Comparison of the Behavior of Steel Structures with Concentric and Eccentric Bracing Systems

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ABSTRACT

Over the years bracings were used as part of an effective lateral load resisting system to enhance stiffness and reduce deformation. Structures need to be strong and at the same time ductile. Besides they also need to be economical and constractable. In recent years with increased concerns about seismic activities more research had been carried out to find the response of structures to seismic forces. The main objective of this study is to investigate the behavior of Concentric and Eccentric Braced (CBF, EBF) steel frames by using linear dynamic, nonlinear time history and nonlinear static pushover analysis. Hence it was decide to investigate the two basic types of bracing systems, X-shape and Λ-shape. Earthquake impact on structure depend on many factors include structures height, number of story, each story height and number of bays on plan. For this reason design and performance analyses were carried out on 4, 8 and 12 story buildings with 3x3 symmetric and 3x5 asymmetric bays on plan. According to ASCE7-10, the value of the deflection amplification factor coefficient (c_d) depends on the type of bracing. This study indicated that it also depends on the number of floors but it is independent from number of bays. In all structures, the initial stiffness of CBF was more than that of EBF. Pushover graphs show that EBF braces have more ductility than CBF. Target shift is larger in both directions for EBF when compared to those of CBF. On the other hand, for 3x3 bays increase in the number of floors lead to increase in the target shift for both CBF and EBF.

Keywords: Eccentric Brace Frame, Concentric Brace Frame, Linear dynamic analysis, Nonlinear Dynamic Time History Analysis, Pushover Analysis

ÖZ

Dayanıklılığı artırmak ve deformasyonları azaltmak için destek sistemleri yıllardır etkin yatay yük taşıma sistemi olarak kullanılmaktadırlar. Yapılar güçlü ama ayni zamanda da sünek olmalıdırlar. Ayrıca yapılar ekonomik ve uygulanabilir olmalıdırlar. Son yıllarda sismik aktivitelerle ilgili artan endişeler sonucunda yapıların sismik yüklere karşı tepkisini bulmak için araştırmalar bu alanda yoğunlaştırılmıştır. Bu çalışmanın ana hedefi doğrusal dinamik, doğrusal olmayan dinamik ve statik (itme) analiz yöntemlerini kullanarak Ortak Merkez Destekli ve Dış Merkez Destekli Çelik Çerçevelerin (OMDÇ ve DMDÇ) davranışını incelemektir. Bu nedenle iki temel destek sistemi X-şeklinde ve Λ-şeklinde incelenmiştir. Yapılarda deprem etkisi birçok parametreye bağlıdır. Bu çalışmada dikkate alınan parametreler şöyledir: yapı yüksekliği, kat sayısı, herbir kat yüksekliği ve planda akslar arasındaki bölme sayısı. Bu nedenlerle 4, 8 ve 2 katlı, planda 3x3 simetrik ve 3x5 asimetrik bölmesi olan yapılar tasarlanıp performans analizleri yapılmıştır. ASCE7-10 standardına göre sehim büyütme faktörü katsayısı (c_d) kullanılan destek türüne bağlıdır. Bu çalışma, c_d katsayısının ayni zamanda kat sayısına da bağlı olduğunu fakat plandaki bölme sayısından bağımsız olduğunu göstermiştir. Bu çalışmada incelenen tüm yapılarda OMDÇ başlangıç ricitliğinin DMDÇ'ninkinden daha fazla olduğu gözlemlenmiştir. İtme analizi grafiklerine göre DMDÇ'nin OMDÇ'ye göre daha sünek olduğu görülmüştür. OMDÇ ile karşılaştırıldığında DMDÇ'sinin her iki yönde hedef kayması daha büyüktür. Diğer taraftan kat sayısında yapılan artış her iki destek sistemini kullanan (OMDÇ DMDÇ) ve 3x3 plan bölmesi olan yapılarda hedef kaymasının artmasına neden olmuştur.

Anahtar Kelimeler: Ortak Merkez Destekli Çerçeve, Dış Merkez Destekli Çerçeve, Doğrusal Dinamik Analiz, Doğrusal Olmayan Dinamik Analiz, İtme Analizi.

DEDICATION

The weakest link in a chain is the strongest, because it can break it.

Stanislare.J.Lee

I dedicate this thesis to my parents, for their endless love, support and encouragement.

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LIST OF ABBREVIATIONS

BRBF	Buckling Restrained Braced Frame
CBF	Concentrically Braced Frame
CMDB	Cast Molecular Ductile Bracing System
CP	Collapse Prevention
DL	Dead Load
EBF	Eccentrically Braced Frame
IO	Immediate accompany
KBF	Knee Braced Frame
LFRS	lateral Force Resisting System
LL	Live Load
LS	Life safely
MBF	Mega Braced Frame
MRF	Moment-Resisting Frame
RLSD	Richard Lees Steel Deck
SCBF	Special Concentrically Braced Frame
SC-CBF	Self-Centering Concentrically Braced Frame
SRSS	Square Root of Sum of Squares

Chapter 1

INTRODUCTION

1.1 General

Earthquake is one of the major natural disasters to happen on earth. On average, 100,000 people die annually due to earthquakes around the world (Engelhardt, M. D. and Popov, E. P. 1989). Between the years 1926 and 1950 the cost of earthquakes is estimated as 10 billion dollar (Engelhardt, M. D.and Sabol, T. A. 1997). According to a report by UNESCO around 200 villages were destroyed due to earthquakes in central Asian countries. Hostrilocal writings state that the men were much concerned about the hazards of earthquake for many years. Very sensitive seasonal graphs were used to study the waves from distant earthquakes during the first half of 1900 (Hjelmstad, K. D. and Popov, E. P. 1983). The seismologists were not able to carry out work on the fundamentals of earthquake since the amplitude of nearby earthquakes with magnitude 5 exceeded the dynamic range of usual seismographs. In recent years the situation has changed. The earthquakes with 6.5 magnitudes also have strong motion record. Fast computers and digital recorders are used by the seismologists to study earthquakes more in detail.

Structures are important for human life and in earthquake prone regions seismic resistant structures are necessary. Therefore, over the years there has been considerable improvement in structural design and construction techniques in areas with seismic activity. For example, countries in the developed world with earthquake vulnerability have strict standards for structures; houses, bridges, tunnels, stadiums, etc., to prevent earthquake damage and hence loss of life. After earthquakes generally structures subjected to severe damage are those that were designed and built before these seismic standards were introduced. Some of the developing countries also have standards for earthquake design however regulations are often ignored due to lack of enforcement of these rules and inadequate awareness of the importance of these matters. Japan is one of the very good examples of a country who managed to build earthquake-resistant structures. Buildings are strengthened in such a way that they are strong and rigid enough to resist seismic forces but at the same time they are ductile enough to absorb the sesimic energy without collapse. High rise buildings are supported with braces and shock absorbers that are bolted to inner steel skeletons. These allow movement but prevent catastrophic sway (Moghadam, 2006). Therefore, nowadays different construction methods and bracing system have been used to prevent the losses due to earthquake.

There are different types of braced frames that can be used for construction of buildings. According to Okazaki and Taichiro (2004), rigid frame systems are not particularly suitable for construction of buildings taller than 20 stories. The reason behind is that the bending of the columns and beams causes the deflection of shear racking component which leads to story drifts. Addition of braces, such as, V-braces or diagonal braces within the frame transforms the system into a vertical truss and it gradually eliminates the bending of beams and columns. As the horizontal shear is primarily absorbed by the web members instead of columns, therefore high stiffness is achieved. The bracing configurations may include I-beams or I-columns, circular, square or rectangular hollow sections (Suita.K and Tsai. K, 2003) and single or back to back double angles or channels connected together (FEMA, 2000). The braces are usually connected to the framing system via gusset plates with bolted or welded frames.

The braced frames may be considered as cantilevered vertical trusses resisting lateral loads, initially through the axial stiffness of columns and braces. The columns, diagonals and beams have different functions (FEMA,2000).The braced frames may be considered as cantilevered vertical trusses resisting lateral loads, where the primary function can be carried out through the axial stiffness of columns and braces. In order to resist the overturning movement, compression on the leeward column and also on the windward column, the columns act as chords. In triangulated truss, the beams are subjected to axial loads. Only when the braces are eccentrically connected to the truss beams then braces may undergo bending.

As the lateral loads are reversible, the braces undergo both compression as well as tension. The braces are mostly designed for the stringent case of compression. Resistance to horizontal shear forces is the principal function of web members. Depending on the configuration of the bracing, substantial compressive forces may be picked up, as the columns shorten vertically under the load of gravity.

1.2 Importance of Bracing Systems

Bracing systems are well represented in the overall evolution of the steel structures. Bracing systems are an assembly of structural elements where the traditional rectangular frame is added with diagonals (braces). The diagonals intersect axially with the elements of the frame thus forming a structure that bears horizontal loads with the help of its bracing members that are mainly subjected to axial forces.

Historically seismic engineering relies on the accumulated knowledge of the theory and practice of structures but extends the theory and practice viewed in the light of specific seismic actions.

Earthquakes might cause very significant damages on structures that can be prevent or reduced by using suitable structural design. Hence, researchers and practicing engineers developed concepts for the design of structures that may successfully absorb seismic energy and preserve structural capacity during and after the seismic impact. In general, bracing systems are used to sustain the effects of seismic actions by operating in elastoplastic phase and are subjected to large displacements and hence produce significant deformations. Over the years these bracings are classified into two as Concentrically Braced Frames (CBFs) and Eccentric Braced Frames (EBFs). Bracing systems are designed for reversal of inelastic response ensuring sustainable hysteretic behavior and leading to absorption of seismic energy without significant decline in ductility level.

1.3 Types of Braces

Depending on the geometric characteristics the braces can be classified into eccentric braced frames and concentric braced frames (Hong.J, 2005). The member forces of CBF are axial as the columns, beams and braces intersect at a common place. The eccentrically braced frames utilize the axis offsets to deliberately introduce flexure and shear into the framing beams. Increasing ductility is the major goal of the eccentrically braced frames. Depending on the magnitude of force, length, clearances and stiffness of the members the diagonal members of the concentrically braced frames can be made of T-sections, channels, double angles, tubes or wide flange shapes. Typical bracing arrangements for steel structure are shown in Figure 1.1.

In majority of the world's tallest buildings, bracing has been used to provide lateral resistance (Johnson. M, 2000). The fully formed triangulated vertical truss is the most efficient type of bracing system for this purpose.

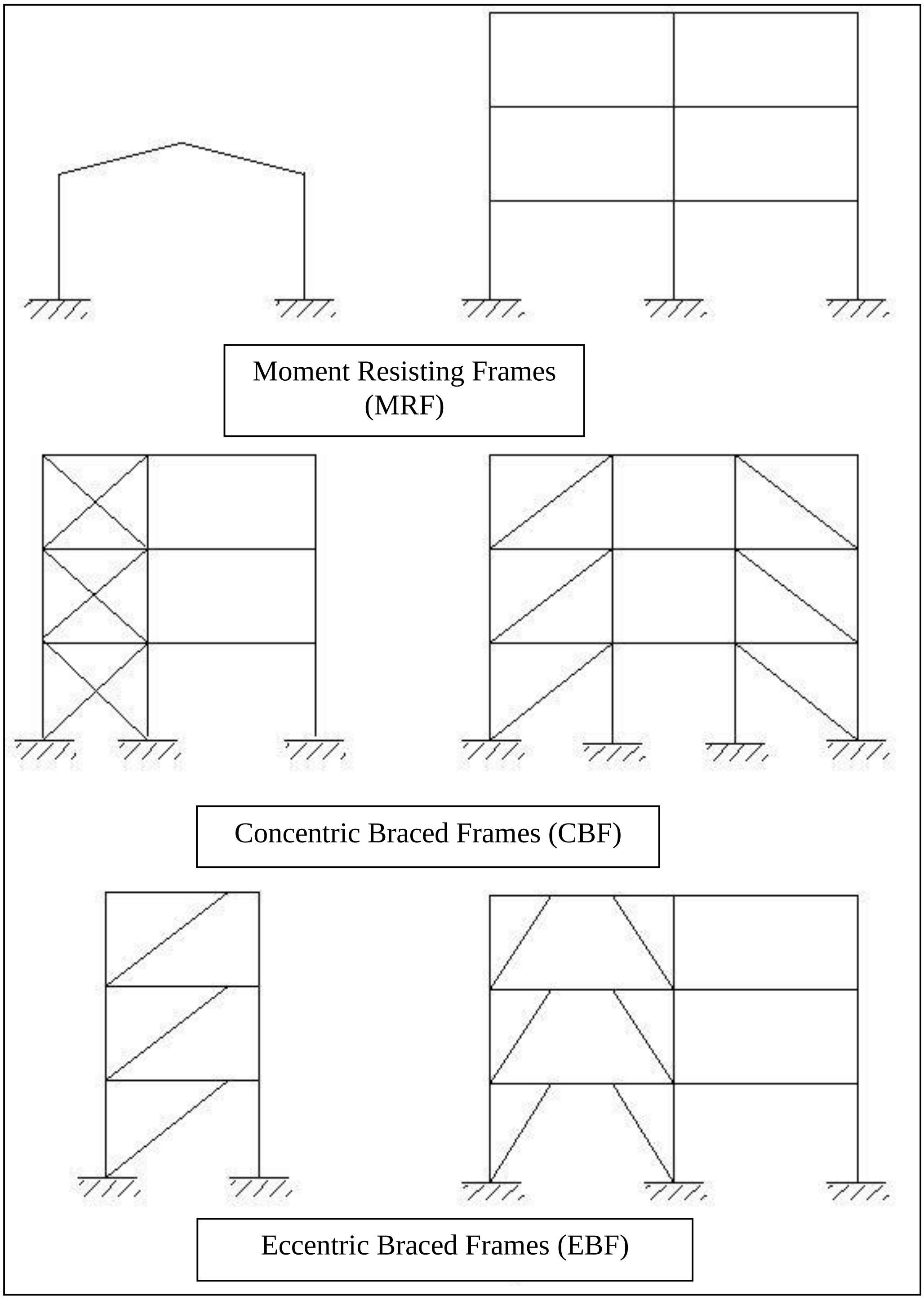


Figure 1.1: Typical bracing arrangements in steel structure
(Okazaki, 2004)

1.3.1 Concentrically Braced Frames

Concentrically braced frames are a different class of structures; they resist the lateral load through a vertical concentric truss system. Since CBFs tend to provide high strength and stiffness they are efficient in resisting the lateral forces. . When subjected to less favorable seismic response they tend to have low drift capacity and high acceleration. In seismic areas structures with CBFs are common. Different arrangements for CBF in practice are shown in Figure 1.5.

In order to maximize inelastic drift capacity a special class of CBF_s is proportioned and detailed. These frames are called Special Concentrically Braced Frames ($SCBF_s$). This type of CBF system is defined for structural steel and composite structure only. The primary source of drift capacity in $SCBF_s$ is through the buckling and yielding of diagonal braced members. Adequate axial ductility is ensured through the detailed and proportionate rules for the braces. SCBF is the same in configuration as CBF but there is a very big difference in the design philosophy. Braces in SCBF are required to have gross-section tensile yielding as their governing limit state so that they will yield in a ductile manner. Since the stringent design and detailing requirements for SCBF are expected to produce more reliable performance when subjected to high energy demands imposed by severe earthquakes, building codes have reduced the design load level below that required for CBF.

As opposed to the ductility approach for the SCBF, the design basis for the CBF is primarily based on strength and more emphasis has been placed on increasing brace strength and stiffness, primarily through the use of higher design loads (R=4.5) in order to minimize inelastic demand (Yoo J.H, Roeder C., and Lehman D, 2008). CBFs consist of two main components: frame and diagonals (bracings). Diagonals are the hallmark of the bracing system. They define its significant stiffness and stressed state in the elements of which they are composed. The frame consists of vertical elements mostly columns and horizontal members mostly beams or struts. Horizontal and vertical members form the frame. Diagonals can be called also bracings. The connection between the frame and diagonals is performed in joints. The main geometrical parameters characterizing CBFs are the distance between columns and the distance between the beams (inter story height).

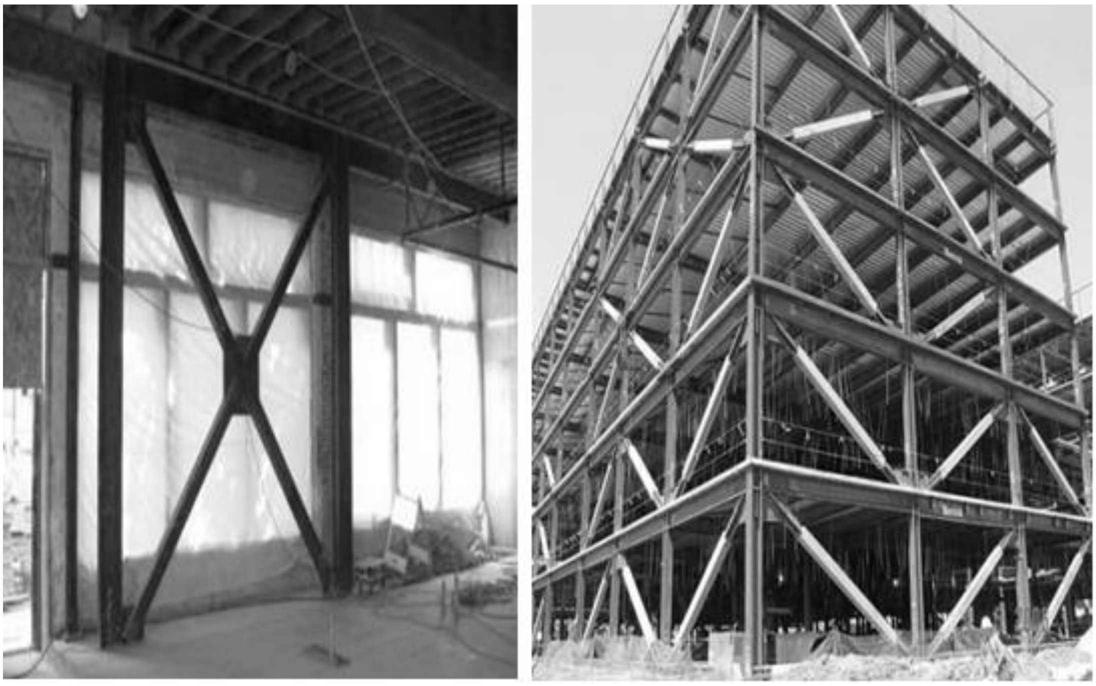


Figure 1.2: Examples of concentrically braced frames in practice (a) cross bracing (b) diagonal bracing (www.civilweb.ir)

1.3.2 Eccentrically Braced Frames

Eccentrically Braced Frames (EBFs) are a lateral force resisting system that combines high elastic stiffness with significant energy dissipation capability to accommodate large seismic forces. A typical EBF consists of a beam, one or two braces and columns. Its configuration is similar to traditional braced frames, with the exception that at least one end of each brace must be eccentrically connected to the frame. The eccentric connections introduce bending and shear forces in the beam adjacent to the brace. The short segment of the frame where these forces are concentrated is called a link.

EBFs are an alternative to the more conventional Moment-Resisting Frames (MRFs) and the Concentrically Braced Frames (CBFs), trying to combine the individual advantages of each. Figure 1.2 shows typical EBF configurations.

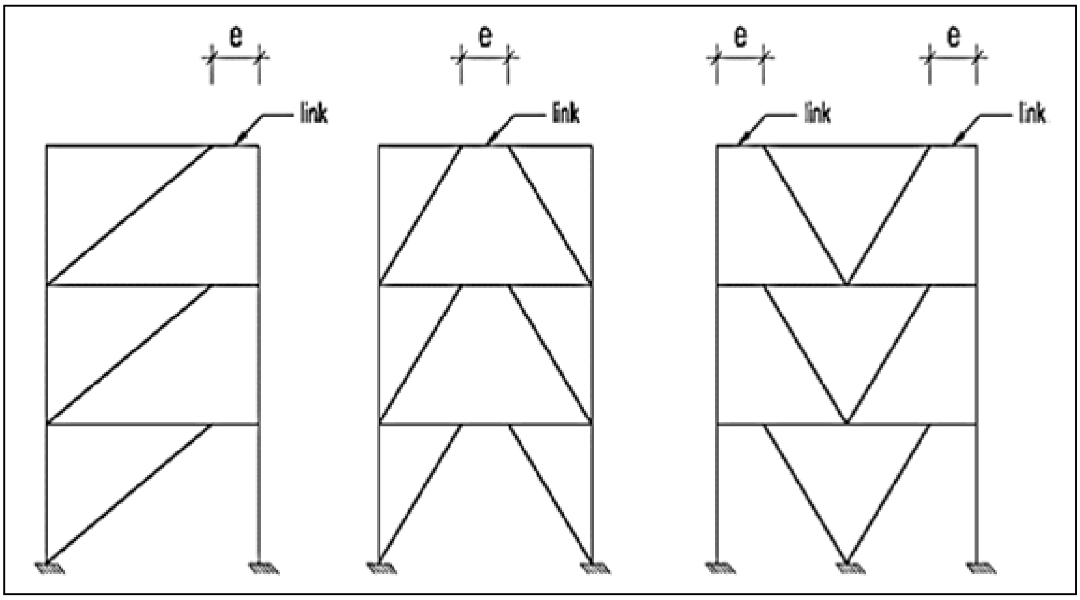


Figure 1.3**:** Typical EBF configurations (Okazaki, 2004)

In EBFs, the axial force which is carried from the diagonal brace is transferred to the column or to another brace through shear and bending of the link. A well designed EBF permits development of large cyclic inelastic deformations. The inelastic action is restricted primarily to the links, which are designed and detailed to be the most ductile elements of the frame (Engelhardt Popov, 1989).

The ductile behavior of the link permits achieving ductile performance of the structure as a whole. Links in EBFs are designed for code level forces, and then detailed in such a way so that non-ductile failure modes such as local buckling, lateral-torsional buckling, or fracture, will be delayed until adequate inelastic rotations are developed. On the other hand, the diagonal braces, the beam segments outside the links, and the columns are not designed for code level seismic forces, but rather for the maximum forces generated by the fully yielded and strain hardened links (Engelhardt.Popov, 1988).Figure 1.4 illustrates the eccentrically braced frames in practice.

Figure 1.4: Eccentrically braced frames in practice
(www.civilweb.ir)

This approach ensures that inelasticity occurs primarily within the ductile links elements.

The forces in an EBF link are characterized by a high shear that is constant along its entire length, reverse curvature bending and a small axial force. On the other hand, the beam segment outside the link as well as the brace, are subjected to high axial forces and bending. The force distribution in EBF can be seen in Figure 1.5.

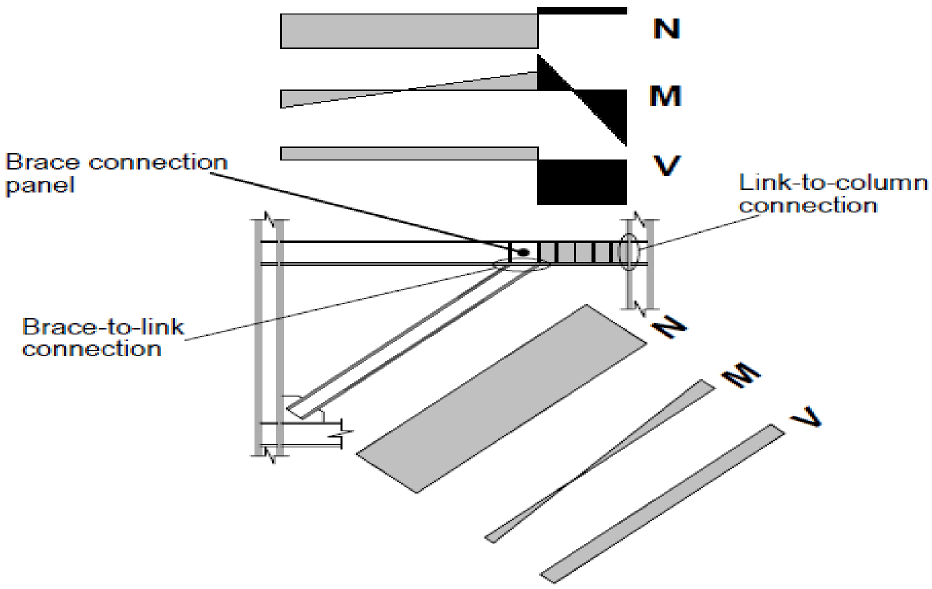


Figure 1.5: Typical force distributions in EBFs
(Okazaki 2004)

The eccentrically braced frame tries to combine the stiffness and strength of a braced frame with its energy dissipation and inelastic behavioral characteristics of a moment frame. Figure 1.5 shows deliberate eccentricity that is formed between the beam-to-bracing connection and beam-to-column connection. In this system the shear load will be distributed to the whole structure. The shear yielding of the beam is a relatively well defined phenomenon. The load required for shear yielding capacity of a beam with given dimension can be calculated fairly accurately. By using of overload factors the braces and columns can also be designed to carry more loads than could be imposed on them by shear yielding.

1.4 The Use of Steel Special Concentrically Braced Frames (SCBF)

Special Concentrically Braced Frames (SCBFs) are a special class of CBF that are proportioned and detailed to maximize inelastic drift capacity. SCBF system is generally used for structural steel and composite structures in high seismicity areas. SCBF are generally economical for low rise building. It is preferred over special moment frames because of its material efficiency and smaller depths of column required. SCBF are only possible for the buildings that can accommodate braces in their architectural layout otherwise special moment frames are better suited for the building frames.

The performance of the SCBF is based on providing high level of brace ductility to achieve large inelastic drifts (AISC, 1997). The SCBFs are designed by using capacity design procedures, in which the braces serve as the fuse of the system. Over strength of the braces can be sometimes beneficial, but care should be taken in order to maintain a well-proportioned design and also to avoid concentration of ductility demands.

The tensional response of the building can be controlled by the braced frames and these are most effective in the building perimeter. ASCE 7 allows buildings with two bays on each of the presumed four outer lines and these are considered sufficiently redundant. These types of layouts are good for torsion control. In the core of the structure the SCBs are often used. Figure 1.6 shows schematic structural model of SCBF panel. It is advantageous to spread the overturning forces out over several bays. This should be done to reduce the anchorage forces on the foundation. It is critical to ensure that the brace ductility remains the primary source of inelastic drift.

The principle behind the designing of the special concentrically braced frames is that the special concentrically braced frames develop the lateral stiffness and strength which is needed to assure the performance behavior. Research stated, $SCBF_s$ are stiff, strong and also are more economical lateral load resisting systems for low- rise building in areas of high seismicity if they are designed properly (lumpkin,2012). SCBF's are capable of much more post elastic response than CBF's. They're detailed so that you have more plastic response prior to brace fracture. In CBF's you get a limited amount of plastic response prior to brace fracture. This makes SCBF's a more reliable system in large seismic events. During the designing of the special concentrically braced frames the beams and the columns are not the goals. So these factors are not much affected. To achieve the performance which is desired, a large number of ductile detailing requirements are applicable. The recent study by Hsiao et al, (2013) states that the increase in inelastic deformation capacity can be developed but some modifications need be done in the connection designs.

