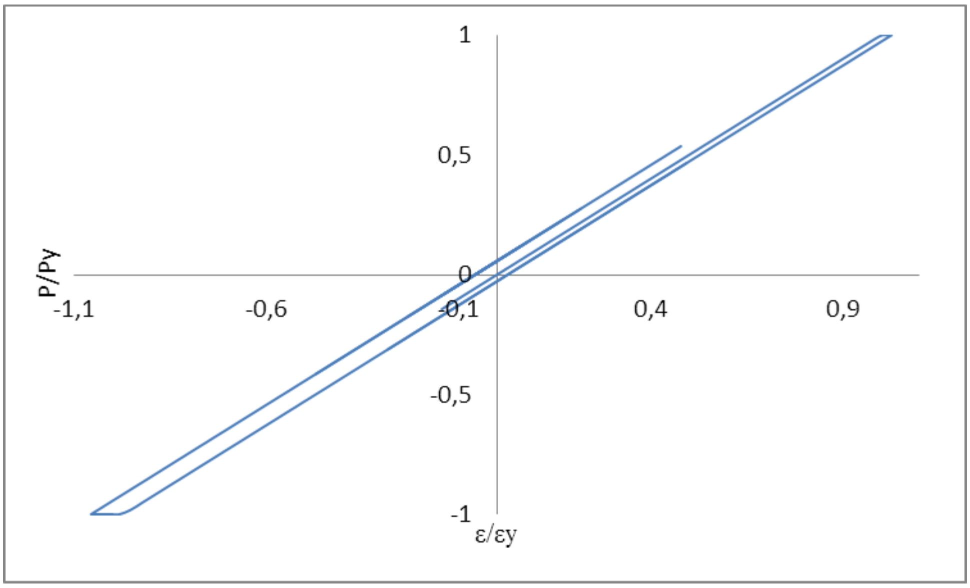

1,5
1
0,5
0
-0,5
-1
-1,5
-1
-0,5
0
0,5
1
1,5
2
2,5
P/Py
δ/δy

b)

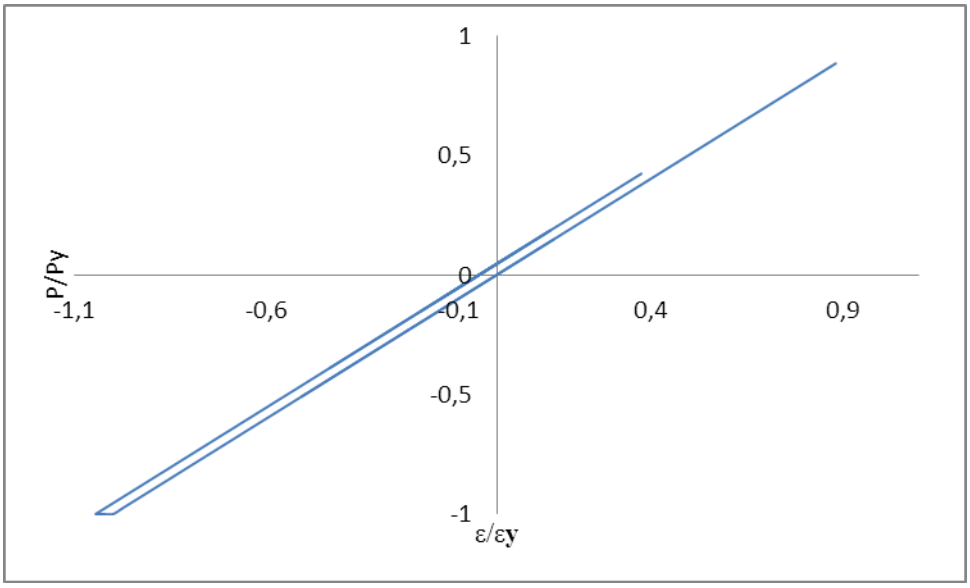


c)