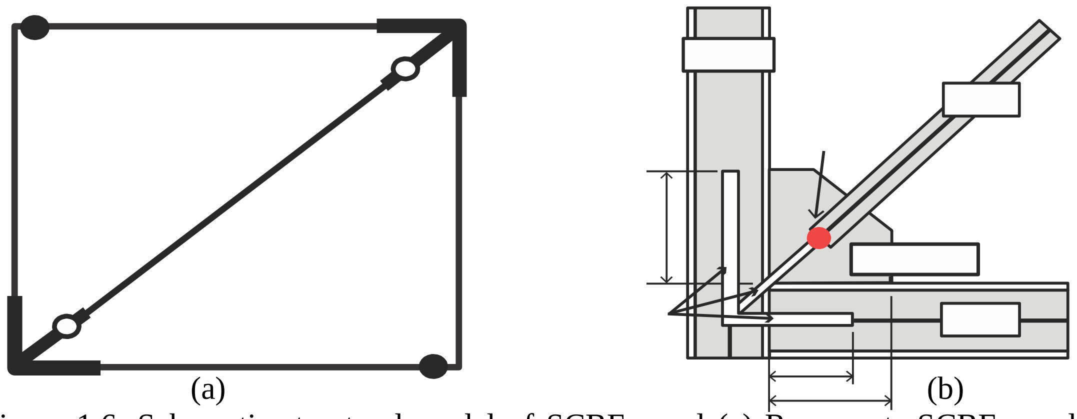


Figure 1.6: Schematic structural model of SCBF panel (a) Represents SCBF panel configuration with rigid links, pin connections and nonlinear spring (b) Represents geometric details identifying typical link lengths and nonlinear spring location (Hsiao et al, 2013)

1.5 Research Objectives

There had been numerous researches on the seismic behavior of frames with different bracing systems. They used different approaches and methods to understand the economy and effectiveness and ways to control the damages due to seismic forces.

In this research it was aimed to analyze and compare behavior of steel structure with concentric and eccentric bracing system. There was variety of concentric and eccentric bracings but only two of them are considered in this study. For concentric bracing X-shape and for eccentric bracing Λ-shape is investigated .Previous research indicates that there are many factors to consider and understand the impact of earthquake on different structures. These parameters may have huge effect in structural behavior. Structural height, number of story, story height parameters. For this reason it was decided to investigate structural behavior by considering the height and different number of stories. Structures with 4 floors, with 8 floors and finally12 floors, were designed. On the other hand the behavior of structure at position of bays can be different.

Therefore, structure 3x3 bays symmetric and 3x5 bays asymmetric with 4, 8 and 12 floors were used for the analysis. All these cases were analyzed with X bracing (cross) and with Λ bracing (inverted V) using liner dynamic, dynamic time history and nonlinear static pushover analyses. Hence 36 building models were used and the details are given as follows:

4, 8 and 12 story having 3x3 bay plan layout with X and Λ bracing system

4, 8 and 12 story having 3x5 bay plan layout with X and Λ bracing system.

1.6 Guide to Thesis

This thesis is comprised of five chapters. Chapter one includes general idea and information about Earthquake worldwide and going more in details about Eccentric and Concentric bracing system in steel structure. Chapter two includes literature review, being divided into three sections. The first section is devoted to a short introduction and optimization of steel frame. Section two explains about concentrically braced frames (CBF) and review of literature for last ten years. Section three go more in deep about eccentrically braced frames (EBF) .Chapter three is associated with the methodology. This chapter is also divided into different sub titles in which the details about the methodology are explained extensively. Chapter four includes results and discussion on topic. Chapter five include final conclusion about results.

Chapter 2

LITERATURE REVIEW

2.1 General

Considerable research has been conducted on the behavior of concentric braced frames and eccentric braced frames. The available collection of literature extends over several decades and is rapidly growing. Therefore, it cannot adequately be summarized in a brief chapter. Instead, an overview of major references is provided here along with useful citations to previous studies that contain detailed reviews of related literature. The literature review in this chapter is divided into three categories as, CBFs reviews and EBFs reviews and significance of this study.

2.2 Concentrically Braced Frame

CBFs viewed in the light of the needs of seismic engineering are subjected to researcher's studies by the end of the 1970s. Those words are mainly concentrated on experimental studies, theoretical research and analysis the behavior of structures having CBFs in past earthquakes. In the past 30 years of worldwide research interest are clearly outlined the pros and cons in the elasto-plastic behavior of bracing structures. This naturally raises the interest of scientists on optimizing the behavior of such frames without losing its strengths.

This led to the use of different design methods to improve the behavior of CBFs. Studies on this subject is also in the following overview.

2.2.1 Behavior of CBFs in Past Earthquakes

The behavior of steel structures designed to withstand strong earthquakes through CBFs is the source from which engineers and researchers can draw information and to make conclusions about the appropriateness of the applied design techniques and methods of analysis. This section also reviews the experience gained from three previous earthquakes. Experience from Northridge earthquake was reviewed by Trembly, Timler, Bruneau and Filiatrault (1995) whilst Kobe earthquake was reviewed by Trmebly. et.al (1996). Furthermore Scawthorn et al (2000) prepared an extensive report on Kocaeli, Turkey earthquake and the consequences. Summary of the behavior performed by CBFs in the past earthquakes are summarized as follows:

- CBFs showed low cycle fatigue failure especially in cases of diagonals having box cross sections with low slenderness ratio.
- A common mode of failure of box diagonals is the fracture of the reduced cross section in the connection between the box and the gusset plate or block shear in connections.
- CBFs with slender diagonals when being framed by a continuous members and having capacity designed gusset plate welds, demonstrated surprisingly good behavior in the Kobe earthquake.
- The loss of stability of a diagonal bracing due to the out of plan buckling may cause damage to the building external cladding. This damage of cladding may cause debris falling from height which is potential danger to human life.

2.2.2 Review of Previous Experimental Studies

Sabelli et al (2003) studied the seismic demands of steel braced frame buildings with buckling-restrained braces. This paper highlighted research being conducted to identify ground motion and structural characteristics that control the response of concentrically braced frames, and to identify improved design procedures and code provisions. They assessed the seismic response of three and six story concentrically braced frames utilizing buckling-restrained braces. Results from detailed nonlinear dynamic analyses were then examined for specific cases as well as statistically for several suites of ground motions to characterize the effect on key response parameters of various structural configurations and proportions. Results presented in this paper have focused on applications of buckling restrained bracing members. The results indicated, that buckling-restrained braces provide an effective means for overcoming many of the potential problems associated with special concentric braced frames. To accentuate potential difficulties with this system, numerical modeling and design assumptions were intentionally selected to maximize predicted brace demands and the formation of weak stories.

Sarno et al (2004) studied bracing systems for seismic retrofitting of steel frames. The seismic performance of steel moment resisting frames (MRFs) retrofitted with different bracing systems were assessed in their study. The three types of braces utilized were special concentrically braces (SCBFs), buckling-restrained braces (BRBFs) and mega-braces (MBFs). The author designed a 9-story steel perimeter MRF with lateral stiffness that was insufficient to satisfy code drift limitations in high seismic hazards zones. The SCBFs, BRBFs and MBFs were then been used to retrofit to the frames. Result from inelastic analyses demonstrated that MBFs were the most cost persuasive. It was also show that reduction in inter story drifts was equal to 70% when compared to original MRF. The author showed that MRFs with insufficient lateral stiffness can be retrofitted with diagonal braces in the present analytical work.

Elghazouli et al (2008) analyzed the seismic behavior of steel-framed structures according to Eurocode 8. The paper evaluate was on the provisions of Eurocode 8 regarding to the seismic design of steel frames. The author studied both the MRF and CBF configurations. The design concepts, behavior factors, ductility considerations and capacity design verifications, in terms of code requirements were examined. The study showed that the implications of stability and drift requirements along with some capacity design checks in moment frames and simultaneously with the distribution of inelastic demand in braced frames were the areas of careful consideration required in the process of design.

Chen et al (2008) studied the seismic performance assessment of CBF buildings. A 3-story and 2-story X-SCBF and BRBF systems were analyzed using Open SEES to identify improved performance-based design and analysis procedures and to improvise the understanding of the behavior of conventionally braced and BRBF. A three-story model building has been designed using 1997 NEHRP and ASCE 7-05 showed similar performance with respect to the damage concentration when it was statistically analyzed. The demands on the braces and framing components were reduced along with the tendency to form a soft story when the R value is reduced from 6 to 3. Thus experimental test results were used to rectify the analytical models and validate the seismic performance of SCBF and BRBF.

Massumi et al (2008) studied the strengthening of low ductile reinforced concrete frames using steel X-bracings. The authors experimentally evaluated the use of steel bracings in concrete framed structures. A series of tests has been conducted on RC model frames, 8 one bay-one story with 1:2.5 scale. The objectives of the tests were to determine the effectiveness of cross bracings with bracing connections to the concrete frames and to increase the in-plane shear strength of the concrete frames. The model frames were tested under constant gravity and lateral cyclic loadings. The ample increase in the lateral strength and displacement ductility of strengthened frames upon bracing was shown from the test result.

Roke et al (2008) studied the design concepts for damage-free seismic self-centering steel concentrically-braced frames .This paper highlighted the goal of providing the self-centering concentrically-braced frame (SC-CBF) systems which were being developed with the goal of providing adequate Nonlinear drift capacity without significant damage and residual drift under the earthquake loads. To evaluate the earthquake responses the static pushover and dynamic time history analyses were performed on several SC-CBF system. Under earthquake loading each SC-CBF was self-centered. To calculate the design demands for the frame members a procedure has been presented which was then validated with analytical results. The design procedure accurately predicted the member forces demanded by the earthquake loading.

Miri et al (2009) studied the effects of using asymmetric bracing on steel structures under seismic loads. The structure was categorized as irregular mass and stiffness source were not coinciding due its architectural layout. The irregular distribution of stiffness and mass of the structure combined with the asymmetric bracing on plan led to eccentricity and torsion in the structural frame. Since there is deficiency in ordinary codes to evaluate the performance of steel structures against earthquake then performance level or capacity spectrum can be used for design purpose. By applying the mentioned methods, it was possible to design a structure and predict its behavior against different earthquakes. According 5- story buildings with different percentage of asymmetry, due to stiffness, were designed. The static and dynamic nonlinear analyses were carried out by using three recorded seismic accelerations.

Viswanath et al (2010) investigated the seismic performance of reinforced concrete (RC) buildings rehabilitated using concentric steel bracing. Bracings were provided at the Peripheral columns. A four story building was analyzed for seismic zone IV as per IS 1893: 2002. The effectiveness of using various types of steel bracing X and Λ (inverted V) in rehabilitating the four story building were examined. The seismic performance of the rehabilitated building was studied with the effect of the distribution of the steel bracing along the height of the RC frame. The performance of the building was evaluated in terms of global and story drifts. It was concluded that the X type of steel bracing significantly contributed to the structural stiffness and reduces the maximum inter story drift of the frames.

Kangavar (2012) compared the seismic behavior of Knee Braced Frame (KBF) against Concentric Braced Frame (CBF) based on stiffness and ductility and utilized the software ETABS and Open System for Earthquake Engineering Simulation (OPENSEES).

2.3 Eccentric Braced Frame

The EBF can be considered as a hybrid structural system that combines the stiffness of conventional concentrically braced frames with ductility and energy dissipation capacity of conventional moment resisting frames. This is the most attractive feature of EBFs for earthquake resistant design.

2.3.1 Review of Previous Experimental Studies

Extensive experimental and analytical research was undertaken at University of California, Berkeley in the 1980's by Popov and his colleagues. After verifying the concept of eccentric bracing for seismic loads on small frames, studies were directed towards investigating the cyclic behavior of individual short shear links (Hjelmstad and Popov, 1983; MaIley and Popov, 1984). Kasai and Popov (1986) formulated criteria for link web buckling control under cyclic loads. The studies (Rides and Popov, 1987a) were concentrated on cyclic behavior of short links in EBFs with composite floors. A series of tests carried out by Engelhardt and Popov (1989, 1992) provided deep understanding on the behavior of EBFs with long links.

In addition to component testing, a full-size EBF was tested in Tskuba, Japan (Roeder et al, 1987) as well as a 0.3-scale replica on a shaking table at Berkeley (Whittaker et al. 1987). Both structures showed excellent overall behavior when subjected to several ground motions.

Hines (2009) also investigated the seismic performance of low ductility steel systems designed for moderate seismic regions. The primary model of the eccentrically braced frames consisted of two frames which include the shake table test of the eccentrically and concentrically braced frames dual system. As a part of the US-Japan cooperative research program the shake table test was carried out. The current study of the eccentrically braced frames is mainly focused on the updated material characteristics and also some of the new insights of the loading protocols.

The recent research also deals with the question of efficiency of the eccentrically braced frames. Some of the questions related to the eccentrically braced frames also raise the topic about the braced frames behavior that is to be discussed in light of the performance assessment tools. R-factor tests were carried out. A list of R factor that is used to study the performance of which is accepted universally is as follows.

Table 2.1: R and R_w factors related to Berkeley shaking table tests. (Okazaki, 2004)

System	ATC3-06 (1984)	SEAOC-R_w (1986)	UCB Rec	UCB U.Bound	BSSC 1998	SEAOC-R_w (1990)
1	2	3	4	5	6	7
CBF	5	---	2	---	5	8
CBDS	6	10	2.5	4.5	6	10
EBF	5	---	4	---	8/7	10
EBDS	6	12	5	6	8/7	12

Table 2.1 shows, R and R_w factors related to Berkeley shaking table tests which has been standardized since the early 90s. A clear understanding of the R factor is very important to understand the eccentrically braced frames. A graph is drawn based on the performance of the eccentrically braced frames where fragility curves resulted from the performance assessment are presented (Fig. 2.1).

The performance implied by the two fragility curves did not match the recommended 10% threshold. Figure 2.1 shows the performance of the eccentrically braced frames at factor R equal to 7. It also states that the design of the eccentrically braced frames which is more suited for the seismic regions may not necessarily provide level of the performance as implied by the R factor.

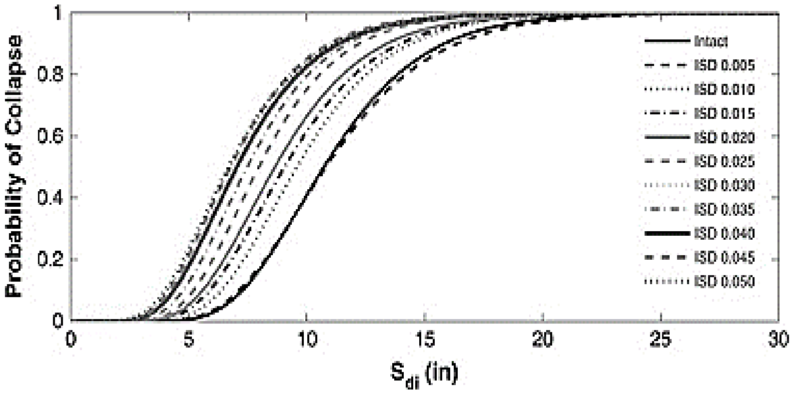


Figure 2.1: Fragility curves for EBF performance assessment.
(Palmer. K.D, 2012)

Özel, and Güneyis (2011) studied the effects of eccentric steel bracing systems on seismic fragility curves of mid-rise R/C buildings. In their study, the seismic reliability of a mid-rise reinforced concrete (R/C) building retrofitted using the eccentric steel braces was investigated through fragility analysis. As a case study, a six story mid-rise R/C building was selected. The effectiveness of using different types of eccentric steel braces in building was examined. The effect of distributing the steel bracing over the height of the R/C frame on the seismic performance of the retrofitted building was studied. For the strengthening of the original structure, K, and V type eccentric bracing systems were utilized and each of these bracing systems was applied with four different spatial distributions in the structure. For fragility analysis, the study employed a set of 200 generated earthquake acceleration records compatible with the elastic code design spectrum. Nonlinear time history analysis was used to analyze the structures subjected to those set of earthquake accelerations generated in terms of peak ground accelerations (PGA). The fragility curves were developed in terms of PGA for these limit states which were slight, moderate, major, and collapse with lognormal distribution assumption. The improvement of seismic reliability achieved through the use of K, and V type eccentric braces was evaluated by comparing the median values of the fragility curves of the existing building before and after retrofits. As a result of this study, the improvement in seismic performance of this type of mid-rise R/C building resulting from retrofits by different types of eccentric steel braces was obtained by formulation of the fragility reduction.

Nourbakhsh (2011) also studied the inelastic behavior of eccentric braces in steel structures. Nine frames were used with three different eccentric braces (V, Inverted V and Diagonal) and three different heights (4, 8 and 12 story). They were designed and analyzed linearly, and then the frames were assessed and reanalyzed by conducting the nonlinear static (pushover) analysis based on FEMA 440 (2005). Finally the results were evaluated using to their inelastic behavior and from economical point of view.

2.4 Significance of this Study

Lateral stability has been one of the important problems of steel structures specifically in the regions with high seismic hazard. The Kobe earthquake in Japan and the Northridge earthquake that happened in the USA were two obvious examples where there was lack of lateral stability in steel structures. One of the most important earthquakes in Iran was Rodbar earthquake in the northern part of Iran but its effect was observed even in capital city Tehran. This issue has been one of the important subjects of research for academics and researchers during the last decade in Iran. Iran is in an earthquake prone region hence for these natural hazard standards has recently being introduced.

Finally they came up with suggesting concentric, such as X, eccentric like inverted V and these were used in real life projects by civil engineers in Iran for several years.One of the principal factors affecting the selection of bracing systems is its performance. The bracing system which has a more plastic deformation capacity prior to collapse has the ability to absorb more energy while it is under seismic excitation. These factors can be changed by numbered floor story, number of bays and type of bracing system.

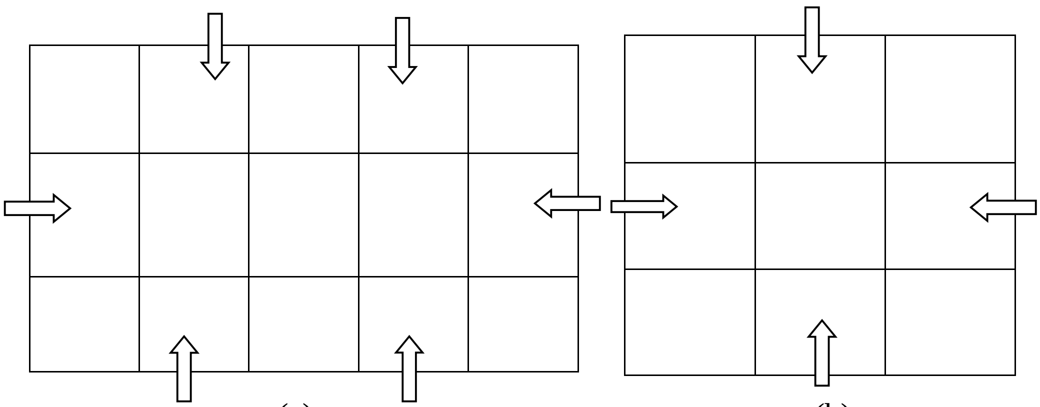
All steel braced frames which are to be designed and constructed should be braced with an appropriate type of bracing system. The two important parameters that can influence the type of the structural system and particularly the type of bracing systems in a structure are economy and performance parameters. By making a comparison with these two paragons, this study may help in shaping the foundation for new approaches for the evaluation of the bracing systems. Meanwhile, precise information relevant to the performance of various structural systems engenders higher quality in their design.

Chapter 3

MATERIALS AND METHODOLOGY

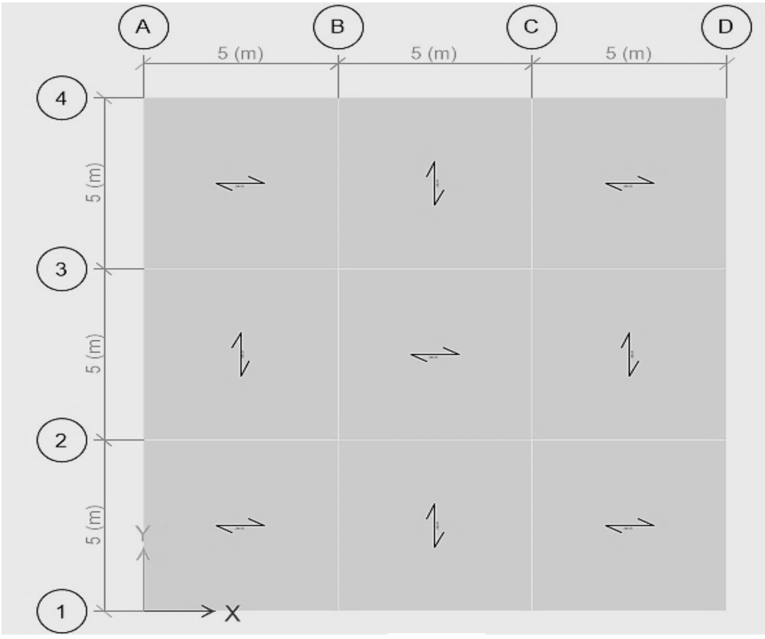
3.1 Introduction of Modeled Structures

A total of thirty six three-dimensional model structures were utilized to investigate the behavior of 4, 8 and 12 story structures having “X” and “Λ” shape bracing with plan layouts of 3 spans of 5 meters each in both dictions, 3 spans by 5 spans in x- and y-directions, respectively, each span length being 5 meters. Each story height was taken as 3 meters. Ribdeck AL 1.0 mm gauge galvanized steel deck produced by Richard Lees Steel Decking (www.richardlees.co.uk) was used for the composite floor system with normal weight concrete. In all models steel frames were braced both in x- and y-directions. Figure 3.1 shows location of bracing in this study.

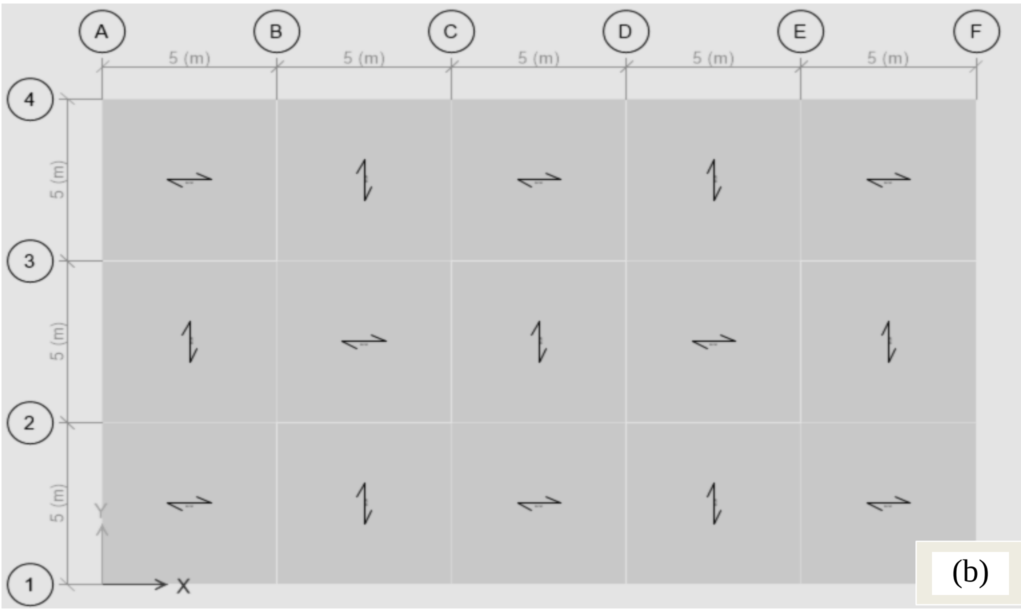


(a) (b)

Figure 3.1: Location of bracing in this study (a) 3x5 bay plan layout (b) 3x3 bay plan layout



(a)



(b)

Figure 3.2: One-way slab load distribution directions for (a) 3x3 bay plan layout and (b) 3x5 bay plan layout

3.2 Applied Specifications, Code and Standards

The design specifications and software used in this study are listed below:

- All Loading (Dead, Live and Earthquake) were adopted using ASCE7-10
- Spectral analysis and seismic loading were assessed according to ASCE7-10
- The building were designed according to AISC 360-10

- ETABS 2015 (https://www.csiamerica.com/)was used for the analyze and design of structural elements
- Accelerogram modification and drawing of spectrum were done by using Seismo signal.(http://www.seismosoft.com/)

3.3 Material Properties

3.3.1 Steel

Applied steel properties in this study are based on information which is listed in Table 3.1.

Table 3.1: Properties of steel

properties of material	
Mass per unit volume	780 kg/m^3
Weight per unit volume	7800 kg/m^3
Poission ratio	0.3
Yield strees,(f_y)	2400 kg/cm^2
Ultimate strength, (f_u)	4000 kg/cm^2
Elasticity module	2.06x10^6 kg/cm^2

3.3.2 Concrete

The concrete properties used are given in Table 3.2.

Table 3.2: Properties of concrete

Properties of concrete		
Mass per unit volume	240	kg/m^3
Weight per unit volume	2400	kg/m^3
Elacticity module	21882	kg/m^2

3.3.3 Nonlinear Material Properties

The nonlinear material properties used are according to tension strain and compression strain that are listed in table 3.3. Stress- strain curve of steel for our structure also is shown in Figure 3.3.

Table 3.3: Nonlinear properties (ASCE 7-10)

	Tension strain	Compression strain
IO	0.01	0.005
LS	0.02	0.01
CP	0.05	0.02

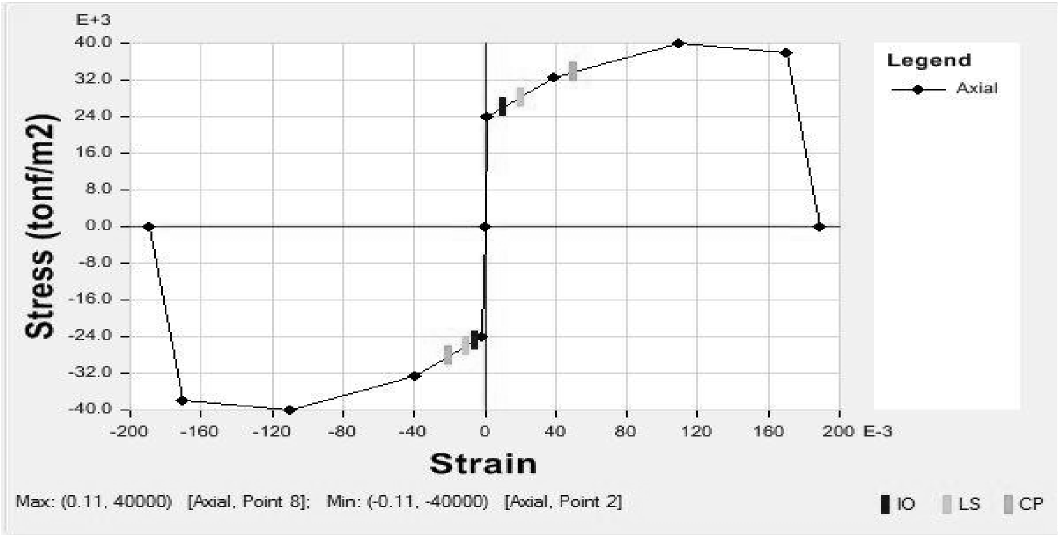


Figure 3.3: Stress- strain curve of steel (ASCE 7-10)

3.4 Loading of the Model Structures

3.4.1 Estimation of Floor Dead Load

3.4.1.1 Dead Load Calculation

For estimation of dead loads, density tables from ASCE7-10 code are used as input to the software so that the weight can be calculated by the program.

3.4.1.2 Detail of Galvanized Metal Deck for Composite Floor

There are many types of galvanized metal decks produced in different countries. For this research Richard Lees Steel Deck (RLSD) from UK is used (www.richardlees.co.uk). There are four different types of deck produced by RLSD Holorib, Ribdeck 80, Ribdeck E60 and Ribdeck AL. In this research Ribdeck AL with 1 mm gauge was used due to its properties which are known to minimize ribbed soffit and slab depth. Figure 3.4 shows a schematic cross section Ribdeck AL.

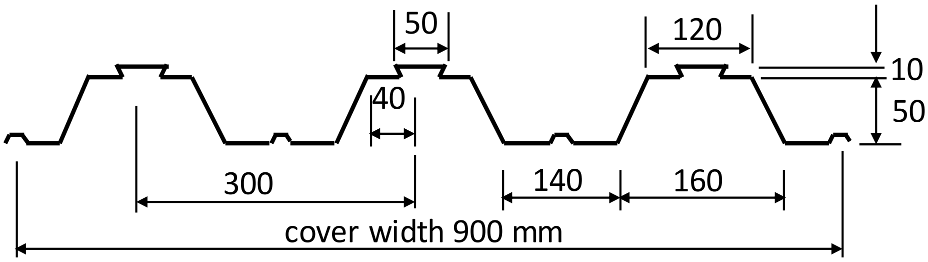


Figure 3.4: Ribdeck AL cross sectional dimensions.

Ribdeck AL galvanized metal deck section properties are listed in Table 3.4.

Table 3.4: Ribdeck AL section properties (per meter width)

Gauge	Self-Weight	Area	Inertia	Y_{NA}
mm	kg/m^2	mm^2	cm^4	mm
0.9	9.5	1.171	67.4	28
1	10.5	1.301	75.2	28
1.2	12.6	1.570	90.9	28

Selected slab depth was 120 mm, corresponding concrete volume was 0.095 m^3/m^2 and for 2.5 kN/m^2 imposed load and 1 mm gauge maximum span was 3.53 m.

3.4.1.3 Side Wall Load

The height of perimeter walls assumed to be 2.7 meter and the parapet for roof was assumed to be 0.8 meters. Therefore, the load applied on the perimeter beams at floor level was 620kg/m and for parapet at roof level was180kg/m.

3.4.2 Floor Live Load

According to ASCE7-10 code for residential structures, live load for typical story is 200 kg/m^2 and for roof it is maximum 150kg/m^2. Snow load can be calculated using the following formula:

$$\mathrm{Pr}=\mathrm{C_s.P_s},\ \begin{bmatrix} \mathrm{C_s} = 1 \\ \mathrm{P_s} = 150\,\frac{\mathrm{Kg}}{\mathrm{m^2}} \end{bmatrix} \rightarrow \mathrm{P_r} = 150\,\frac{\mathrm{Kg}}{\mathrm{m^2}} \qquad \text{(Eq 3.1)}$$

Therefore 150 kg/m^2 is used as live load for the roof. Partitioning load also was considered equal to 100kg/m^2.

3.4.3 Earthquake Load

ASCE7-10 was used to calculate the earthquake loads. Earthquake is assumed to act in two directions, x and y directions. Earthquake load parameters are given below and the area spectrum is shown in Figure 3.5.

Time period: $T= 0.02\ {h_n}^{0.75}$ (Eq 3.2)

Site properties: Washington

$S_S = 0.68$ $\mathrm{S_1} = 0.27$ Site class : D

$\mathrm{f_a} = 1.25$ $\mathrm{f_V} = 1.87$ $\mathrm{S_D} = 0.57$ $\mathrm{S_{D_1}}=0.33$

S_1 the mapped maximum considered earthquake spectral response acceleration

S_D design spectral response acceleration parameter

S_{D1} the design spectral response acceleration parameter at a period of 1.0 s

SS mapped MCER, 5 percent damped, spectral response acceleration parameter at short periods

F_a Acceleration-based site coefficient,

T the fundamental period of the structure(s)

h_n Structure height

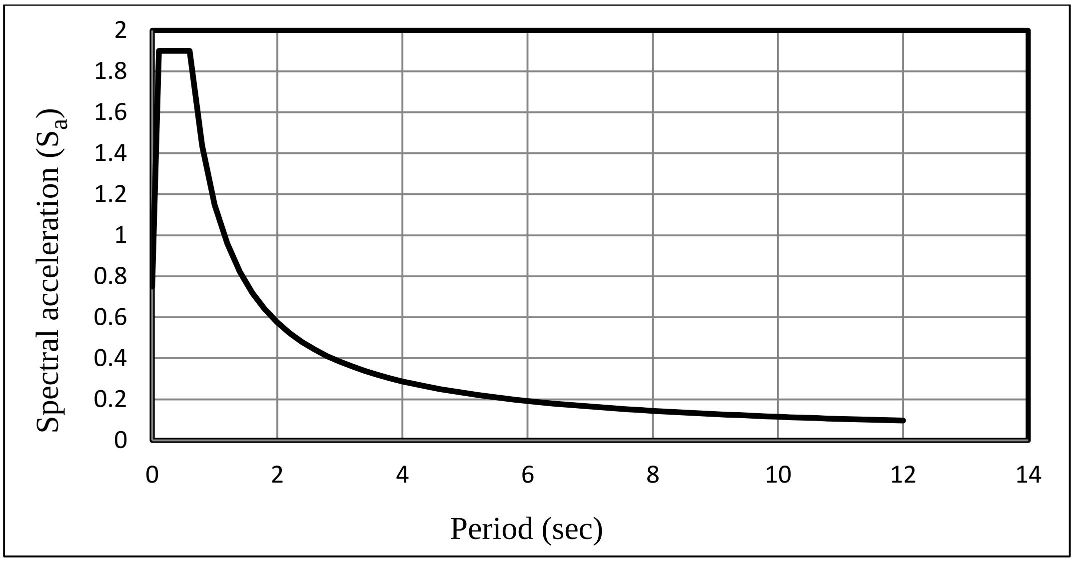


Figure 3.5: Area spectrum

In order to the earthquake loads different procedures for EBF and CBF are given as following:

EBF:

$$T = 0.03\ h_n^{0.75} \quad \text{(Eq 3.3)}$$

R= 8 $\Omega = 2$ $C_d = 4$ $I = 1$

CBF:

$$T = 0.02\ h_n^{0.75} \quad \text{(Eq 3.4)}$$

R=6 $\Omega = 2$ $C_d = 5$ $I = 1$

T the fundamental period of the structure(s)

C_d the deflection amplification factor

R the response modification coefficient

I the importance factor

Ω over strength factor

3.4.4 Gravity Loads Applied on the Structure

Gravity loads considered in this research are mass, of the building and live loads which are summarized in Table 3.5.

Table 3.5: Gravity loads applied on each floor of structure

	Self weight kg/m^2	Live load kg/m^2	Dead load kg/m^2
Floor load	-----	200	300
Roof load	50	150	230

3.5 Steel Sections Used for the Structural Design

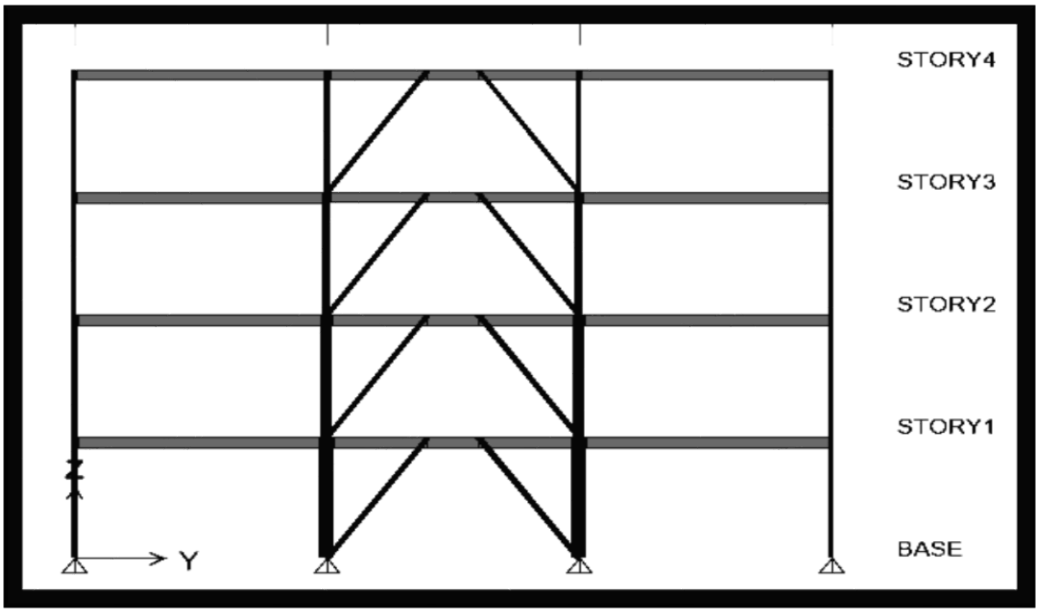
Steel sections used for design were beams, columns and bracings as they summarize in table 3.6.

Table 3.6: Steel section used for the structural design

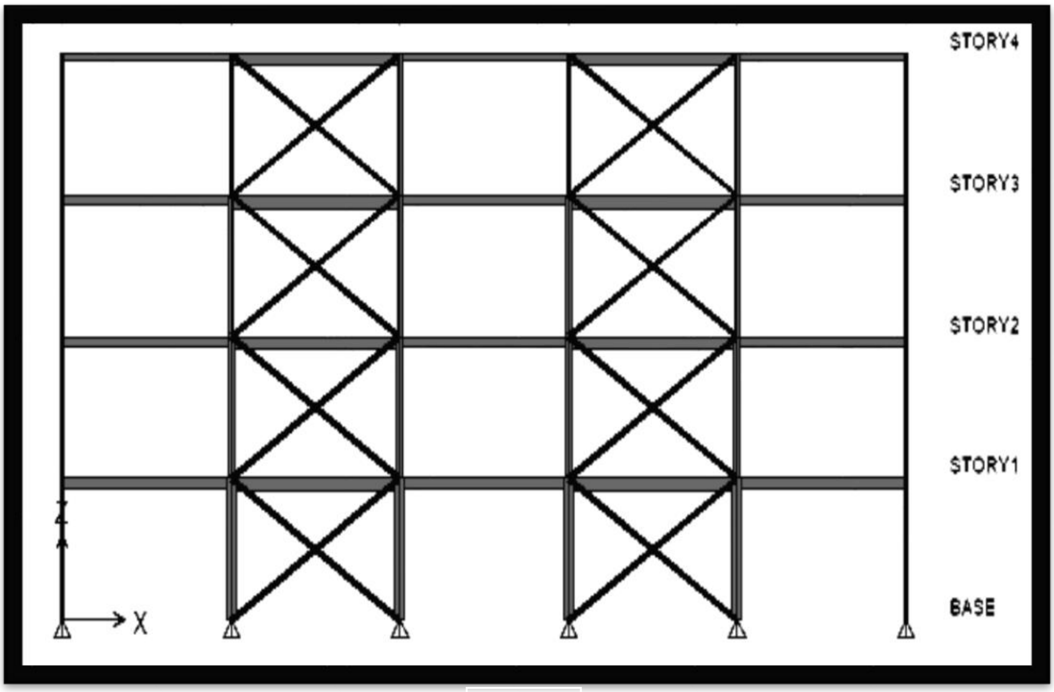
Beams	**IPE**
Columns	**HEB**
Bracings	**2 x UNP**

3.5.1 Steel Sections Used for Bracing System

A pair of UNP sections that are connected to each other, back to back, with a 1 cm space between the two members. The length of link beam is one meter for EBF brace. It’s clear that the cross-sections of braces are bigger in lower floors while toward upper floors, the cross-sections of the braces decrease. Also as the, increasing number of floors increased the structure gets heavier and consequently the design shearing force increases and therefore dimensions of the braces become bigger. Table 3.7 shows detail of sections that used for bracing system in our experiments. Figure 3.6 shows dimensions of structure with for EBF and CBF, for structures three with five bays.



(a)



(b)

Figure 3.6: (a) EBF and (b) CBF used in this study

Table 3.7: Different sections that used for bracing system in our experiments

Story	3x3 Bay					
	4 Story		8 Story		12 Story	
	EBF	CBF	EBF	CBF	EBF	CBF
1	2 UNP 14	2 UNP 18	2 UNP 14	2 UNP 14	2 UNP 20	2 UNP 26
2	2 UNP 14	2 UNP 18	2 UNP 14	2 UNP 14	2 UNP 20	2 UNP 26
3	2 UNP 12	2 UNP 16	2 UNP 14	2 UNP 22	2 UNP 18	2 UNP 22
4	2 UNP 10	2 UNP 16	2 UNP 14	2 UNP 22	2 UNP 18	2 UNP 24
5			2 UNP 12	2 UNP 20	2 UNP 18	2 UNP 24
6			2 UNP 12	2 UNP 20	2 UNP 16	2 UNP 24
7			2 UNP 10	2 UNP 16	2 UNP 16	2 UNP 22
8			2 UNP 10	2 UNP 16	2 UNP 16	2 UNP 22
9					2 UNP 14	2 UNP 20
10					2 UNP 14	2 UNP 20
11					2 UNP 12	2 UNP 16
12					2 UNP 12	2 UNP 16

3.6 Calculation of Structures Weight

The following loading combination is used for calculate the weight of structures according to ASCE7-10.

DL+0.2 LL (Eq 3.5)

3.7 Design Load Combinations

Different load are used to design for our experiments and is shown in Table 3.8.

Table 3.8: Load design combinations according to AISC360-10

No	Load combination
1	DL
2	DL+LL
3	$1.1DL+0.75LL+0.80E_x$
4	$1.1DL+0.75LL-0.8E_x$
5	$1.1DL+0.75LL+0.8E_y$
6	$1.1DL+0.75LL+0.8E_y$
7	$1.1DL+1.07E_x$
8	$1.14DL-1.07E_x$
9	$1.1DL+1.07E_y$
10	$1.1DL-1.07E_y$
11	$0.75DL+1.07E_x$
12	$0.75DL-1.07E_x$
13	$0.75DL+1.07E_y$
14	$0.75DL-1.07E_y$

3.8 Analysis

3.8.1 Time History Analysis

3.8.1.1 Scaling Earthquake Records Procedure

In this thesis earthquake accelerogram of Elcentro, Northridge, and Loma Prieta was used in both perpendicular directions. For this, raw accelerogram was first processed and then base line modification and modifying suitable frequency line was done. Seismo signal software (http://www.seismosoft.com) was used for modifying accelerogram.

To determine scale coefficient for accelerogram, first of all accelerogram were scaled in maximum. This means that in two directions of perpendicular pair accelerogram, maximum acceleration of the direction where PGA is the higher is equal to gravitated acceleration and the other direction is also multiplied with the above mentioned scale. Then the acceleration response spectrum of each pair scaled accelerogram is illustrated considering the 5 percent damping. Moreover response spectrums of each paired accelerogram were combined with each other with the usage of Square Root of Sum of Squares (SRSS) method. Since three accelerograms are used, then the average of three spectrums is obtained. For the purpose of linear analysis Linear Direct Integration and for nonlinear analysis, Nonlinear Direct Integration methods were used. The method that was considered both for linear and nonlinear analyses are called new mark. For Gamma and Beta coefficient 0.5 and 0.25 were considered, respectively. Damping for 5% first and second modules was considered.

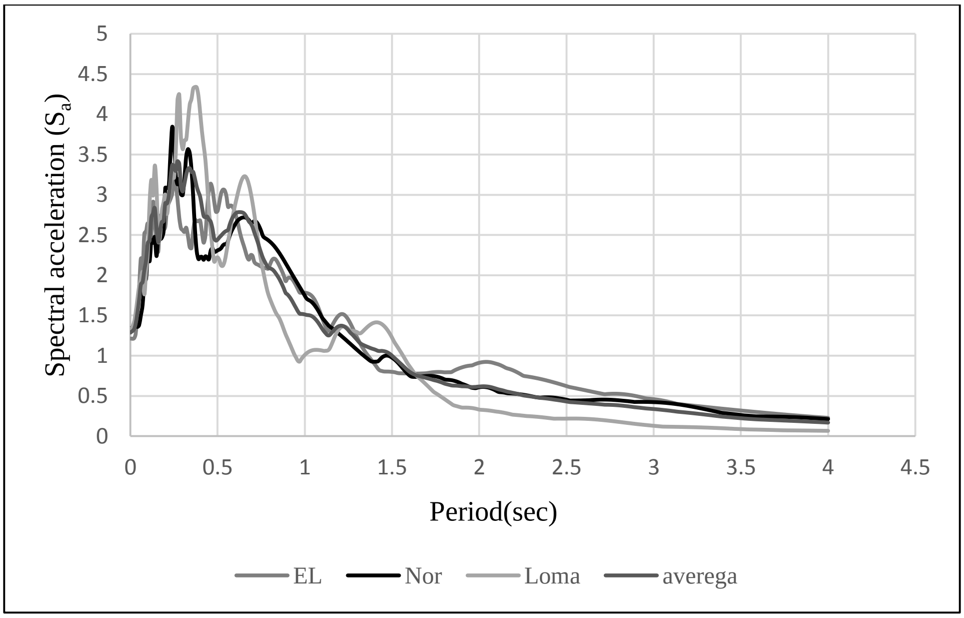


Figure 3.7: Scaling of accelerogram

3.8.1.2 Definition of Plastic Joints

To change a model to a nonlinear one, plastic joints should be defined. Plastic joints are calculated from table ASCE 41-13 and they were shown in Table 3.9 below.

Table 3.9: Plastic joints

Element type	Table from AISC 360-10	Joint type	Distances
Beam	Table 9-6 Steel Beams- Flexure	M_3	0.05 and 0.95 of element
Column	Table 9-6 Steel Columns- Flexure	$P\text{-}M_2\text{-}M_3$	0.05 and 0.95 of element
Bracing	Table 9-7 Steel Brace- Axial	P	In 0.5 from element

3.8.2 Pushover Analysis

3.8.2.1 Definition

Pushover analysis can be used in the structural design of new buildings and refurbishment of existing structures based on structural behavior under lateral loads. Internal forces in structural members increase with gradual increase in displacements. This continuous until forces exceed yield forces in some of the structural joints and hence plastic hinges form at joints. According to the operating level chosen for the structure, it should be able to withstand a pre-determined lateral displacement without any change in Force-Displacement diagram. For instance, assuming that life safety (LS) is selected for a structure statically nonlinear pushover analysis should be carried out for this structure until the capacity diagram is obtained (Fig. 3.8) Capacity diagram is base shear versus lateral roof displacement. During operation until a limiting lateral displacement is reached none of the structural elements shouldn't have displacements more than LS limited. If any element has force or stresses more than the limiting amount then these elements should be reinforced or strengthened.

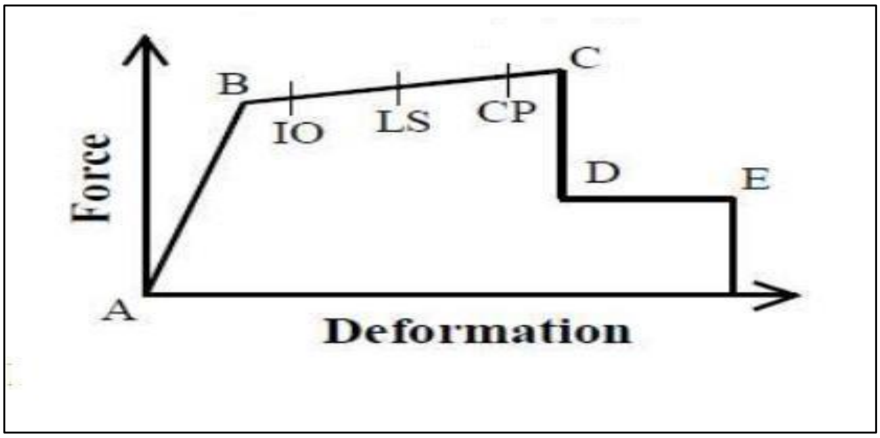


Figure 3.8: Pushover analysis

3.8.2.2 Nonlinear Static Pushover Analysis

Nonlinear static pushover method has an acceptable ability for the estimation of nonlinear behavior of structures. When this method is applied the required displacements can be determined with high accuracy. In this method in addition to the static gravity loads there is lateral loads that gradually increased. Nonlinear statically analysis is used for many applications including:

- Accuracy of new structures
- Earthquake resistance check and strengthening of existing structures
- Design based on function

3.8.2.3 Pushover Analysis Procedure in ETABS Software

ETABS software was used to perform the nonlinear analysis.

Gravity loads for structural elements should be considered according to code (ASCE7-10) and it is given below.

Gravity1=1.1(DL+0.2LL) (Eq 3.6)

For pushover analysis lateral load distribution was also defined.

3.9 Lateral Load Model

Lateral load distribution in the structure height in earthquake vibration is very complicated. From design point, lateral load distribution should be selected in a way that makes the most critical situation. If the structure is in linear elastic behavior, lateral load distribution depends on many parameters including earthquake vibration latitude, frequencies and mode shapes. If the structure has nonlinear behavior, in addition to the above mentioned parameters the lateral load distribution will also depend on local yielding of structural elements and for this reason is much more complicated. In this study, for static pushover analysis lateral load distribution was assumed to be in two positive directions of X and Y (distribution corresponding to E_x and E_y). Roof center was determined as a point for target displacement and the target value was assumed to be equal to 0.02 of structure height.

Chapter 4

RESULTS AND DISCUSSION

4.1 Introduction

This chapter provides a summary of the results produced using different ETABS Models. Buildings of 4, 8, and 12 stories, and bays of 3x3 and 3x5 were included in the analyses. Buildings were designed and analyzed using linear dynamic, nonlinear time history and static pushover methods. Seven primary aspects were selected for comparison as listed below.

1) Comparing interstory drifts and displacements of models

2) Comparing the linear dynamic and nonlinear dynamic time history behavior of EBF and CBF by using three accelerograms in x- and y- directions for selected models

3) Comparing base shears obtained from the above mentioned analysis

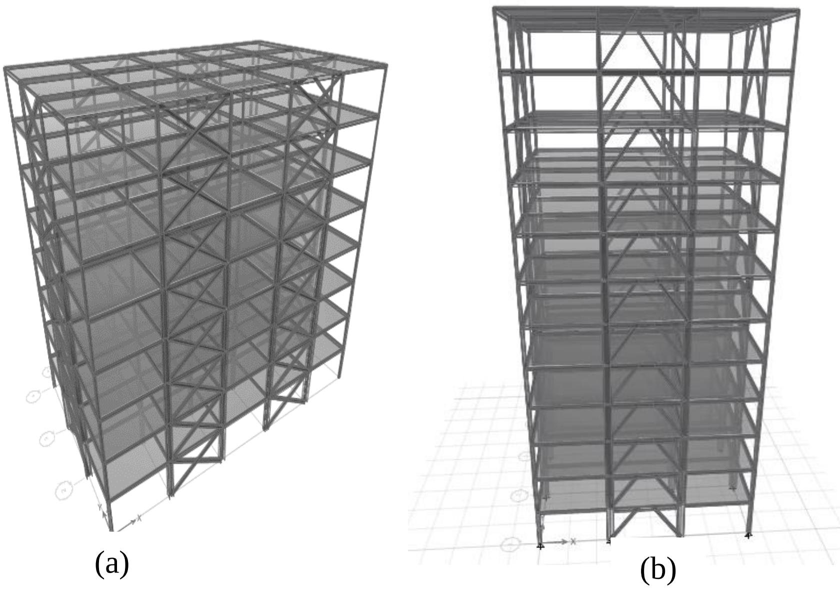
4) Investigating the behavior of EBF and CBF by using nonlinear static Pushover analysis

5) Target displacement

6) Deformation for structure under pushover analyses

4.2 Structures Overview

Structure with EBF and CBF were designed by using ETABS software (Fig. 4.1).



(a) (b)

Figure 4.1 Structure overview for (a) CBF and (b) EBF

4.3 Results of Analysis

4.3.1 Comparing Interstory Displacements of Models

Story level displacements in x- and y- directions for linear and nonlinear analysis are provided in Tables 4.2 to 4.4. According to standards (AISC360-10) displacements obtained from linear analyses are multiplied by c_d to get the real displacements of structures.c_d is the lateral relative displacement coefficient resulting from linear analysis and it is to lateral displacement. The values of c_d are 5 and 4 for CBF and EBF, respectively. Tables 4.2 to 4.4 show the values of linear displacements multiplied by c_d and the values of nonlinear displacements for the same models.