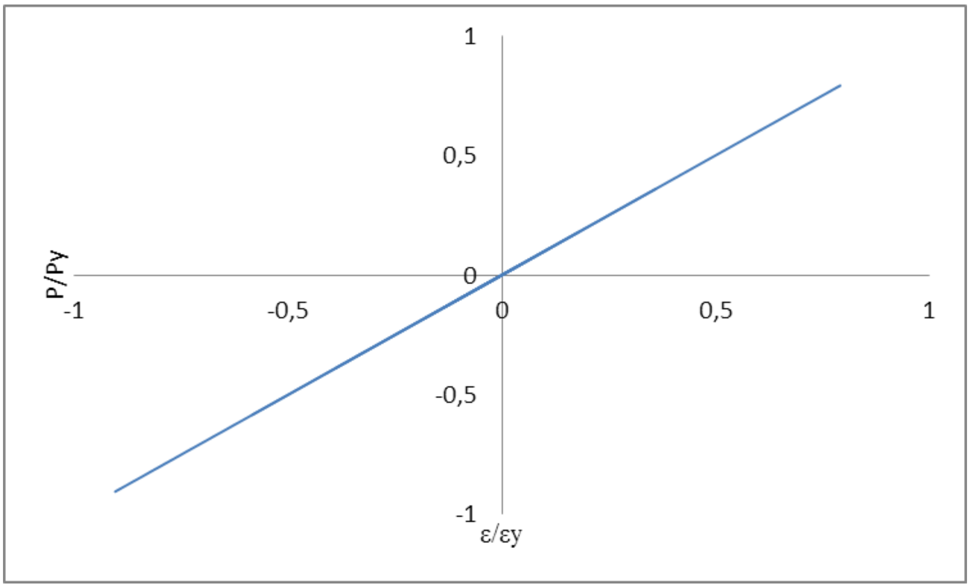
Figure 4.11 Load deformation behavior of BRBFs with β=1.0 and a) α=0.25, b) α=0.50, and c) α=0.75 for 6 storey under Erzincan earthquake



a)



b)



c)

Figure 4.12 Load deformation behavior of BRBFs with β=1.0 and a) α=0.25, b) α=0.50, and c) α=0.75 for 3 storey under Erzincan earthquake

CHAPTER 5

CONCLUSIONS

In this analytical study, the seismic behavior of the existing reinforced concrete buildings and those retrofitted by buckling restrained braces were evaluated. In the seismic retrofitting, different design parameters of buckling restrained braces were taken into account. From the results of this study, the following conclusions can be drawn:

- It was observed that in the design of the buckling restrained braces with non-prismatic core sections, the selection of the α and β parameters affected considerably the seismic response of the structures such that the increase in the α value and decrease in the β value resulted in dramatic decrease in the seismic deformation of the buckling restrained braced frames.

- The change in the maximum deformation demand due to β value was small especially under the earthquakes with 10% probability of exceedance.

- Moreover, it was pointed out that for all buckling restrained braces, the maximum strain demand was less than the limit since with the inclusion of the buckling restrained braces, the frames became stiffer. Because of these reasons, any value of β in the design of the buckling restrained brace could be utilized in case of providing enough rigidity for the existing reinforced concrete structure.

- As expected, the buckling restrained braces were more effective in reducing the seismic response of the original frame when subjected to earthquakes with 10% probability of exceedance in 50 years since their contribution to seismic energy dissipation increased in the inelastic range.

- It was found that frame system became more rigid and the free vibration period of the system decreased by adding buckling restrained braces to existing reinforced concrete frames.

- Results of nonlinear dynamic analyses demonstrated that ratio of maximum storey drifts of buckling restrained braced frames and existing frames was about 4.4 times less. Results of nonlinear dynamic analyses with the earthquake group 50% probability of exceedance in 50 years with earthquake group showed that the reduction in the deformation of structural system was more limited.

- Similar findings was also observed in the curves of storey drifts, especially with %10 probability of exceedance under earthquake group, the buckling restrained braces worked more effectively and that the ratio of storey drift in the inter-stories was more uniformed and balanced.

- Analysis of the results indicated that the proper design of buckling restrained braces with adequate strength and stiffness remarkably reduced the demand for structure’s non-linear deformation.

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