Table 4.1: Linear and Nonlinear displacements for 12 story models

Displacement in x-direction (m)								
Story	3x3 EBF		3x3x CBF		3x5 EBF		3x5 CBF	
	Linear 3x3 EBF	Nonlinear 3x3 EBF	Linear 3x3 CBF	Nonlinear 3x3 CBF	Linear 3x5 EBF	Nonlinear 3x5 EBF	Linear 3x5 CBF	Nonlinear 3x5 CBF
12	0.339	0.359	0.382	0.269	0.522	0.372	0.315	0.239
11	0.319	0.342	0.338	0.242	0.493	0.357	0.280	0.216
10	0.291	0.319	0.294	0.212	0.456	0.337	0.244	0.191
9	0260	0.293	0.251	0.183	0.411	0.313	0.210	0.167
8	0.227	0.264	0.210	0.155	0.363	0.288	0.176	0.142
7	0.195	0.236	0.171	0.128	0.315	0.262	0.144	0.118
6	0.162	0.207	0.135	0.102	0.266	0.233	0.114	0.096
5	0.130	0.174	0.102	0.079	0.216	0.200	0.087	0.075
4	0.100	0.140	0.073	0.057	0.168	0.163	0.062	0.056
3	0.072	0.105	0.048	0.038	0.122	0.123	0.041	0.038
2	0.046	0.070	0.028	0.023	0.080	0.083	0.024	0.024
1	0.023	0.035	0.012	0.011	0.039	0.042	0.011	0.012

Displacements in y-direction (m)								
Story	3x3 EBF		3x3x CBF		3x5 EBF		3x5 CBF	
	Linear 3x3 EBF	Nonlinear 3x3 EBF	Linear 3x3CBF	Nonlinear 3x3 CBF	Linear 3x5 EBF	Nonlinear 3x5 EBF	Linear 3x5 CBF	Nonlinear 3x5 CBF
12	0.325	0.367	0.384	0.272	0.640	0.364	0.483	0.302
11	0.305	0.350	0.340	0.244	0.597	0.346	0.430	0.271
10	0.279	0.328	0.295	0.214	0.544	0.325	0.376	0.239
9	0.249	0.303	0.253	0.185	0.485	0.301	0.324	0.208
8	0.216	0.276	0.211	0.156	0.420	0.275	0.273	0.178
7	0.186	0.250	0.172	0.129	0.357	0.249	0.223	0.148
6	0.154	0.221	0.135	0.103	0.296	0.224	0.177	0.120
5	0.123	0.187	0.103	0.079	0.238	0.199	0.134	0.093
4	0.094	0.150	0.073	0.057	0.183	0.170	0.096	0.068
3	0.067	0.113	0.048	0.038	0.132	0.134	0.063	0.046
2	0.043	0.076	0.027	0.023	0.083	0.092	0.036	0.027
1	0.021	0.038	0.012	0.010	0.040	0.047	0.015	0.012

Table 4.2: Linear and Nonlinear displacements for 8 story models

Displacement in x-direction (m)								
Story	3x3 EBF		3x3x CBF		3x5 EBF		3x5 CBF	
	Linear 3x3 EBF	Nonlinear 3x3 EBF	Linear 3x3CBF	Nonlinear 3x3 CBF	Linear 3x5 EBF	Nonlinear 3x5 EBF	Linear 3x5 CBF	Nonlinear 3x5 CBF
8	0.161	0.237	0.170	0.166	0.202	0.248	0.163	0.153
7	0.149	0.224	0.144	0.144	0.186	0.234	0.138	0.134
6	0.130	0.203	0.116	0.119	0.163	0.213	0.112	0.111
5	0.108	0.178	0.090	0.094	0.135	0.188	0.088	0.089
4	0.085	0.150	0.065	0.071	0.106	0.158	0.064	0.067
3	0.063	0.118	0.044	0.049	0.079	0.125	0.043	0.046
2	0.042	0.081	0.026	0.030	0.052	0.087	0.025	0.028
1	0.021	0.041	0.011	0.014	0.026	0.044	0.011	0.013

Displacement in y-direction (m)								
Story	3x3 EBF		3x3x CBF		3x5 EBF		3x5 CBF	
	Linear 3x3EBF	Nonlinear 3x3 EBF	Linear 3x3CBF	Nonlinear 3x3 CBF	Linear 3x5EBF	Nonlinear 3x5 EBF	Linear 3x5CBF	Nonlinear 3x5 CBF
8	0.292	0.221	0.205	0.149	0.292	0.328	0.234	0.196
7	0.269	0.207	0.172	0.128	0.269	0.316	0.195	0.169
6	0.236	0.186	0.138	0.104	0.236	0.298	0.157	0.139
5	0.196	0.161	0.106	0.082	0.196	0.274	0.120	0.109
4	0.153	0.134	0.076	0.060	0.153	0.237	0.087	0.081
3	0.113	0.108	0.050	0.041	0.113	0.189	0.058	0.056
2	0.075	0.076	0.028	0.024	0.075	0.131	0.033	0.033
1	0.037	0.039	0.012	0.011	0.037	0.067	0.014	0.015

Table 4.3: Linear and Nonlinear displacements for 4 story models

Displacement in x-direction (m)

Story	3x3 EBF		3x3x CBF		3x5 EBF		3x5 CBF	
	Linear 3x3EBF	Nonlinear 3x3 EBF	Linear 3x3CBF	Nonlinear 3x3 CBF	Linear 3x5EBF	Nonlinear 3x5 EBF	Linear 3x5CBF	Nonlinear 3x5 CBF
4	0.099	0.143	0.057	0.087	0.118	0.158	0.051	0.082
3	0.080	0.126	0.041	0.067	0.093	0.135	0.037	0.064
2	0.052	0.099	0.025	0.042	0.057	0.099	0.022	0.042
1	0.026	0.055	0.011	0.020	0.028	0.054	0.010	0.019

Displacement in y-direction (m)

Story	3x3 EBF		3x3x CBF		3x5 EBF		3x5 CBF	
	Linear 3x3EBF	Nonlinear 3x3 EBF	Linear 3x3CBF	Nonlinear 3x3 CBF	Linear 3x5EBF	Nonlinear 3x5 EBF	Linear 3x5CBF	Nonlinear 3x5 CBF
4	0.099	0.142	0.057	0.087	0.147	0.168	0.076	0.107
3	0.080	0.126	0.041	0.067	0.120	0.154	0.054	0.082
2	0.052	0.102	0.025	0.042	0.078	0.131	0.032	0.054
1	0.026	0.061	0.011	0.020	0.039	0.075	0.013	0.020

Tables 4.5 to 4.7 show the ratio of nonlinear to linear displacements for all structures. When this ratio approaches to 1 the value of c_d for both CBF and EBC become more accurate. The results show that, particularly for 4-story buildings, this ratio was exceeded 1.0. It is also known that this ratio depends on the type of bracing and the number of openings in the perimeter of building. Results indicate that in nonlinear analysis displacements are generally larger than those of the linear analysis. In 8-story buildings the ratio is generally close to 1.0, whereas in a 12-story building it is lower than 1.0. In addition to the type of brace, according to standards (AISC360-10), c_d also depends on the number of floors and on the number of openings.

Table 4.4: Ratios of Nonlinear to linear displacements for 12 story models

Story	Ratios of Nonlinear to linear displacements							
	3x3 EBF		3x3 CBF		3x5 EBF		3x5 CBF	
	x-direction	y-direction	x-direction	y-direction	x-direction	y-direction	x-direction	y-direction
12	1.060	1.130	0.705	0.708	0.713	0.569	0.758	0.625
11	1.073	1.149	0.715	0.718	0.723	0.580	0.772	0.630
10	1.095	1.177	0.721	0.724	0.739	0.597	0.784	0.635
9	1.125	1.218	0.729	0.732	0.762	0.621	0.796	0.643
8	1.165	1.275	0.736	0.739	0.793	0.655	0.808	0.653
7	1.215	1.345	0.745	0.749	0.830	0.698	0.823	0.664
6	1.274	1.433	0.755	0.759	0.876	0.759	0.841	0.676
5	1.338	1.522	0.768	0.772	0.924	0.838	0.864	0.690
4	1.400	1.603	0.782	0.785	0.970	0.925	0.892	0.707
3	1.456	1.679	0.799	0.799	1.009	1.015	0.932	0.728
2	1.511	1.758	0.828	0.829	1.043	1.104	0.992	0.757
1	1.549	1.824	0.870	0.877	1.068	1.170	1.083	0.797

Table 4.5: Ratios of Nonlinear to linear displacements for 8 story models

Story	Ratios of Nonlinear to linear displacements							
	3x3 EBF		3x3 CBF		3x5 EBF		3x5 CBF	
	x-direction	y-direction	x-direction	y-direction	x-direction	y-direction	x-direction	y-direction
8	1.468	0.759	0.978	0.726	1.233	1.125	0.942	0.838
7	1.501	0.771	1.005	0.743	1.259	1.174	0.969	0.863
6	1.559	0.789	1.027	0.754	1.309	1.261	0.989	0.884
5	1.648	0.824	1.054	0.771	1.386	1.400	1.011	0.907
4	1.762	0.874	1.083	0.787	1.487	1.547	1.033	0.928
3	1.863	0.950	1.120	0.810	1.582	1.664	1.063	0.957
2	1.943	1.015	1.166	0.836	1.661	1.756	1.102	0.988
1	2.002	1.061	1.231	0.890	1.716	1.822	1.169	1.044

Table 4.6: Ratios of Nonlinear to linear displacements for 4 story models

Story	Ratios of Nonlinear to linear displacements							
	3x3 EBF		3x3 CBF		3x5 EBF		3x5 CBF	
	x-direction	y-direction	x-direction	y-direction	x-direction	y-direction	x-direction	y-direction
4	1.446	1.433	1.518	1.524	1.341	1.139	1.614	1.412
3	1.578	1.578	1.608	1.615	1.454	1.286	1.725	1.516
2	1.896	1.969	1.693	1.700	1.747	1.678	1.848	1.675
1	2.115	2.349	1.827	1.836	1.905	1.909	1.985	1.524

Figures 4.2 to 4.7 show the displacement of all structures both in x- and y- directions, with results derived from both linear and nonlinear time history analysis. As the number of floors increase, so do the displacements in both CBF and EBF. Results are in line with the results of Moghaddam, H and Hajirasouliha, I (2006) and Güneyisi, M. and Muhyaddin. G (2012). In 4 story models the amount of displacement (in both directions) resulting from the nonlinear analysis is larger than that of the linear analysis, with EBF displacements being more than the CBF. In 8 story models the displacements related to nonlinear analysis are generally marginally higher and in some cases almost equal to the linear ones. In 12 story models nonlinear displacements are less than the linear ones in both directions in all structures.

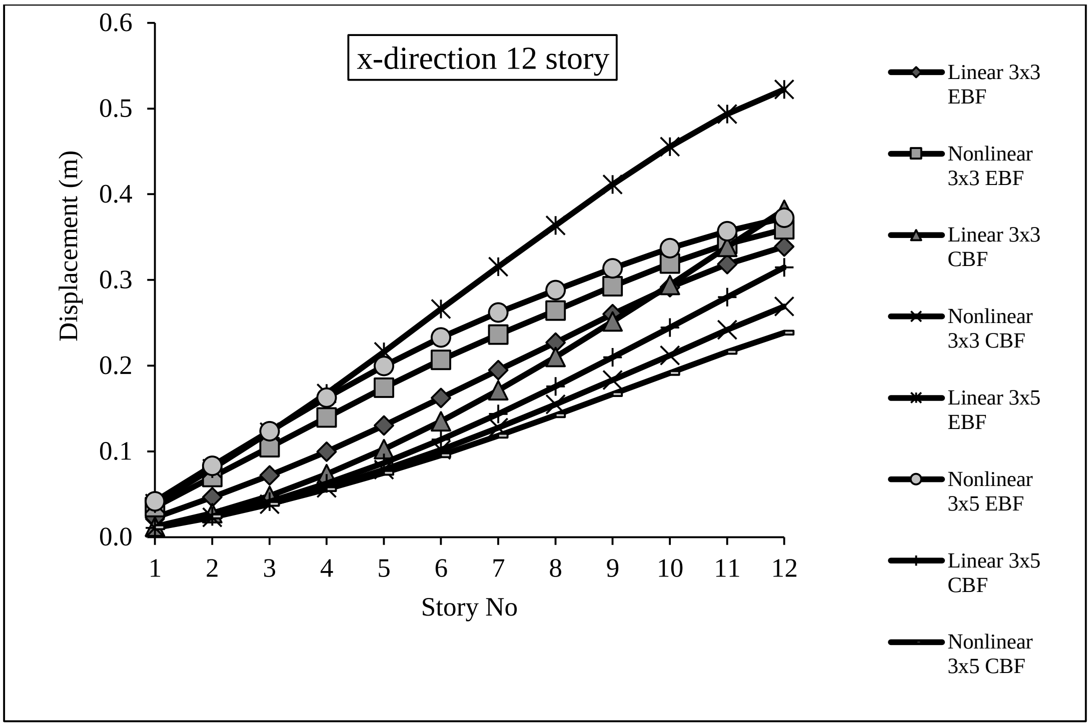


Figure 4.2: Displacement of 12 story structures in x-direction using linear and nonlinear dynamic analysis

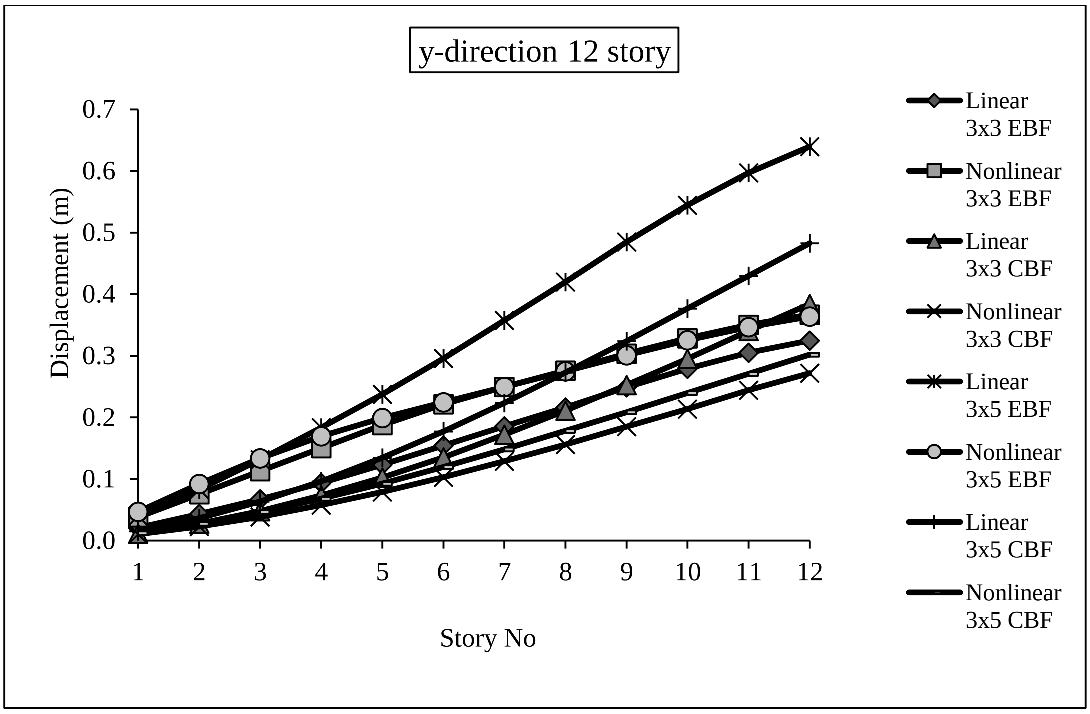


Figure 4.3: Displacement of 12 story structures in y-direction using linear and nonlinear dynamic analysis

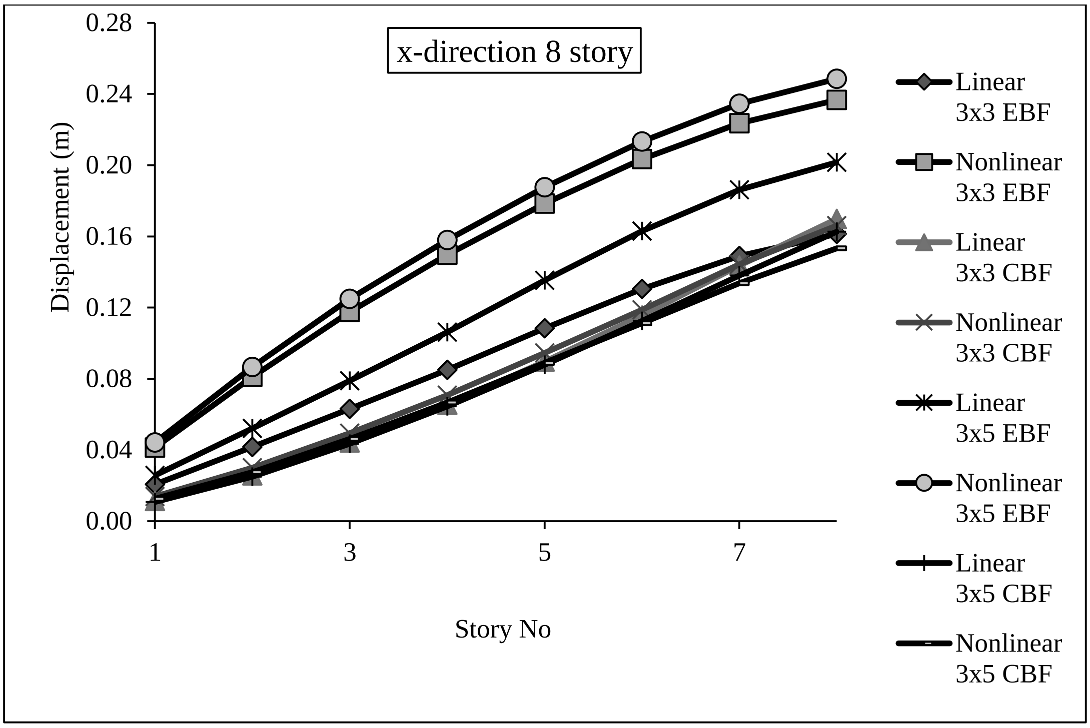


Figure 4.4: Displacement of 8 story structures in x-direction using linear and nonlinear dynamic analysis

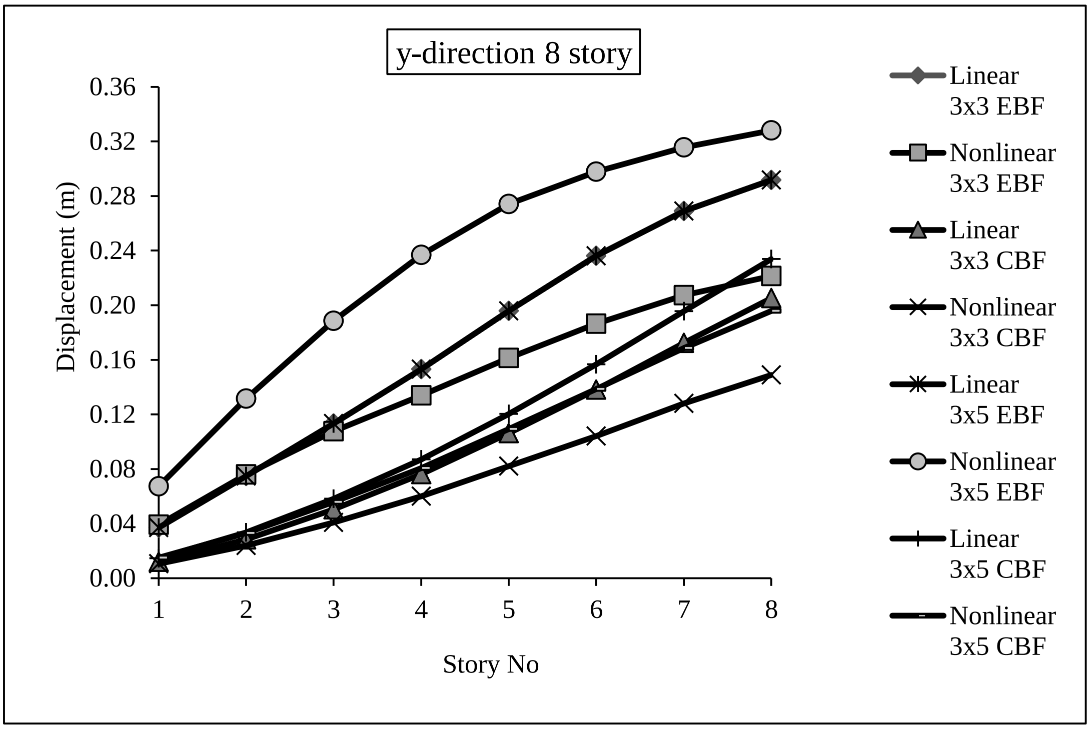


Figure 4.5: Displacement of 8 story structures in y-direction using linear and nonlinear dynamic analysis

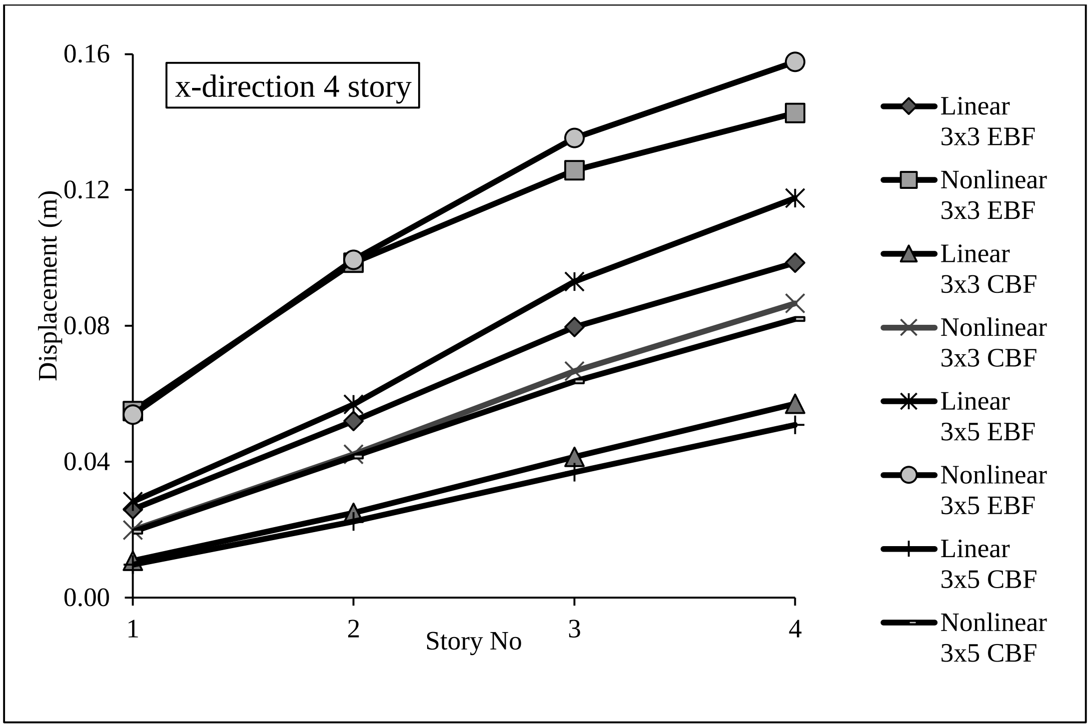


Figure 4.6: Displacement of 4 story structures in x-direction using linear and nonlinear dynamic analysis

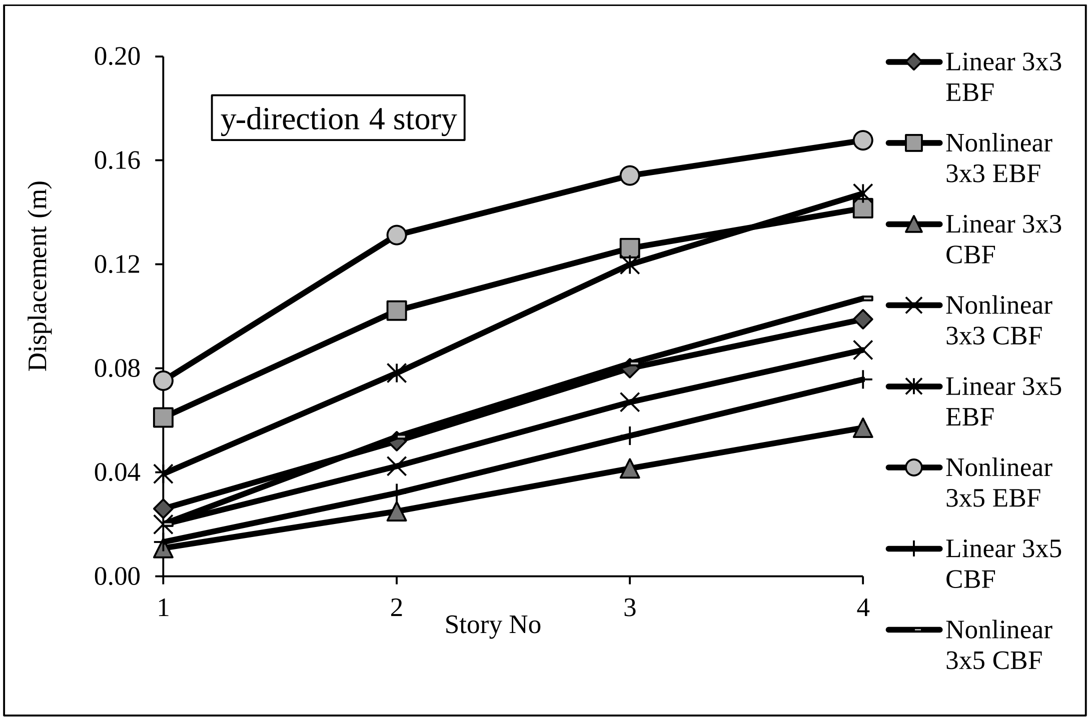


Figure 4.7: Displacement of 4 story CBF and EBF in y-direction using linear and nonlinear dynamic analysis

4.3.2 Comparing the Drift of Structures

Figures 4.8 to 4.13 show drift for all structures both in x- and y-directions using linear and nonlinear dynamic analysis. According to AISC360-10, the allowed drift relative for floor are given in Table 4.8.

Table 4.7: Max allowable drift according to AISC 360-10

Story limitations	Max allowed drift
Buildings < 5-story	0.025
Others	0.02

Results indicate that the value of the drift in all structures is lower than the maximum allowed value, with the exception of the drift for the first two stories and 1st story for 8 and 4 story buildings, respectively (Figures 4.11 and 4.13). These results are in proportionate with results of Güneyisi (2012) and Zasiah (2013). They also had found that the value of drift for all structures were lower than the maximum allowed value. Drift for these cases were exceeded the allowable value only in the y-direction for EBF Nonlinear with 3x5 bays case.

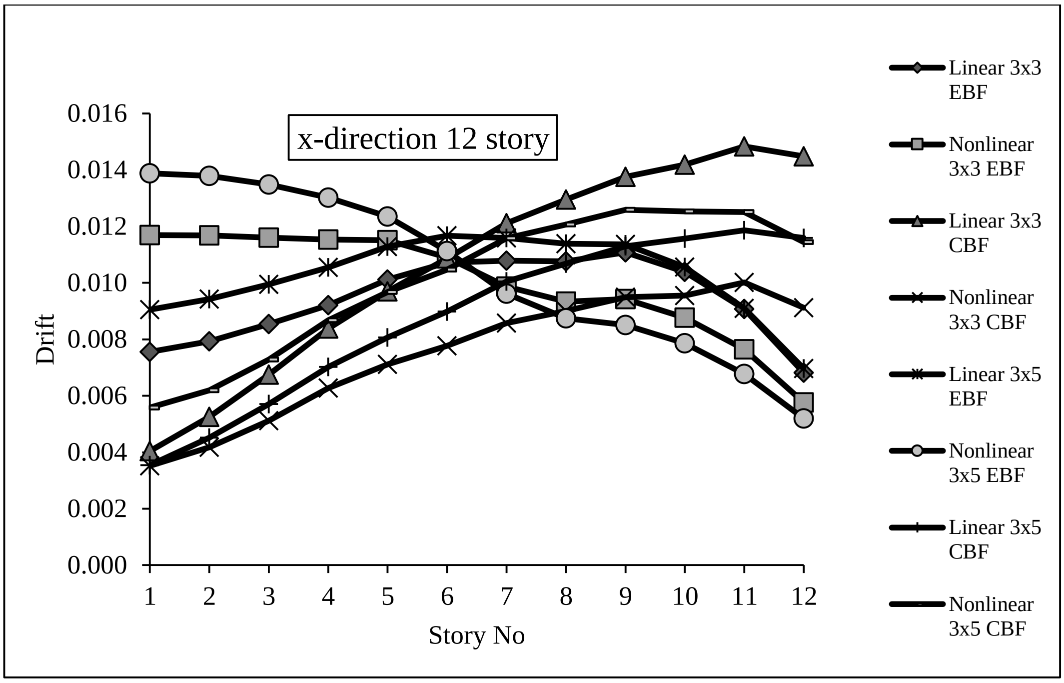


Figure 4.8: Drift of 12 story CBF and EBF in x-direction using linear and nonlinear dynamic analysis

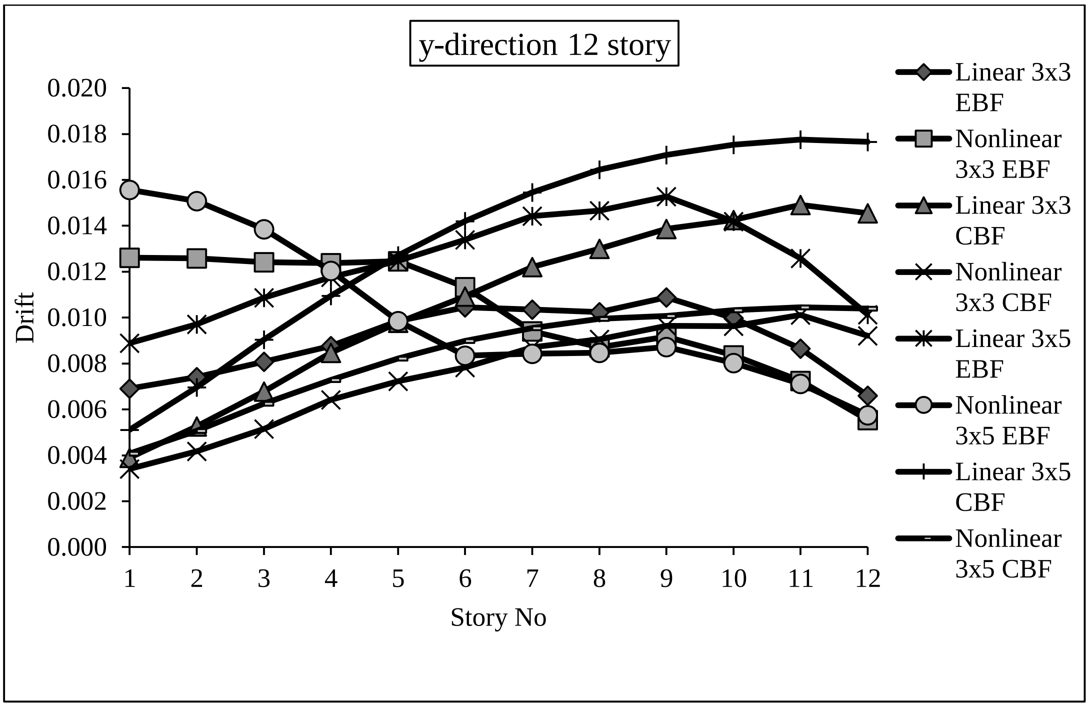


Figure 4.9: Drift of 12 story CBF and EBF in y-direction using linear and nonlinear dynamic analysis

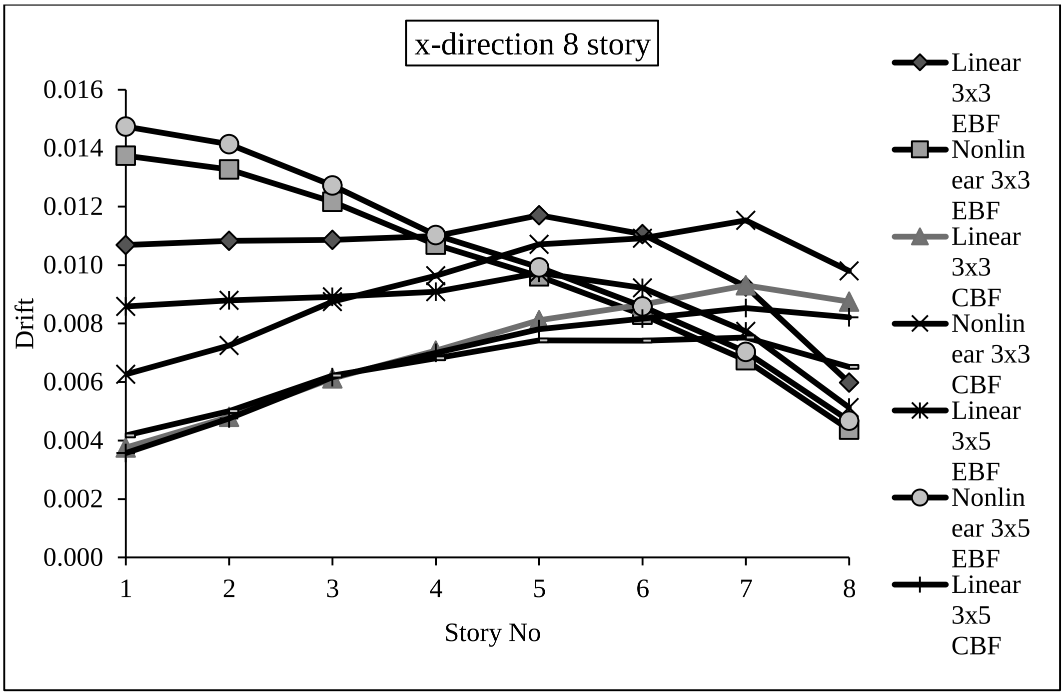


Figure 4.10: Drift of 8 story CBF and EBF in x-direction using linear and nonlinear dynamic analysis

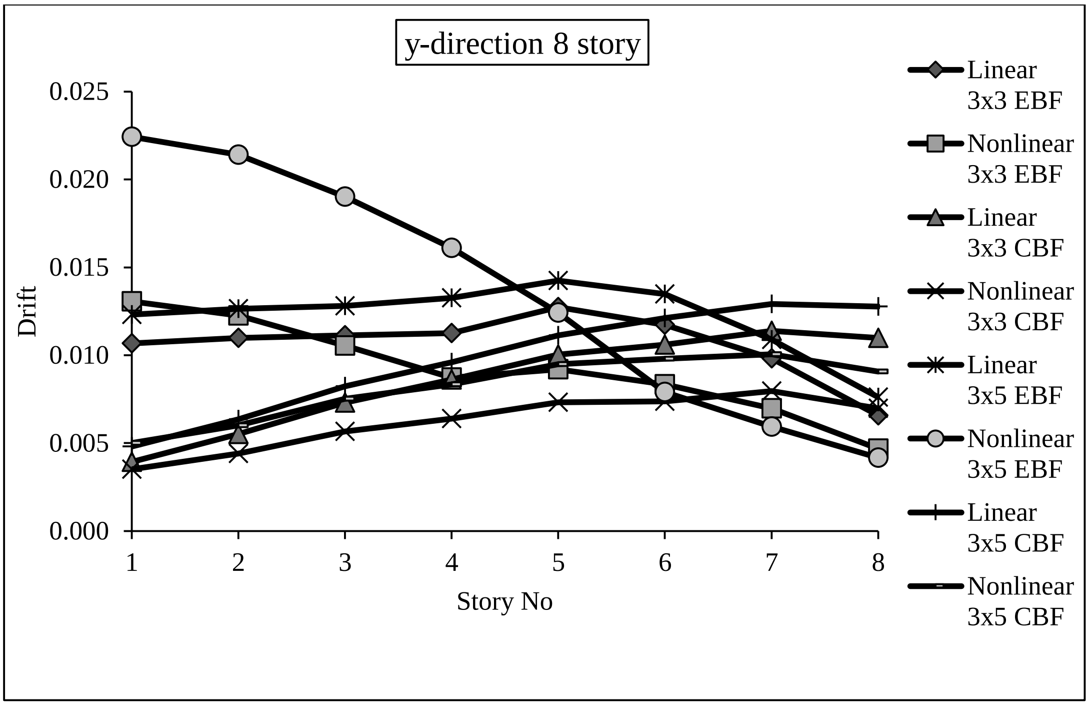


Figure 4.11: Drift of 8 story CBF and EBF in y-direction using linear and nonlinear dynamic analysis

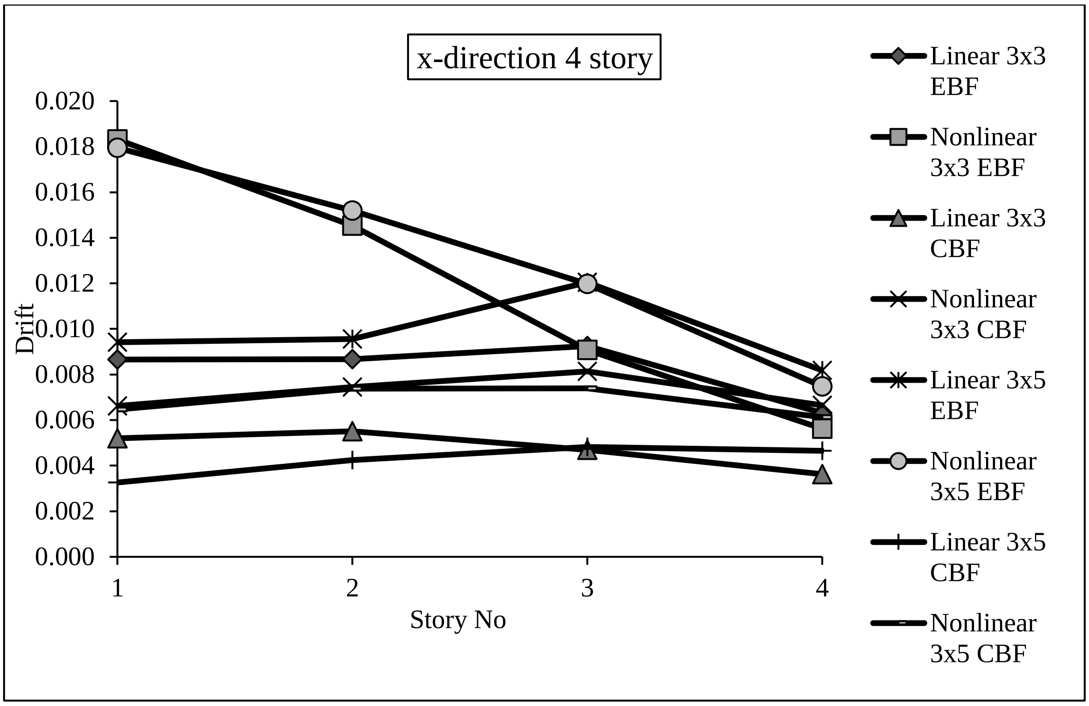


Figure 4.12: Drift of 4 story CBF and EBF in x-direction using linear and nonlinear dynamic analysis

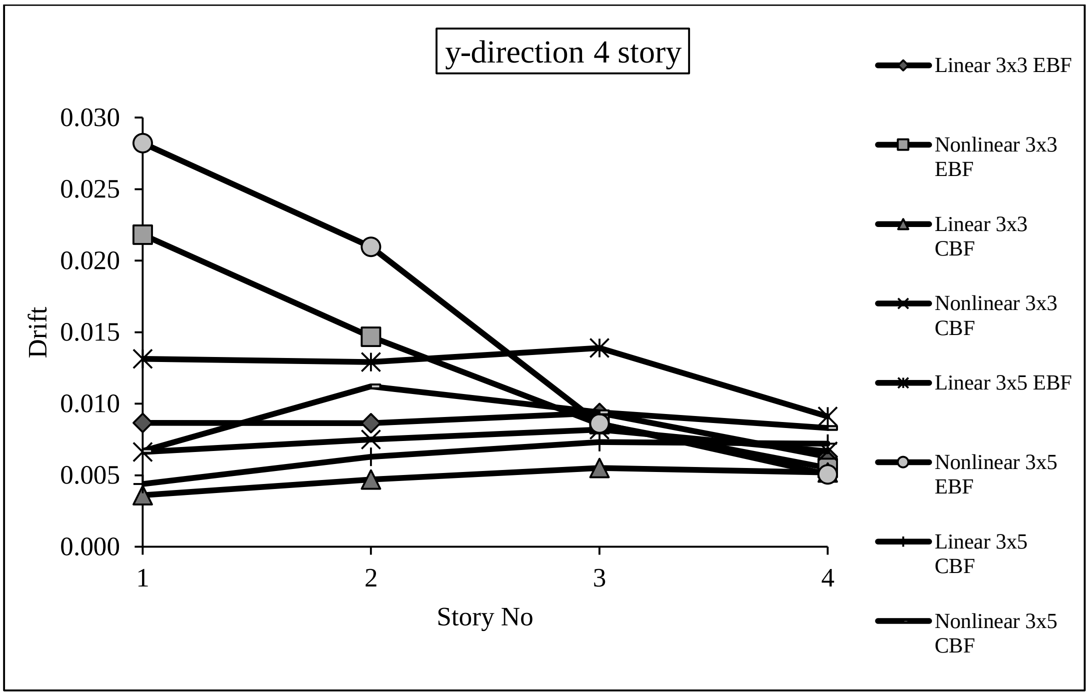


Figure 4.13: Drift of 4 story CBF and EBF in y-direction using linear and nonlinear dynamic analysis

4.3.3 Comparing the Linear Dynamic and Nonlinear Dynamic Time History Behaviour of EBF and CBF by Using Three Accelerograms in X- and Y-Directions for Selected Models

History of base shear plots for linear and nonlinear states for CBF and EBF structures are given in Figures 4.14 and A.1 to A.11 in the appendix. According to the linear and nonlinear results are similar in both models. In non-modified accelerograms nonlinear base shear values are lower than linear ones, which indicate that the structure reached to a nonlinear boundary. Since accelerogram spectrum and modified design spectrum are within a linear boundary in modified accelerograms, it won't enter a nonlinear range and base shears resulted from linear and nonlinear model analysis are the same. However, when using unmodified accelerograms the structure enters in a nonlinear boundary and results vary, with the resulting base shear of nonlinear model becoming lower than linear model.

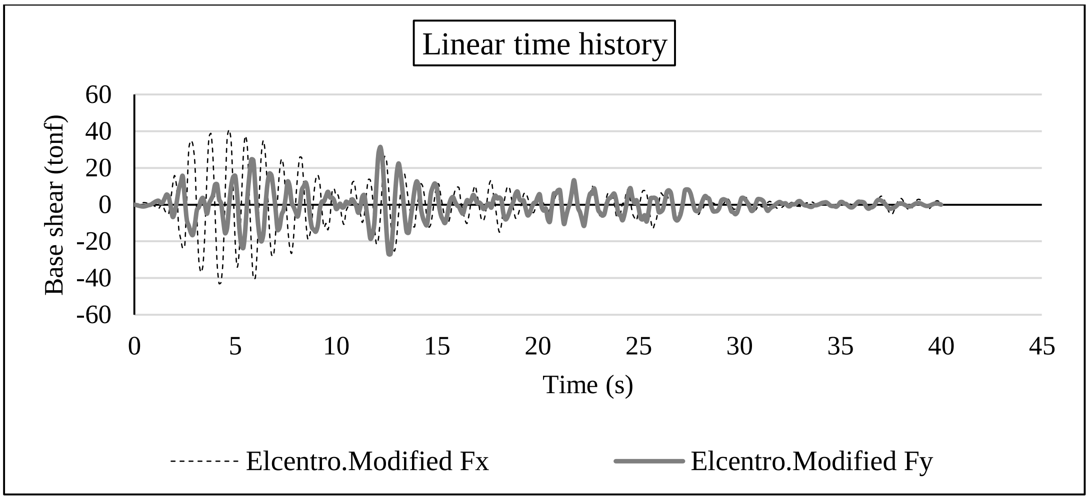


Figure 4.14: History of modified accelerogram base shear of Elcentro earthquake in linear analysis

4.3.4 Comparing Base Shears Obtained From the Above Mentioned Analysis

In linear static analysis, base shear values depend on the type of conveying system, weight of the structure and the location. Equivalent static base shear force (E_X and E_Y) in all EBF structures is lower than that of CBF structures because of the higher behavior coefficient in EBF structures. Increasing the number of floors and bays will result in increased structure weight and higher static shear force. Analysis of history of base shear force depends on mass, period of structure and earthquake PGA; in other words the base shear force is the result of mass multiplied by acceleration. Base shear values of modified accelerograms are similar to equivalent static results. Furthermore linear and nonlinear results are also similar as the structure has not yet entered into the nonlinear range.

When base shear force is considered the unmodified accelerograms are varying considerably in both linear and nonlinear models and the base shear values of the nonlinear model are lower than that of the linear one. Indeed, as base shear of the structure increases it enters the nonlinear range and leads to reduced base shear force compared to the linear model. The ratio of nonlinear shear force to linear base shear in structures is mostly dependent on earthquake PGA and structure period and it is not a fix value. These findings are in line with results of Tremblay. R, Robert. N(2001) and Güneyisi .M, Muhyaddin.G (2012).

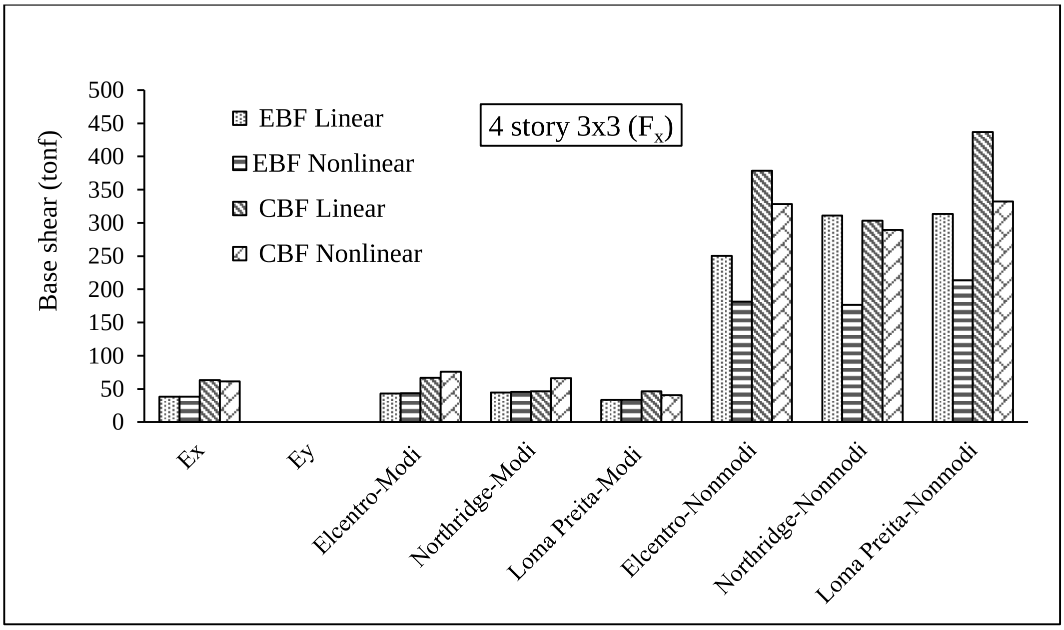


Figure 4.15: Base shear resulting from linear and nonlinear analysis of 4 story structures for 3x3 bay in x-directions

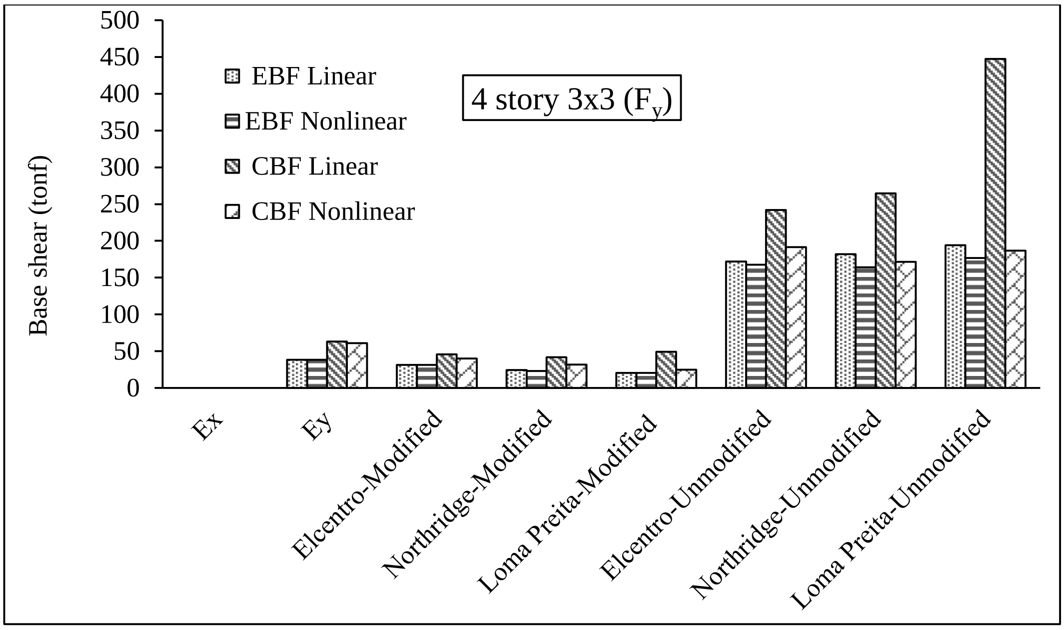


Figure 4.16: Base shear resulting from linear and nonlinear analysis of 4 story structures for 3x3 bay in y-directions

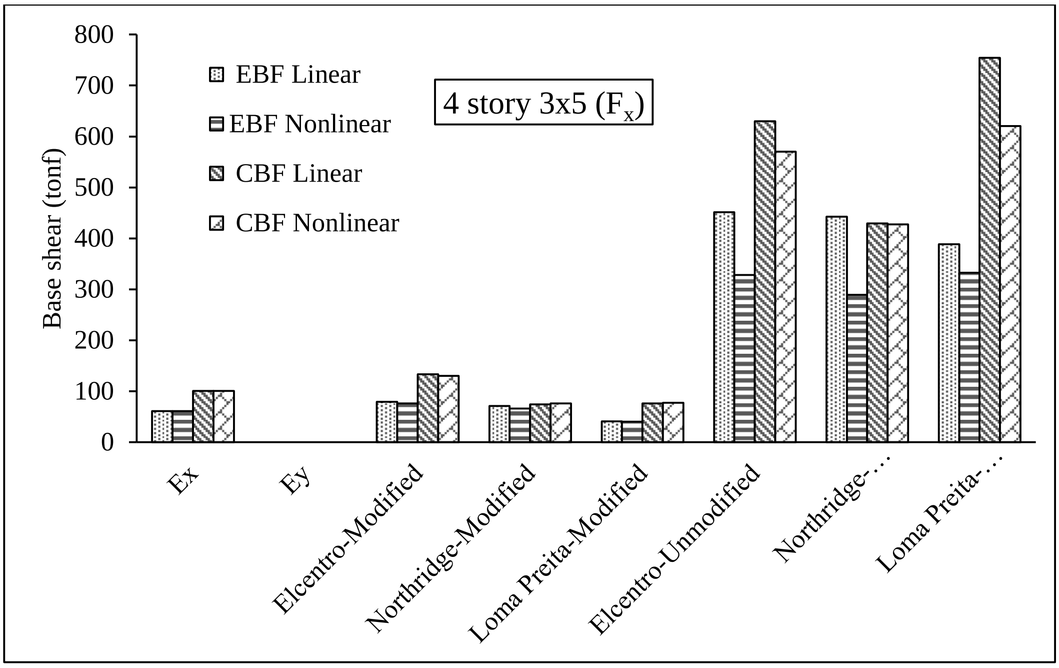


Figure 4.17: Base shear resulting from linear and nonlinear analysis of 4 story structures for 3x5 bay in x-directions

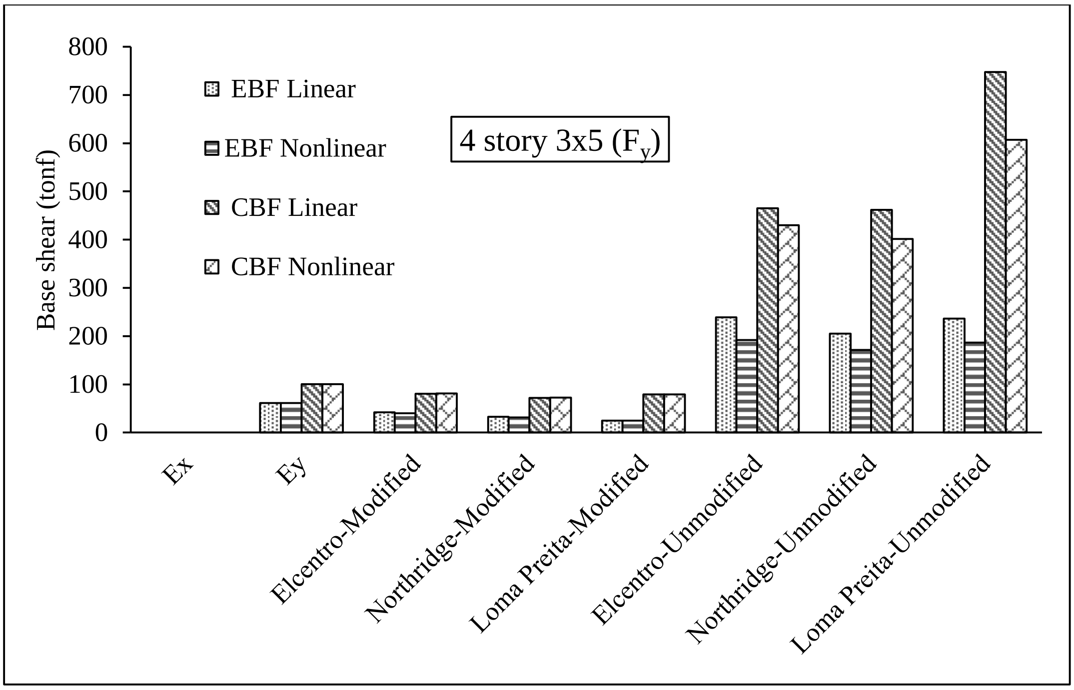


Figure 4.18: Base shear resulting from linear and nonlinear analysis of 4 story structures for 3x5 bay in y-directions

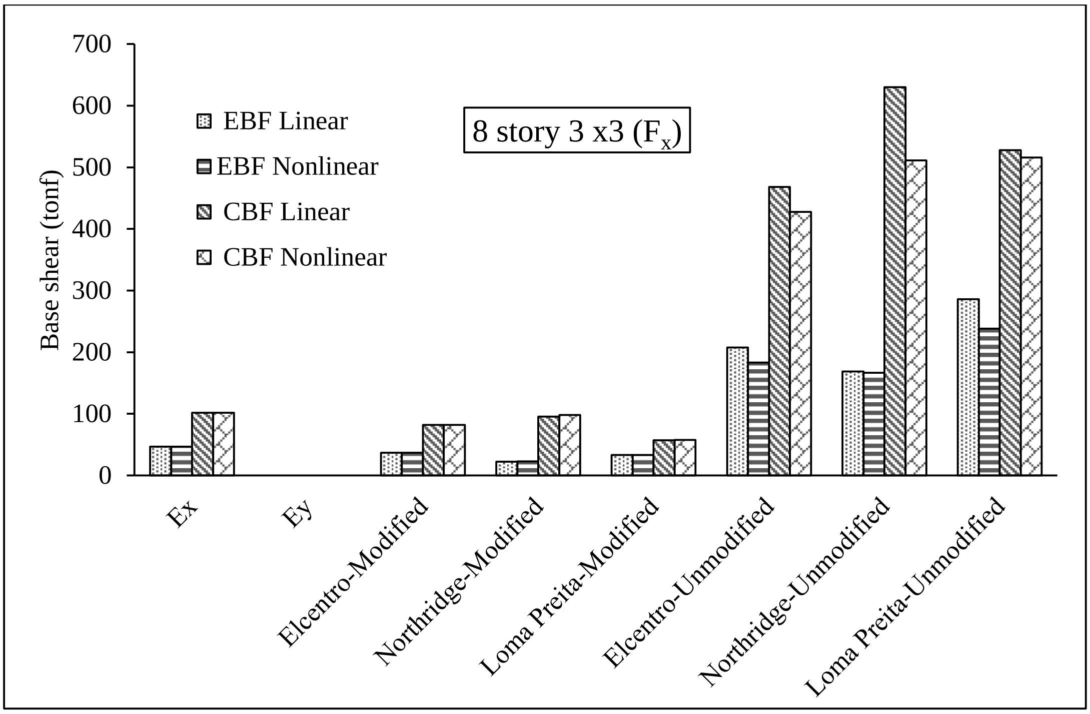


Figure 4.19: Base shear resulting from linear and nonlinear analysis of 8 story structures for 3x3 bay in x-directions

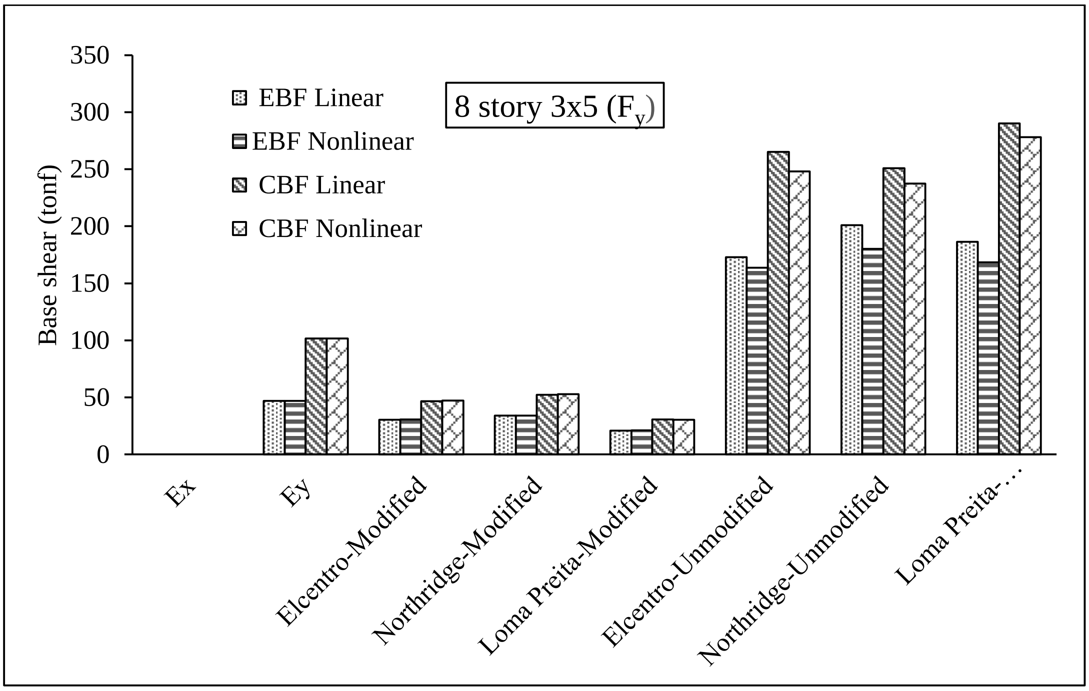


Figure 4.20: Base shear resulting from linear and nonlinear analysis of 8 story structures for 3x3 bay in y-directions

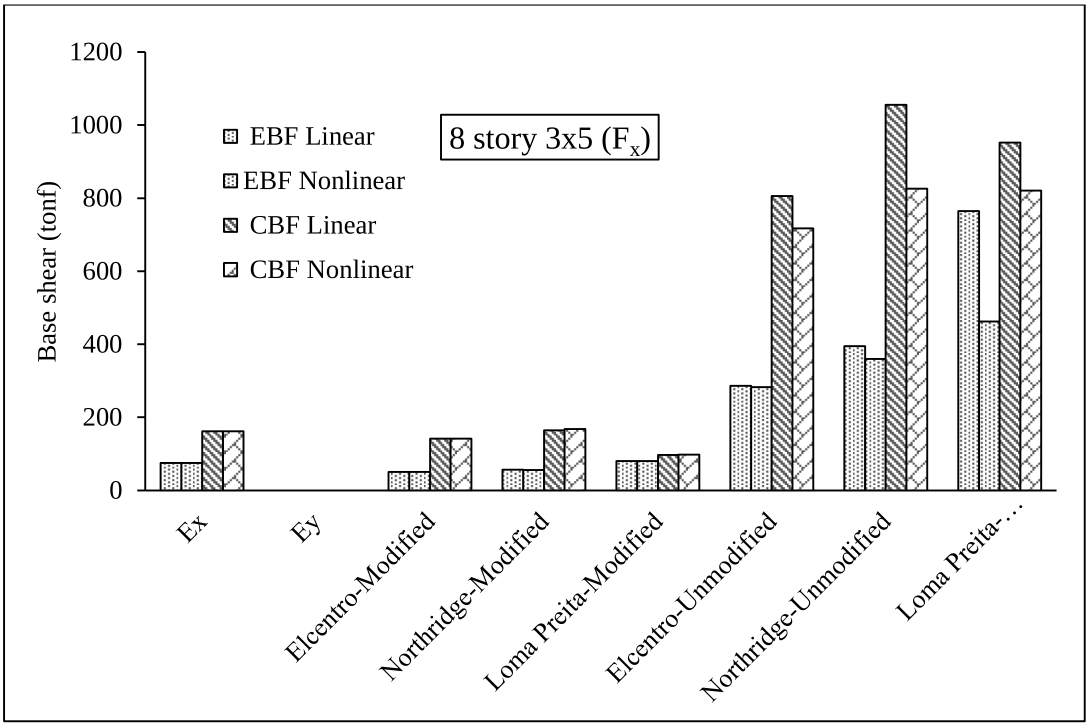


Figure 4.21: Base shear resulting from linear and nonlinear analysis of 8 story structures for 3x5 bay in x-directions

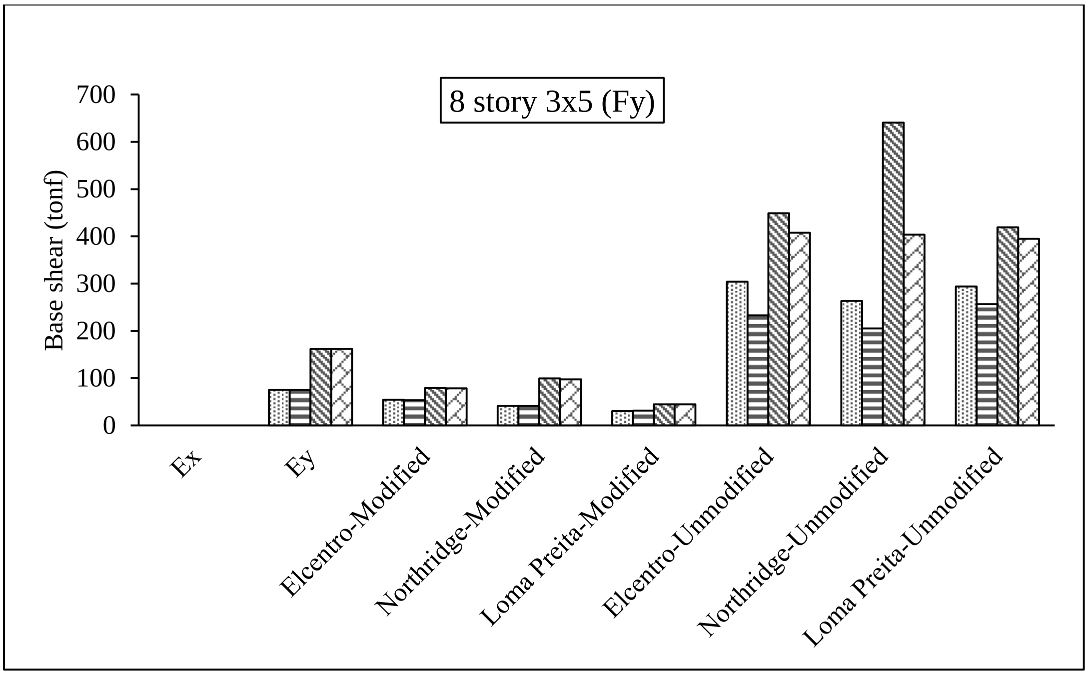


Figure 4.22: Base shear resulting from linear and nonlinear analysis of 8 story structures for 3x5 bay in y-directions

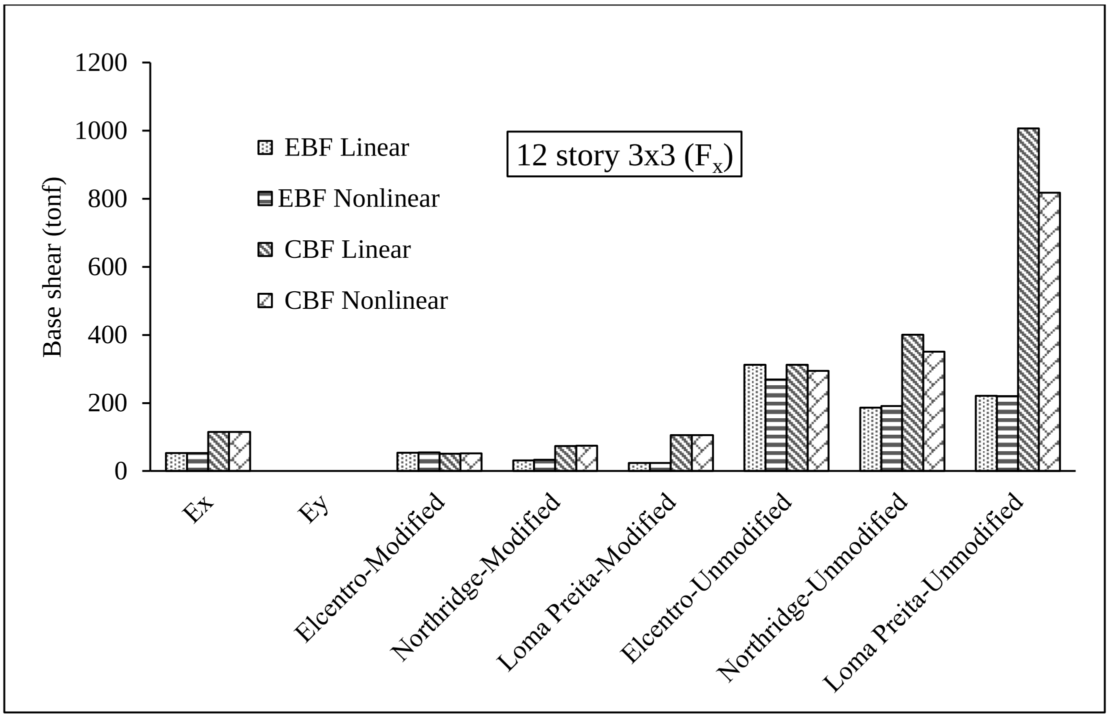


Figure 4.23: Base shear resulting from linear and nonlinear analysis of 12 story structures for 3x3 bay in x-directions

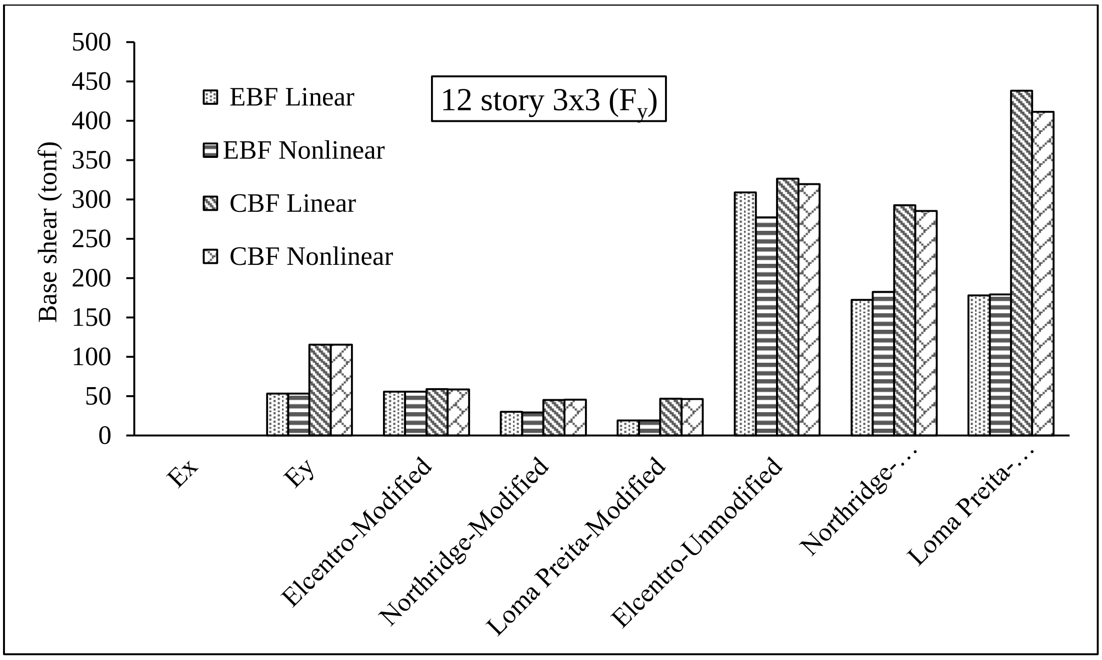


Figure 4.24: Base shear resulting from linear and nonlinear analysis of 12 story structures for 3x3 bay in y-directions

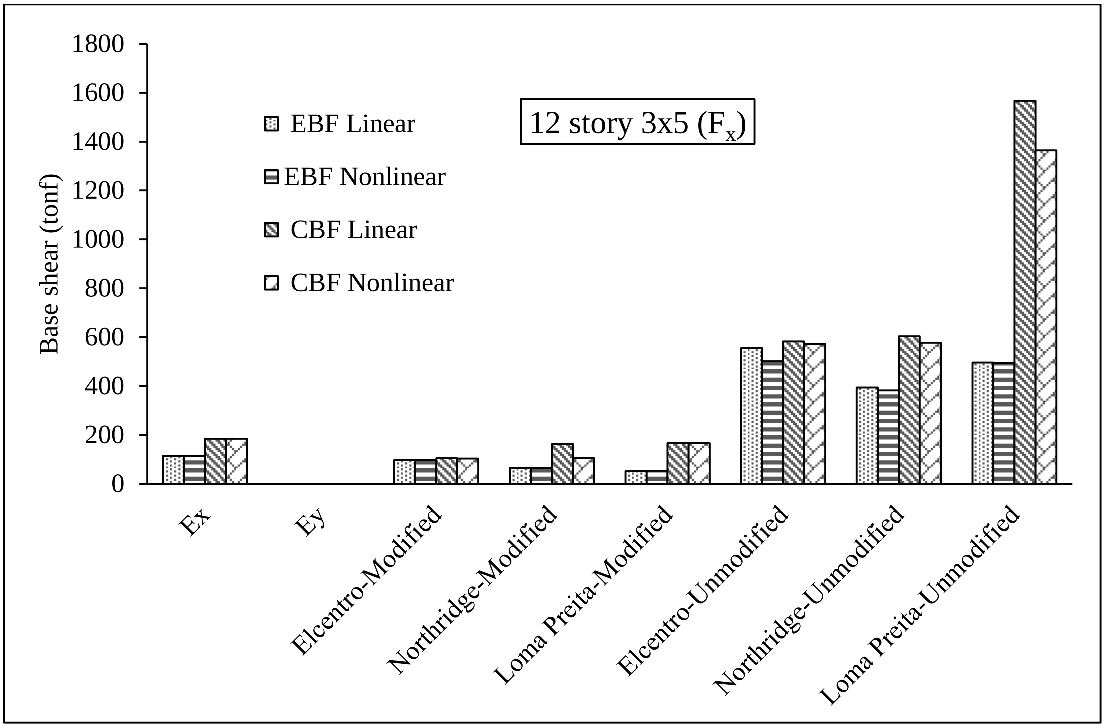


Figure 4.25: Base shear resulting from linear and nonlinear analysis of 12 story structures for 3x5 bay in x-direction

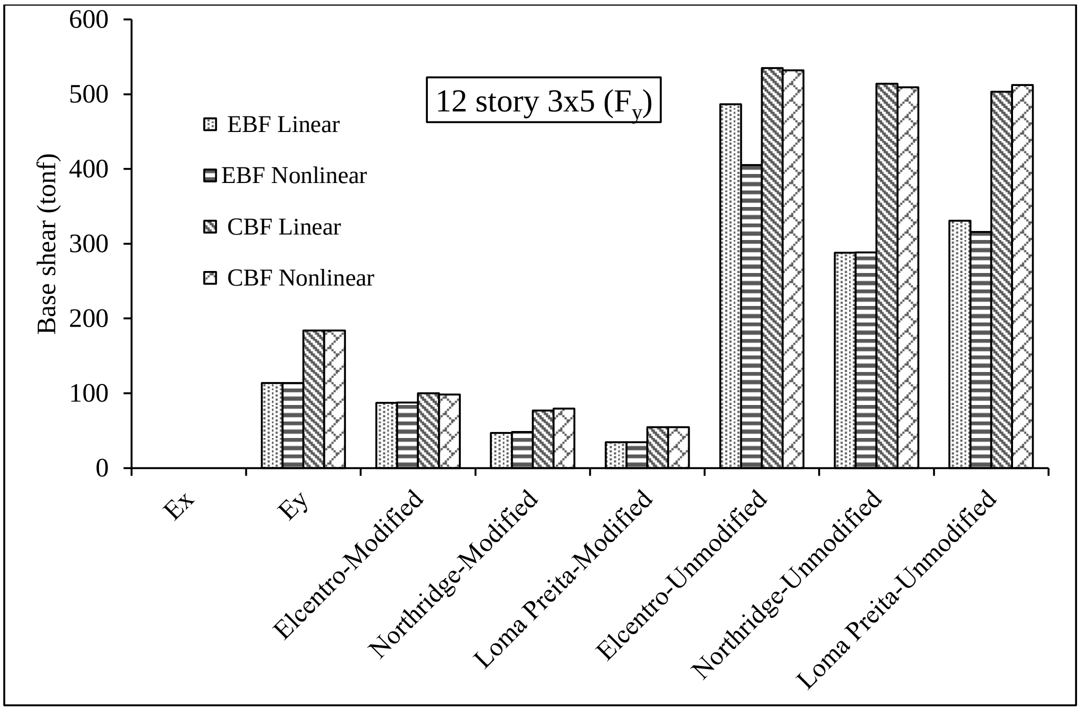


Figure 4.26: Base shear resulting from linear and nonlinear analysis of 12 story structures for 3x5 bay in y-direction

4.3.5 Investigating the Behaviour of EBF and CBF by Using Nonlinear Static Pushover Analysis

Figures 4.38 to 4.43 show the pushover for all 4, 8, and 12 story buildings in both x- and y- directions for CBF and EBF. As results show, structures with equal bays on both sides have similar pushover graphs on both sides. However, for structures with a different number of bays on both sides, the pushover for the side with more bays is larger. This pattern holds for both kinds of framing. In all structures, the initial stiffness of CBF is higher than that of the EBF.

The pushover graph of structures with a CBF located above those with an EBF indicating that the CBF enters the nonlinear range later and acts nonlinear with more force. Moreover, these two framing approaches undergo a small displacement in the nonlinear range indicating that these braces have friable behaviors. Graphs show that EBF have more plasticity than CBF.

The above mentioned results are dependent on the number of floors and the number of floors has no effect on the behavior of braces. As the number of floors increase so does the shear force and displacement. As the number of bays increase the shear force and displacement would also increase. Hence, the graph for pushover of structures with 5 bays located higher than those with 3 bays.

The study done by Mwafy and Elnashai (2001) has illustrated that pushover analysis is reliable and has the ability to predict the nonlinear behavior of ordinary frames if the lateral load distribution is chosen properly. In a similar study done by Maheri and Akbari (2003), pushover analysis is used to assess the ductility of ordinary frames from the work of Mwafy and Elnashai (2001).

Consequently, the nonlinear static procedure has been selected for this research where the models geometrically similar to those of Maheri and Akbari (2003). The results of the pushover analysis have been compared and verified before by Mwafy and Elnashai (2001). The pushover results of this study can also be easily verified by referring to the two mentioned studies. The choice of method is also in line with the findings of Moghaddam and Hajirasouliha (2006), Kim and Choi (2005) and the methodology used by Kim and Choi (2005) for a study analogous to this study.These results for EBF have similarity with Nourbakhsh (2011) and For CBF they are in line with Frazam (2009). They found structures with equal bays on both sides have similar pushover graphs on both sides. However, for structures with a different number of bays on both sides, the pushover for the side with more bays is larger.

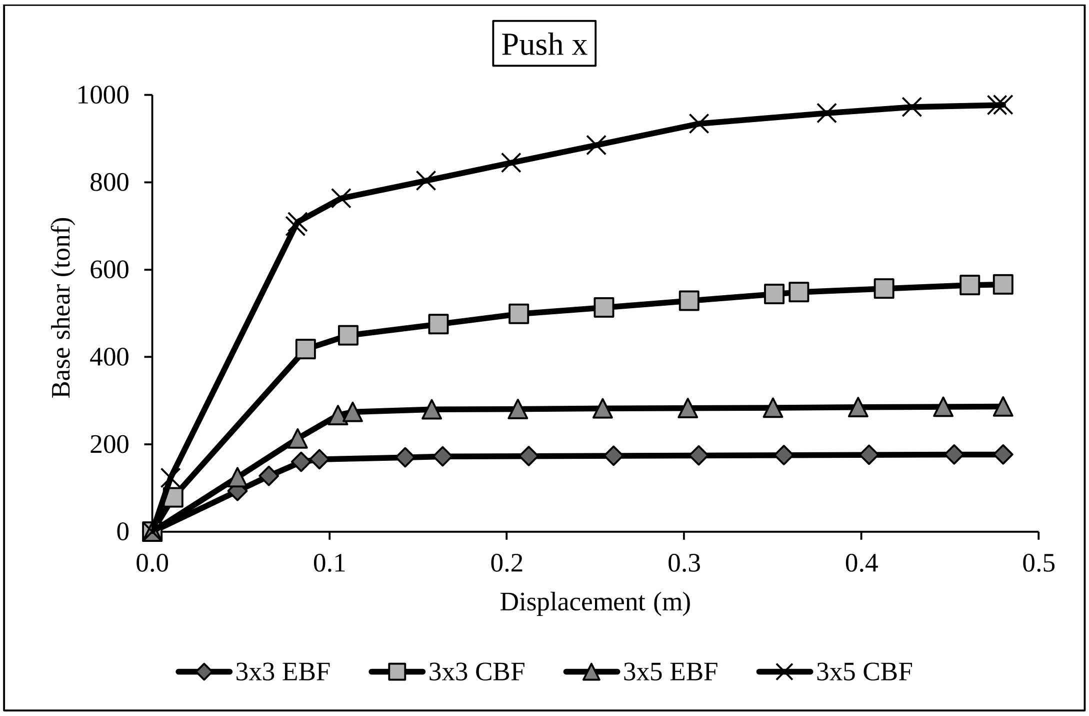


Figure 4.27: Pushover in x-direction for 4 story structure

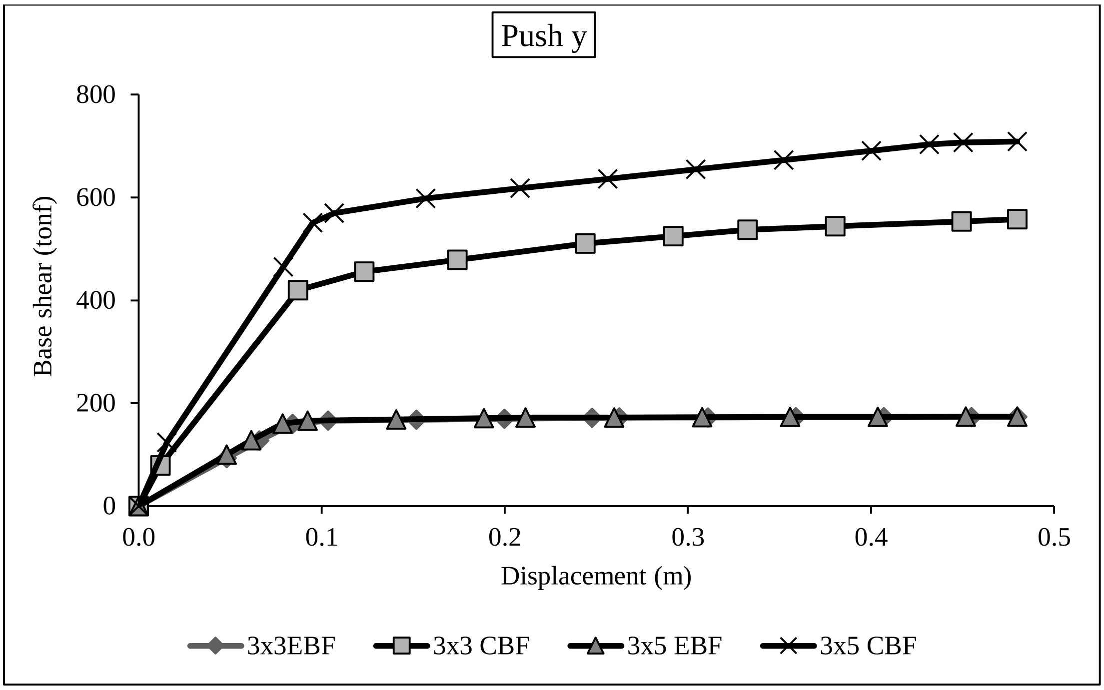


Figure 4.28: Pushover in y-direction for 4 story structure

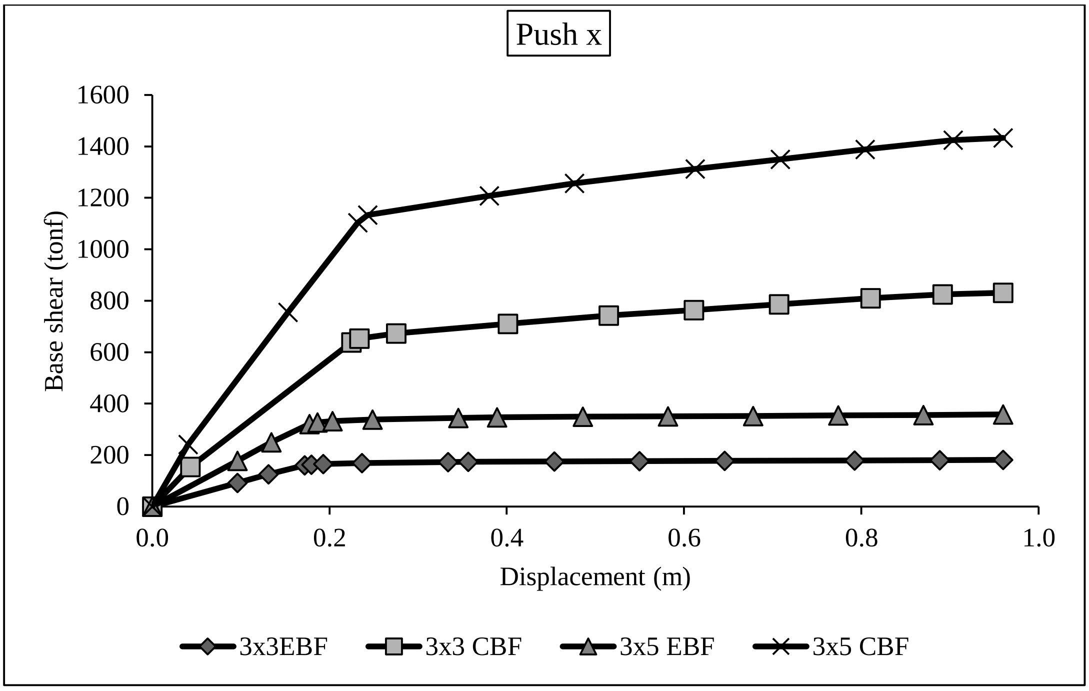


Figure 4.29: Pushover in x-direction for 8 story structure

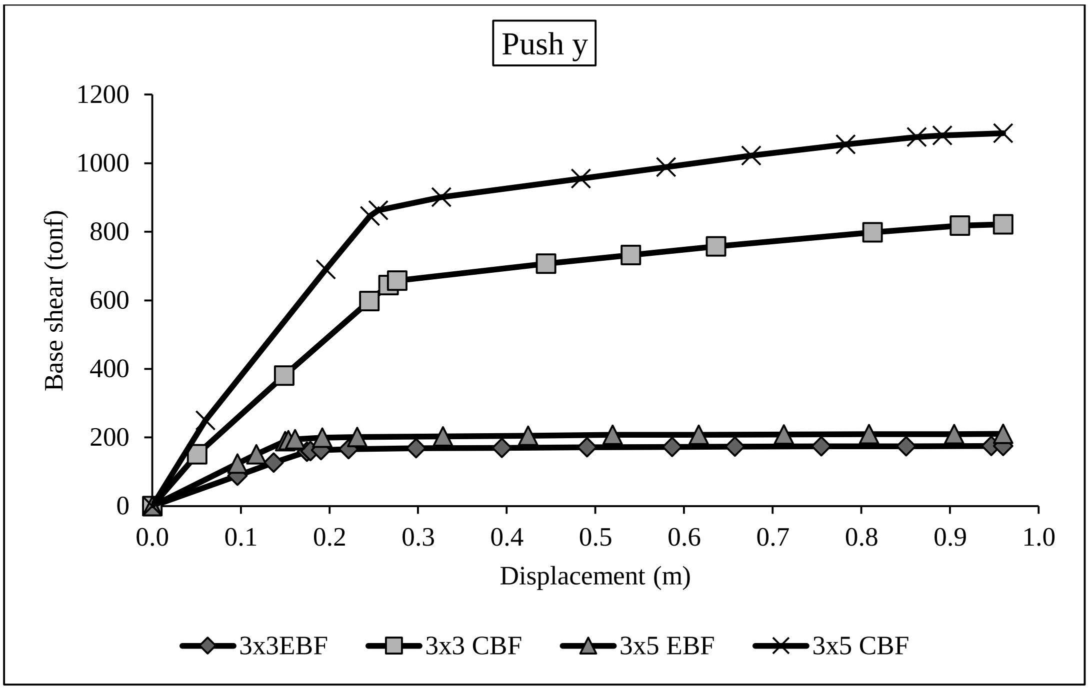


Figure 4.30: Pushover in y-direction for 8 story structure

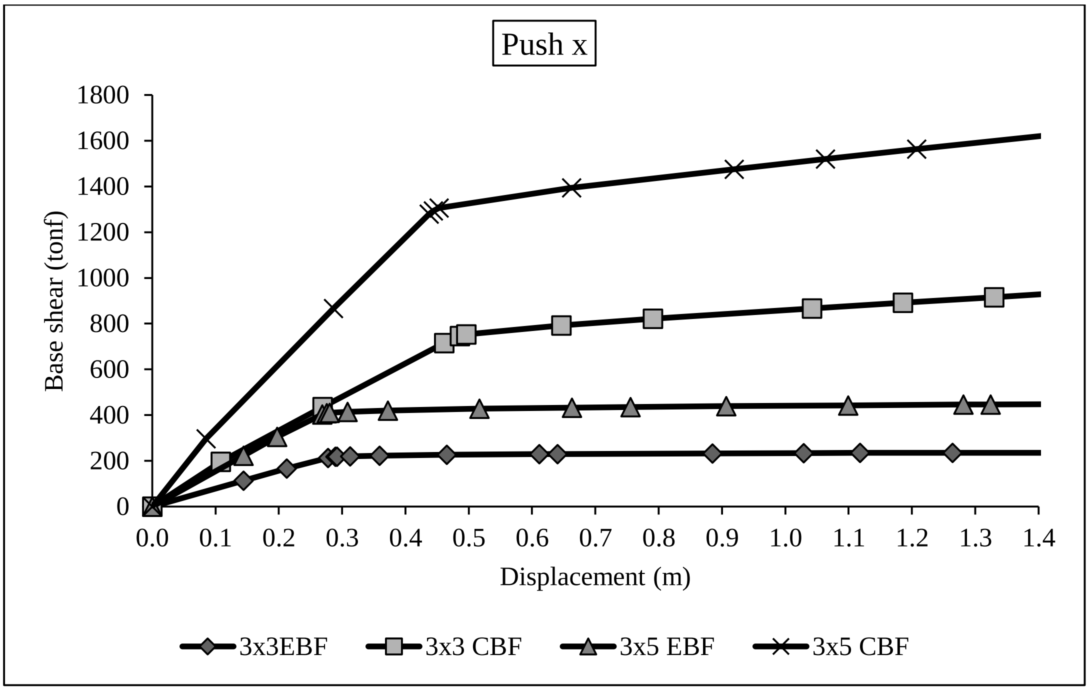


Figure 4.31: Pushover in x-direction for 12 story structure

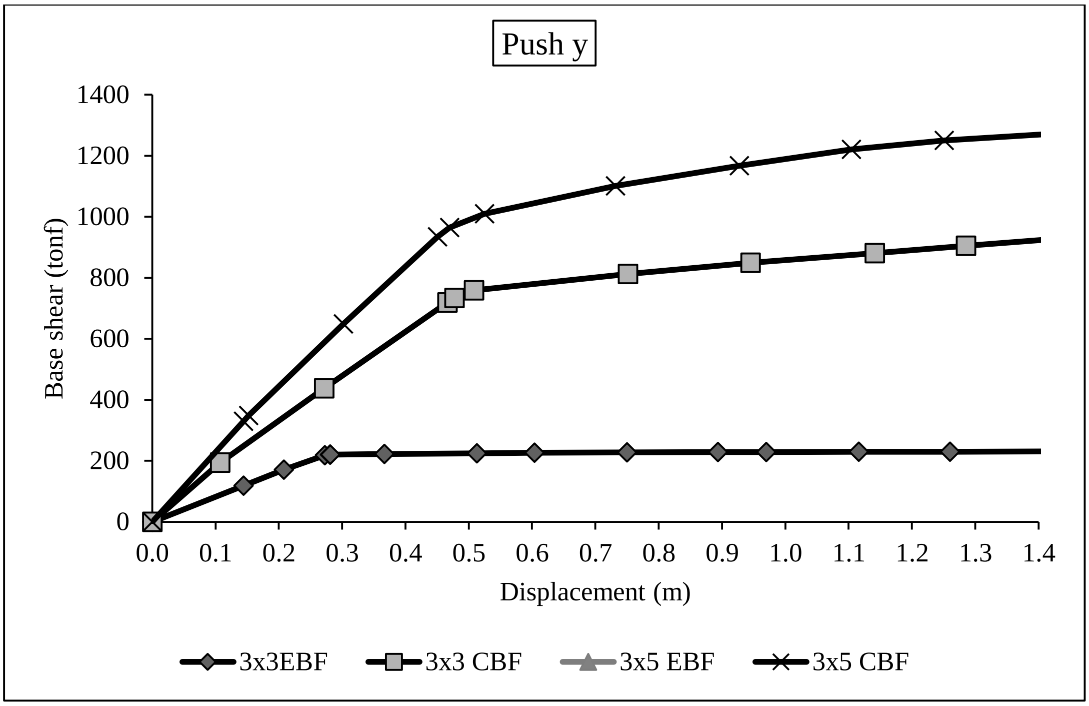


Figure 4.32: Pushover in y-direction for 12 story structure

4.3.6 Target Displacement (shift)

The target shift and corresponding force were digitally calculated for all structures and results are provided in Figures 4.44 to 4.49 and Tables A4-A7 in Appendix. In structures with the same number of bays in both directions of x- and y- (equaling up to 3) the target shift is equal on both sides, and it is in depended from the type of bracing. In structures with the unequal number of bays in both directions of x- and y- (3x5), the side with more bays has a higher target shift. This shift is bigger in structures with EBF. Given this, one can conclude that the EBF shows higher plasticity than those of CBF as it shown in Figures 4.44 to 4.49.

4.3.6.1 Effects of the Type of Brace

Target shift is larger on both sides for EBF than CBF. As a result, 4-story building with 3 bays on both sides has a shift of 40% versus a 30% shift for a 12-story building. So, increasing the number of floors decreased the target shift for both CBF and EBF.

4.3.6.2 Effect of the Number of Floors

Increasing the number of floors makes the target shift increase, such that, in an EBF-structure with 3 bays on both sides, increasing the number of floors from 4 to 8 increased the target shift by 100%. Increasing the number of floors from 8 to 12 produced a target shift of 37%.

In structures with similar specifications but with CBF increasing the number of floors from 4 to 8 increased the target shift by 85%. Increasing the number of floors from 8 to 12 floors produced a target shift of 57%.

4.3.6.3 Effects of the Number of Bays

The target shift increases in the direction with more bays. In a 4-story structure with EBF the target shift increased by 6% when the number of bays increase. For a structure with 12 floors there was a 24% increase for CBF.

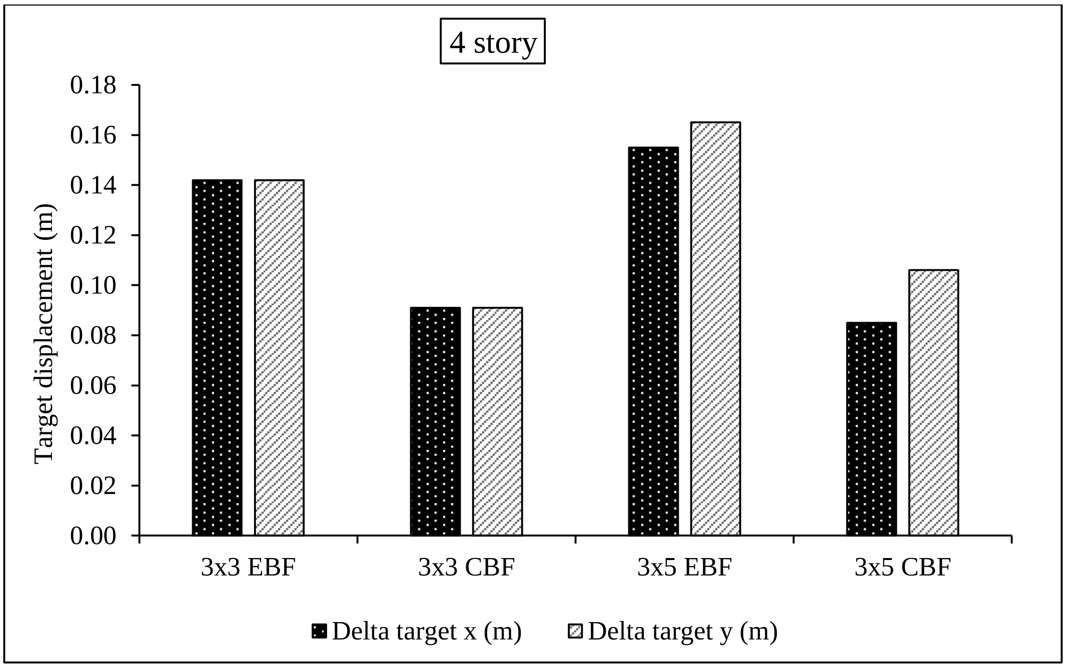


Figure 4.33: Target displacment 4 story

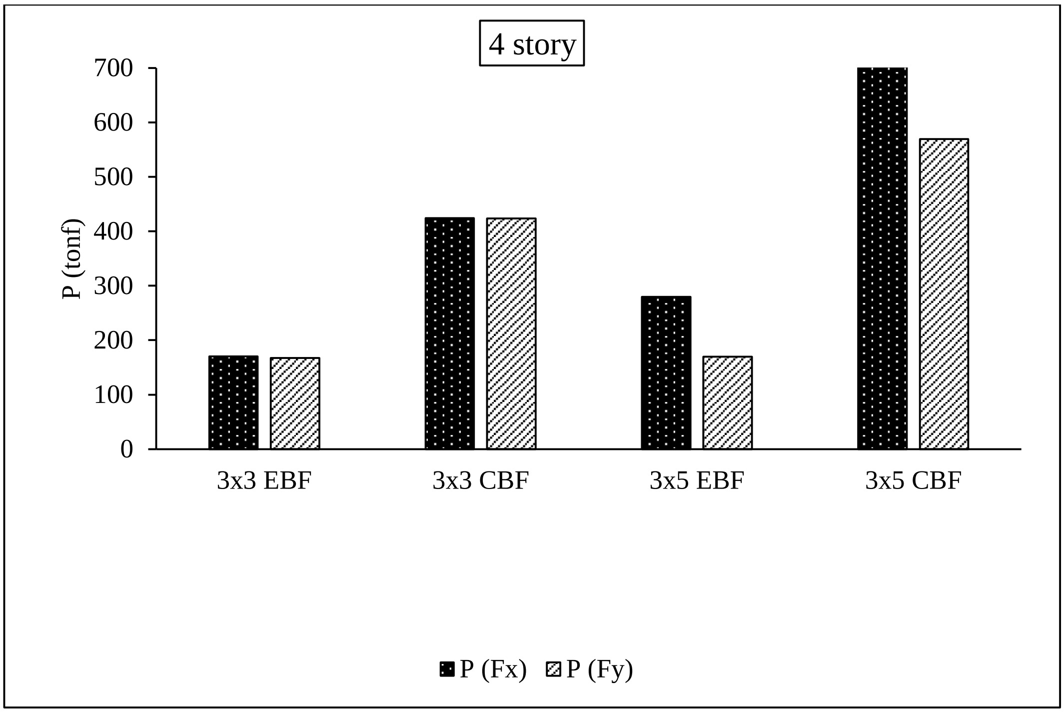


Figure 4.34: Target displacement according to force for 4 story

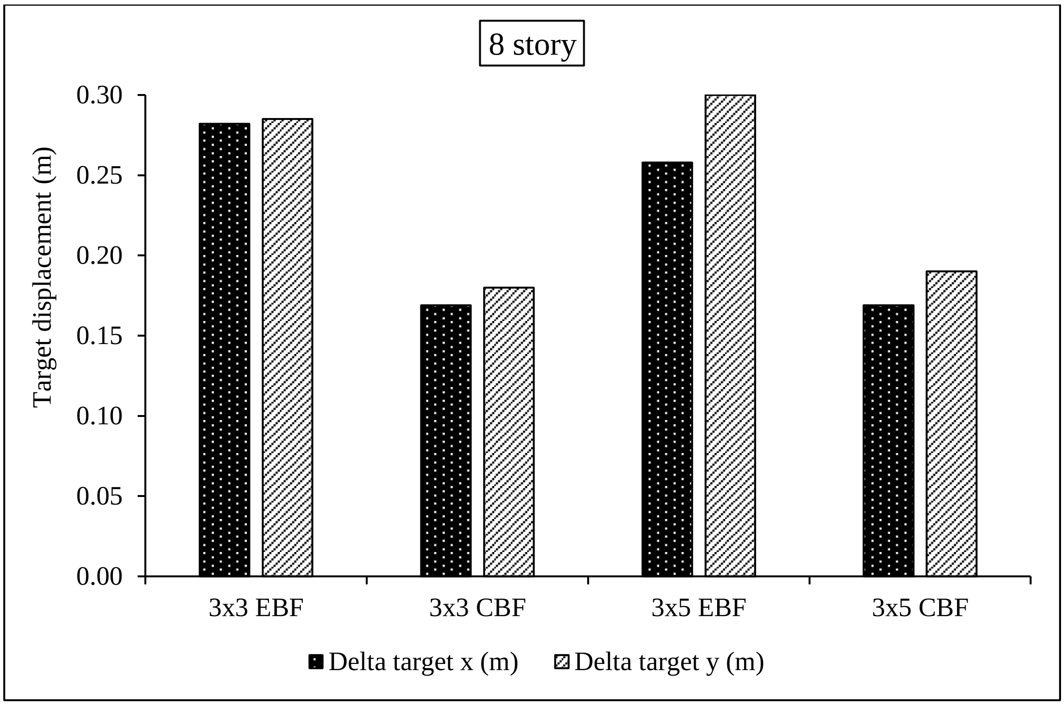


Figure 4.35: Target displacement 8 story

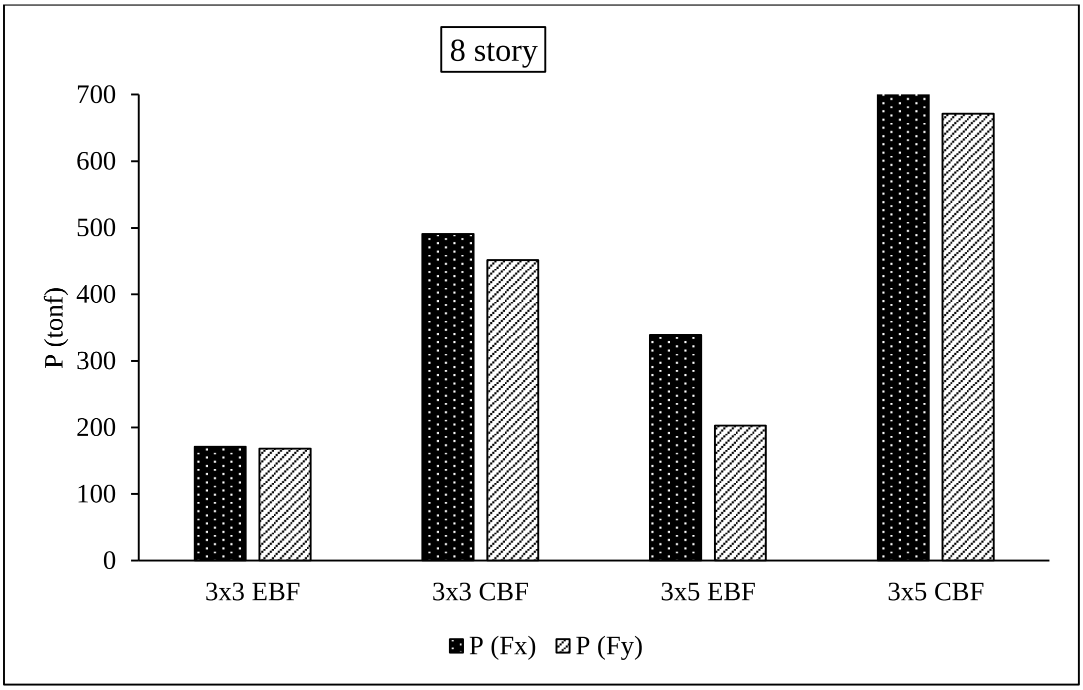


Figure 4.36: Target displacement according to force for 8 story

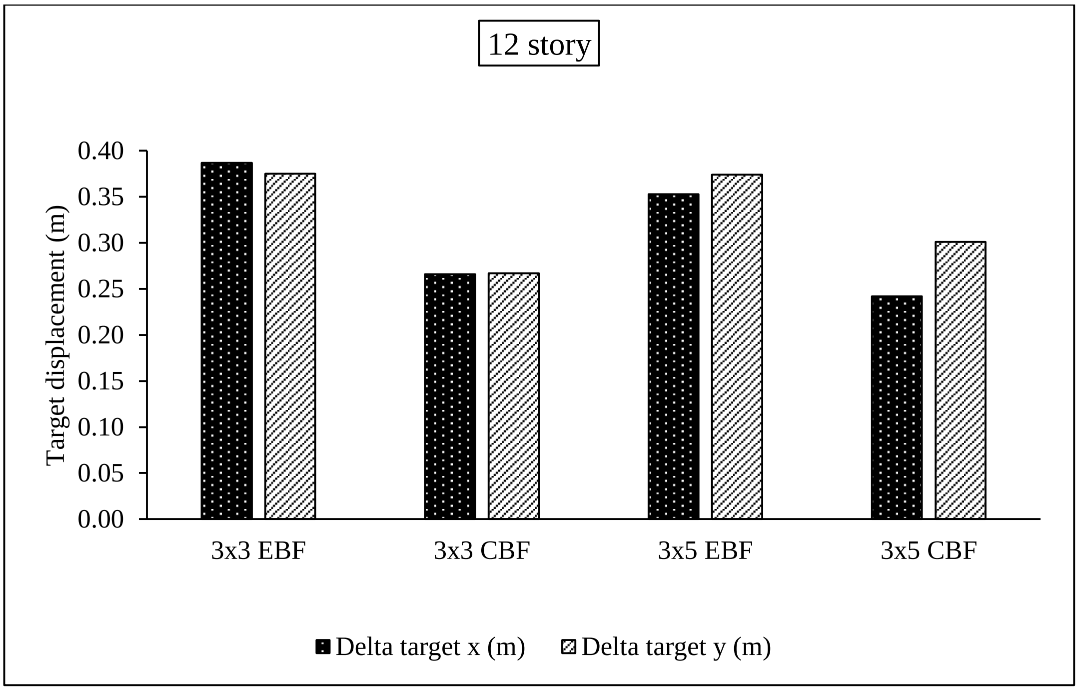


Figure 4.37: Target displacement 12 story

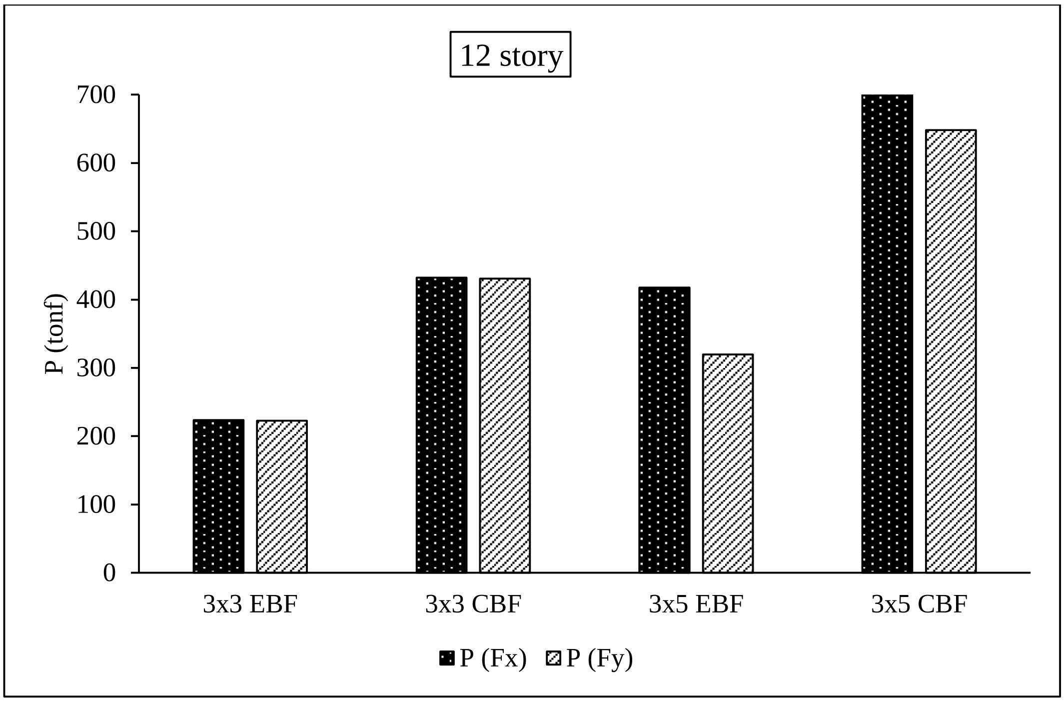


Figure 4.38: Target displacement according to force for 12 story

4.3.7 Deformation for Structure Under Pushover Analyses

Figures 4.50 to 4.53 show the position of hinges at target displacement. For CBF structures first hinge formed in bracing connections while the target displacements are linear. Hinges reach IO (Immediate Occupancy) for both 3x3 and 3x5 CBF. For structure with EBF bracing first hinge was formed in beam to bracing connections as the connection behavior becomes non-linear. It can be seen that at the lower two and three floors for 3x3 and 3x5 bays, respectively, reached Collapse Prevention (CP) behavior.

According to FEMA 2000, decreasing of more than 20% or more decrease in the lateral force of the idealized pushover curve of the frame can be considered as a failure mode. This failure mode had also been considered by other researchers, such as: Inel and Ozmen (2006), Arash Farzam (2009) and Mehrdad Nourbakhsh (2011) in their study. In this study, the same failure mode has been exerted.

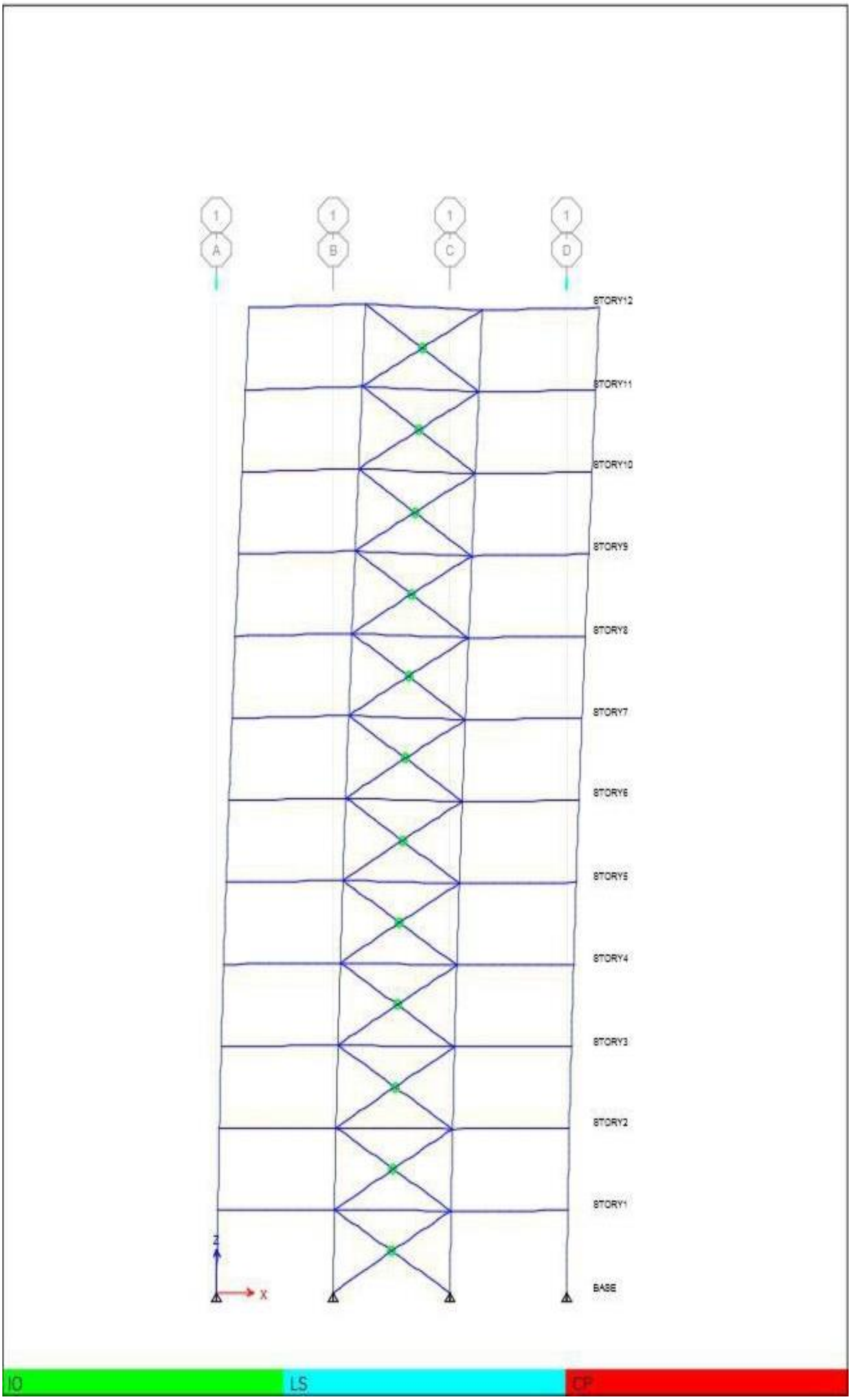


Figure 4.39: Deformation of 12 story 3x3 CBF structure due to pushover analysis.

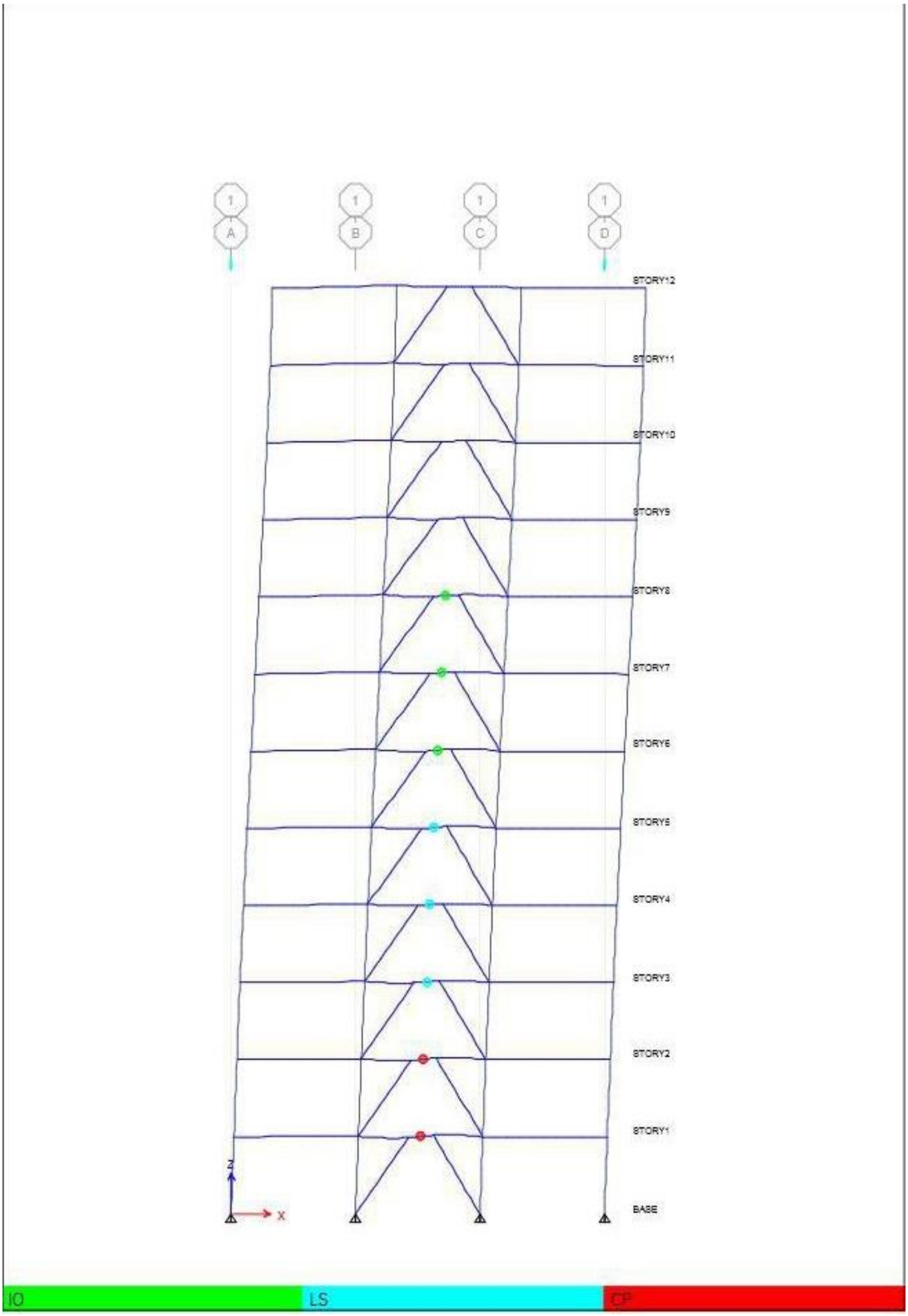


Figure 4.40: Deformation of 12 story 3x3 EBF structure due to pushover analys.

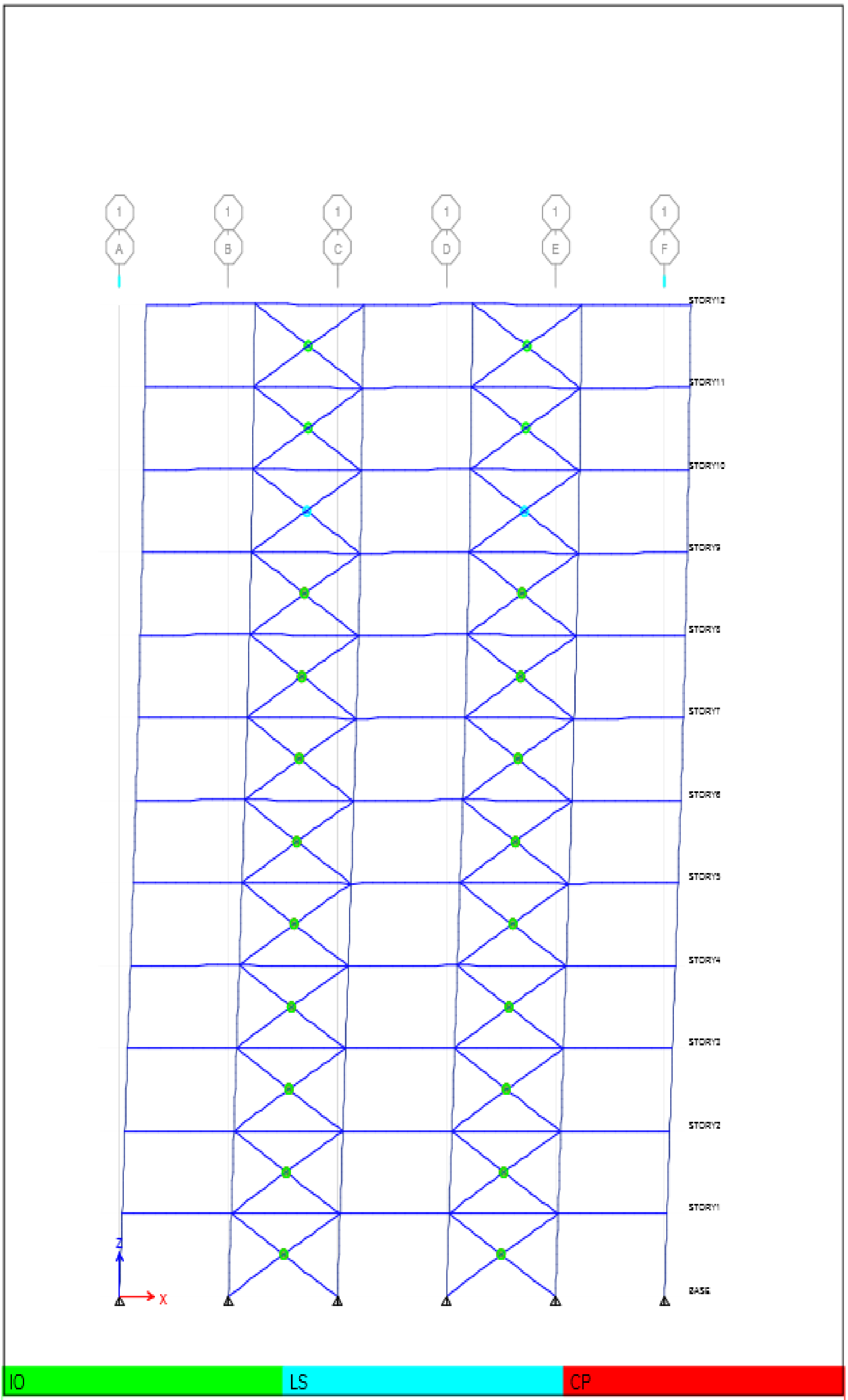


Figure 4.41**:** Deformation of 12 story 3x5 CBF structure due to pushover analys.

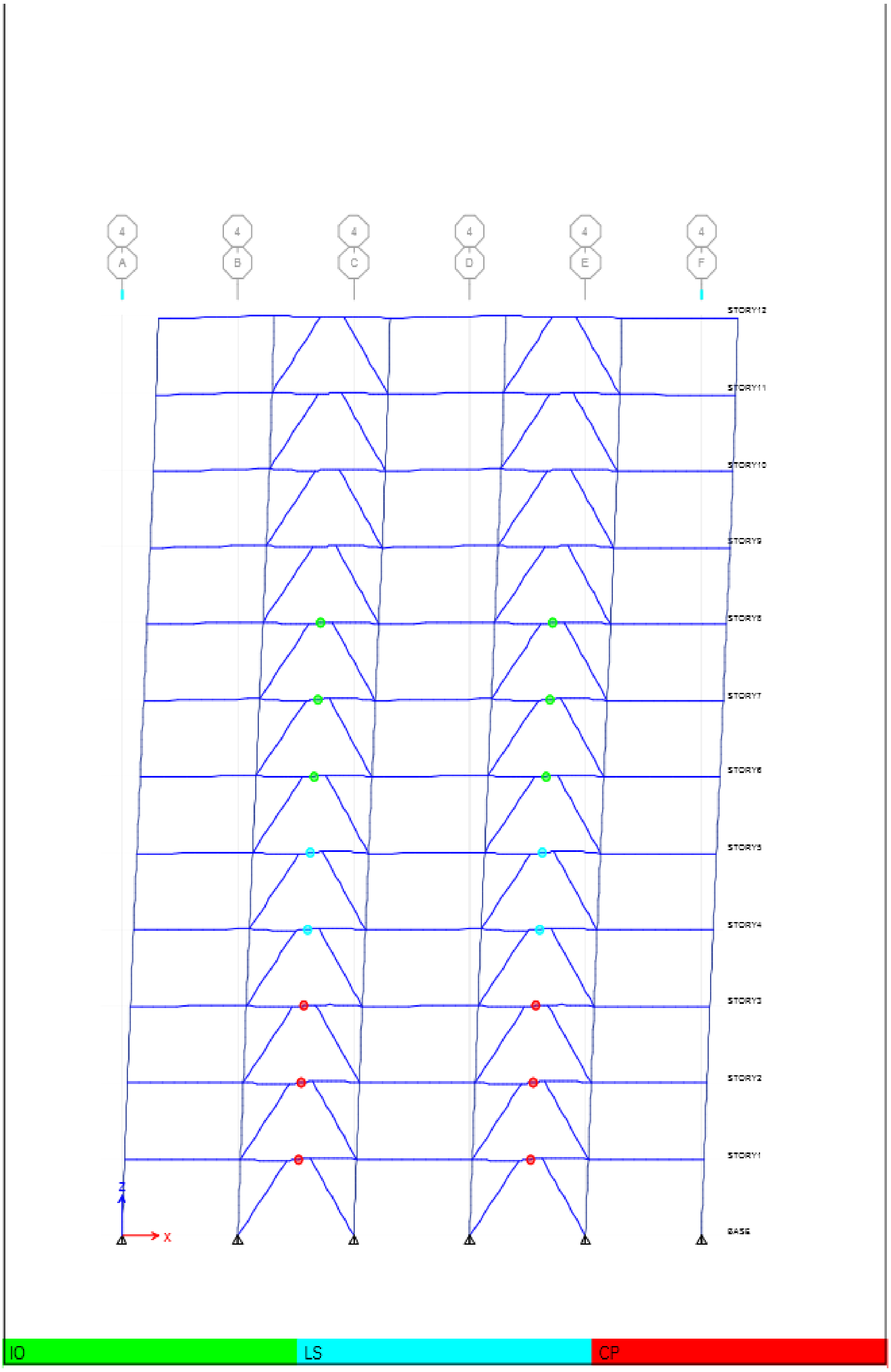


Figure 4.42**:** Deformation of 12 story 3x5 EBF structure due to pushover analys.

Chapter 5

CONCLUSION AND RECOMMENDATION FOR FUTURE WORK

5.1 Conclusions

This thesis aimed at investigating the behavior of structures with concentric and eccentric braces. For this purpose, 36 buildings with 4, 8, and 12 stories, and with two types of plans were considered. One of the plans composed of 3 bays on both directions and the other had 5 bays in the x-direction and 3 bays on y-direction. Buildings were modeled in both linear and nonlinear dynamic states. Models have been loaded by three modified and three unmodified accelerogarms of Electro, Northridge and Loma prieta.

The following are the conclusions:

1) The EBF displaces more than CBF for linear dynamic analysis.
2) For 4 story buildings, c_d coefficient suggested by ASCE7-10 for both types of braces is the lowest and independent from number of bays.
3) For 8 story buildings, the c_d coefficient was nearly equal to the predicted values. On the other hand linear and nonlinear analysis displacement values were similar.
4) For 12 story buildings, c_d coefficient is conservative in ASCE7-10 and the linear analysis displacement is larger than the nonlinear.

5) In addition to the type of brace mentioned in ASCE7-10, c_d coefficient depends on the number of floors and is independent from number of bays.
6) The history of base shear states; that in modified accelerograms linear and nonlinear had same results. In unmodified accelerograms results of the nonlinear base shear are lower than linear ones indicating that the structure has entered the nonlinear range.
7) Results show that base shear in modified accelerograms is nearly equal to equivalent static ones, and linear and nonlinear results are the same because the structure has not yet entered into the nonlinear range. The base shear increases as both the number of bays and floors increases due to rise of the weight of structure.
8) There is considerable difference between linear and nonlinear results of base shear of the unmodified accelerograms; the linear results are greater than that of the nonlinear since with the increase in base shear the structures enter the nonlinear range.
9) In all structures, the initial stiffness of CBF was more than that of EBF.
10) The pushover graph of structures with CBF is higher than that of EBF ones indicating that the CBF enters the nonlinear range later than EBF. Moreover, these braces enter the nonlinear range in a small shift indicating friable behavior of these braces. Graphs indicate that EBF braces have more plasticity than CBF.
11) Target shift is larger in both directions for EBF when compared to those of CBF.
12) On the other hand, for 3x3 bays increase in the number of floors lead to increase in the target shift for both CBF and EBF. For example, when the

number of floors increased from 4 to 8 for an EBF structure the target shift was increased by 100% and increasing the number of floors from 8 to 12 also increased the target shift by 37%. In structures with similar specifications but with CBF when the number of floors increased from 4 to 8 and 8 to 12 the target shift was also increased by 85% and 57%, respectively.

13) The target shift increases in the direction with more bays. In a 4-story structure with EBF the target shift increased by 6% when the number of bays increases. For a structure with 12 floors there was a 24% increase for target shift for CBF.

5.2 Overall Conclusions

On the basis of the above mentioned reasons and discussions about numerous frames in this study it can be concluded that

- Displacement for EBF was higher than CBF but the drift for both bracing systems were almost similar and lower than allowed value.
- In all structural frames stiffness for CBF was higher than that of EBF.
- Pushover analysis graphs indicate that CBF will enter the nonlinear region later than EBF.
- Target shift for EBF was bigger in both directions when compared to CBF.

All in all we can conclude that stiffness and behavior of CBF is better than EBF in case of earthquake.

5.3 Recommendations for Future Work

It is recommended that more studies need to be carried out to be able to compare for other type of braces with different number of floors and different bays.

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APPENDIX

Appendix A: Pushover Results

Table A.1: Result of pushover analyses in x-direction for 4 story

Displacement and force in x-direction							
3x3 EBF		3x3 CBF		3x5 EBF		3x5 CBF	
Displacement (m)	Force (tonf)	Displacement (m)	Force (tonf)	Displacment (m)	Force (tonf)	Displacment (m)	Force (tonf)
0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
0.048	93.144	0.012	78.483	0.048	124.794	0.010	123.183
0.066	127.558	0.087	417.793	0.082	213.397	0.081	699.441
0.084	159.719	0.110	449.754	0.105	267.064	0.082	709.378
0.094	165.562	0.162	475.447	0.113	274.109	0.107	763.314
0.143	170.226	0.207	498.264	0.158	279.799	0.155	803.858
0.164	172.277	0.255	513.421	0.206	280.861	0.203	844.401
0.212	172.967	0.303	528.578	0.254	281.912	0.251	884.945
0.260	173.651	0.351	543.735	0.302	282.964	0.308	933.873
0.308	174.335	0.365	548.133	0.350	284.015	0.381	958.150
0.356	175.018	0.413	556.332	0.398	285.067	0.429	972.020
0.404	175.702	0.461	564.613	0.446	286.119	0.477	976.799
0.452	176.386	0.480	565.761	0.480	286.859	0.480	977.139
0.480	176.780	0.000	0.000	0.000	0.000	0.000	0.000

Table A.2: Result of pushover analyses in y-direction for 4 story

Displacement and force in y-direction							
3x3EBF		3x3 CBF		3x5 EBF		3x5 CBF	
Displacement (m)	Force (tonf)	Displacement (m)	Force (tonf)	Displacment (m)	Force (tonf)	Displacment (m)	Force (tonf)
0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
0.048	92.928	0.012	78.861	0.048	99.678	0.015	123.206
0.066	127.499	0.087	41.436	0.062	127.831	0.079	464.934
0.084	159.494	0.123	45.532	0.079	159.949	0.095	550.578
0.103	165.564	0.174	478.551	0.092	165.616	0.107	569.479
0.152	167.479	0.244	510.132	0.141	168.134	0.157	598.322
0.200	169.383	0.292	524.904	0.189	170.637	0.208	617.937
0.248	171.287	0.333	537.385	0.211	171.821	0.256	636.195
0.263	171.874	0.381	543.933	0.260	172.217	0.304	654.452
0.311	172.133	0.450	553.339	0.308	172.610	0.352	672.710
0.359	172.390	0.480	557.496	0.356	173.003	0.400	690.968
0.407	172.646	0.000	0.000	0.404	173.396	0.432	702.994
0.455	172.903	0.000	0.000	0.452	173.789	0.450	707.085
0.480	173.036	0.000	0.000	0.480	174.020	0.480	708.758

Table A.3: Result of pushover analyses in x-direction for 8 story

Displacement and force in x-direction							
3x3EBF		3x3 CBF		3x5 EBF		3x5 CBF	
Displacement (m)	Force (tonf)	Displacement (m)	Force (tonf)	Displacment (m)	Force (tonf)	Displacment (m)	Force (tonf)
0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
0.096	92.160	0.043	154.723	0.096	178.489	0.040	241.416
0.131	125.996	0.225	637.734	0.135	250.154	0.153	754.845
0.172	160.811	0.234	653.617	0.177	320.597	0.232	110.952
0.180	163.568	0.275	673.451	0.187	326.594	0.243	113.916
0.193	165.971	0.401	710.545	0.203	331.912	0.380	1208.020
0.237	169.553	0.515	742.229	0.248	338.543	0.476	1256.332
0.334	173.109	0.611	764.365	0.345	344.601	0.612	1312.673
0.357	173.952	0.707	786.501	0.389	347.322	0.708	1350.353
0.454	175.197	0.810	810.325	0.486	349.119	0.804	1388.033
0.550	176.428	0.892	825.224	0.582	350.897	0.904	1424.762
0.646	177.659	0.960	831.626	0.678	352.676	0.960	1433.679
0.793	179.519	0.000	0.000	0.774	354.454	0.000	0.000
0.889	180.728	0.000	0.000	0.870	356.232	0.000	0.000
0.960	181.627	0.000	0.000	0.960	357.878	0.000	0.000

Table A.4: Result of pushover analyses in y-direction for 8 story

Displacement and force in y-direction							
3x3EBF		3x3 CBF		3x5 EBF		3x5 CBF	
Displacment (m)	Force (tonf)	Displacment (m)	Force (tonf)	Displacment (m)	Force (tonf)	Displacment (m)	Force (tonf)
0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
0.096	88.375	0.051	151.102	0.096	123.352	0.060	249.369
0.137	126.090	0.149	380.031	0.117	150.789	0.196	689.903
0.175	157.878	0.245	597.957	0.150	188.869	0.246	846.074
0.179	160.159	0.267	644.700	0.154	191.667	0.255	862.717
0.191	163.111	0.276	656.965	0.161	194.392	0.326	900.516
0.221	166.059	0.444	706.500	0.192	199.042	0.484	955.240
0.298	168.539	0.540	731.708	0.231	201.281	0.580	988.599
0.395	169.790	0.636	756.917	0.328	203.272	0.676	102.958
0.491	171.027	0.813	797.892	0.424	205.243	0.782	105.080
0.587	172.265	0.911	818.185	0.519	207.196	0.862	107.539
0.658	173.180	0.960	821.238	0.617	207.827	0.891	108.833
0.755	173.628	0.000	0.000	0.713	208.452	0.960	1087.399
0.851	174.071	0.000	0.000	0.809	209.077	0.000	0.000
0.947	174.513	0.000	0.000	0.905	209.701	0.000	0.000
0.960	174.575	0.000	0.000	0.960	210.062	0.000	0.000

Table A.5: Results of pushover analyses in x-direction for 12 story structure

Displacement and force in x-direction							
3x3EBF		3x3 CBF		3x5 EBF		3x5 CBF	
Displacement (m)	Force (tonf)	Displacement (m)	Force (tonf)	Displacment (m)	Force (tonf)	Displacment (m)	Force (tonf)
0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
0.144	112.999	0.108	196.258	0.144	222.577	0.085	297.288
0.213	166.897	0.269	435.584	0.197	305.095	0.286	866.763
0.278	213.242	0.461	714.587	0.268	403.349	0.437	127.626
0.290	217.945	0.486	745.417	0.275	408.210	0.444	1293.081
0.292	218.500	0.496	753.244	0.280	409.569	0.453	1305.727
0.312	220.335	0.646	792.183	0.308	413.753	0.662	1393.715
0.359	222.837	0.791	822.095	0.372	419.513	0.919	1475.847
0.465	226.307	1.042	866.531	0.517	427.585	1.063	1519.693
0.611	229.231	1.186	891.215	0.663	432.743	1.207	1563.539
0.640	229.809	1.330	915.899	0.756	435.749	1.404	1620.469
0.885	232.296	1.440	934.722	0.906	438.786	1.440	1628.791
1.029	233.744	0.000	0.000	1.099	442.602	0.000	0.000
1.118	234.640	0.000	0.000	1.281	446.149	0.000	0.000
1.264	235.146	0.000	0.000	1.324	446.874	0.000	0.000
1.408	235.644	0.000	0.000	1.440	447.839	0.000	0.000
1.440	235.755	0.000	0.000	0.000	0.000	0.000	0.000

Table A.6: Results of pushover analyses in y-direction for 12 story structure

Displacement and force in y-direction							
3x3EBF		3x3 CBF		3x5 EBF		3x5 CBF	
Displacement (m)	Force (tonf)	Displacement (m)	Force (tonf)	Displacement (m)	Force (tonf)	Displacment (m)	Force (tonf)
0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
0.144	118.030	0.107	193.555	0.144	183.945	0.144	328.920
0.208	170.340	0.272	437.370	0.186	237.788	0.152	348.201
0.273	217.604	0.466	719.022	0.241	301.056	0.302	649.089
0.281	219.996	0.478	733.786	0.251	306.335	0.451	934.594
0.281	220.129	0.508	758.339	0.265	310.109	0.470	964.663
0.367	222.336	0.751	812.922	0.292	313.913	0.525	1009.530
0.513	224.542	0.945	848.848	0.364	319.271	0.732	1100.963
0.604	225.920	1.141	881.027	0.391	320.428	0.928	1166.992
0.750	227.089	1.285	904.625	0.509	323.330	1.104	1220.445
0.894	228.242	1.429	928.223	0.655	325.410	1.251	1250.167
0.970	228.851	1.440	929.964	0.761	326.905	1.415	1271.722
1.116	229.361	0.000	0.000	0.907	327.919	1.440	1274.051
1.260	229.864	0.000	0.000	1.170	329.728	0.000	0.000
1.404	230.367	0.000	0.000	1.314	330.713	0.000	0.000
1.440	230.494	0.000	0.000	1.440	331.577	0.000	0.000

Table A.7: Delta target in x and y directions for 4 story

Form	P (F_x)	Delta target x (m)
3x3 EBF	170.19	0.142
3x3 CBF	423.94	0.091
3x5 EBF	279.48	0.155
3x5 CBF	716.54	0.085
Form	P (F_y)	Delta target y (m)
3x3 EBF	167.09	0.142
3x3 CBF	423.62	0.091
3x5 EBF	169.44	0.165
3x5 CBF	569.35	0.106

Table A. 8: Delta target in x and y direction for 8 story

Form	P (F_x)	Delta target x (m)
3x3 EBF	171.21	0.282
3x3 CBF	490.94	0.169
3x5 EBF	339.14	0.258
3x5 CBF	809.41	0.169
Form	P (F_y)	Delta target y (m)
3x3 EBF	168.15	0.285
3x3 CBF	451.39	0.180
3x5 EBF	202.71	0.300
3x5 CBF	671.64	0.190

Table A.9: Delta target in x and y direction for 12 story

Form	P (F_x)	Delta target x (m)
3x3 EBF	223.75	0.387
3x3 CBF	432.02	0.266
3x5 EBF	417.76	0.353
3x5 CBF	743.68	0.242
Form	P (F_y)	Delta target y (m)
3x3 EBF	222.47	0.375
3x3 CBF	430.53	0.267
3x5 EBF	319.71	0.374
3x5 CBF	648.37	0.301

Table A.10: Different sections that were used for column system in 4 story 3x3 EBF

Story no	Label	Design type	Section type	Design procedure	Design section
STORY1	C1	Column	Steel I/Wide Flange	Steel Frame Design	HE120B
STORY1	C2	Column	Steel I/Wide Flange	Steel Frame Design	HE280B
STORY1	C3	Column	Steel I/Wide Flange	Steel Frame Design	HE280B
STORY1	C4	Column	Steel I/Wide Flange	Steel Frame Design	HE120B
STORY1	C5	Column	Steel I/Wide Flange	Steel Frame Design	HE280B
STORY1	C6	Column	Steel I/Wide Flange	Steel Frame Design	HE200B
STORY1	C7	Column	Steel I/Wide Flange	Steel Frame Design	HE200B
STORY1	C8	Column	Steel I/Wide Flange	Steel Frame Design	HE280B
STORY1	C9	Column	Steel I/Wide Flange	Steel Frame Design	HE280B
STORY1	C10	Column	Steel I/Wide Flange	Steel Frame Design	HE200B
STORY1	C11	Column	Steel I/Wide Flange	Steel Frame Design	HE200B
STORY1	C12	Column	Steel I/Wide Flange	Steel Frame Design	HE280B
STORY1	C13	Column	Steel I/Wide Flange	Steel Frame Design	HE120B
STORY1	C14	Column	Steel I/Wide Flange	Steel Frame Design	HE280B
STORY1	C15	Column	Steel I/Wide Flange	Steel Frame Design	HE280B
STORY1	C16	Column	Steel I/Wide Flange	Steel Frame Design	HE120B

Table A.11: Different sections that were used for column system in 4 story 3x3 EBF

Story no	Label	Design type	Section type	Design procedure	Design section
STORY2	C1	Column	Steel I/Wide Flange	Steel Frame Design	HE120B
STORY2	C2	Column	Steel I/Wide Flange	Steel Frame Design	HE240B
STORY2	C3	Column	Steel I/Wide Flange	Steel Frame Design	HE240B
STORY2	C4	Column	Steel I/Wide Flange	Steel Frame Design	HE120B
STORY2	C5	Column	Steel I/Wide Flange	Steel Frame Design	HE240B
STORY2	C6	Column	Steel I/Wide Flange	Steel Frame Design	HE160B
STORY2	C7	Column	Steel I/Wide Flange	Steel Frame Design	HE160B
STORY2	C8	Column	Steel I/Wide Flange	Steel Frame Design	HE240B
STORY2	C9	Column	Steel I/Wide Flange	Steel Frame Design	HE240B
STORY2	C10	Column	Steel I/Wide Flange	Steel Frame Design	HE160B
STORY2	C11	Column	Steel I/Wide Flange	Steel Frame Design	HE160B
STORY2	C12	Column	Steel I/Wide Flange	Steel Frame Design	HE240B
STORY2	C13	Column	Steel I/Wide Flange	Steel Frame Design	HE120B
STORY2	C14	Column	Steel I/Wide Flange	Steel Frame Design	HE240B
STORY2	C15	Column	Steel I/Wide Flange	Steel Frame Design	HE240B
STORY2	C16	Column	Steel I/Wide Flange	Steel Frame Design	HE120B

Table A.12: Different sections that were used for column system in 4 story 3x3 EBF

Story no	Label	Design type	Section type	Design procedure	Design section
STORY3	C1	Column	Steel I/Wide Flange	Steel Frame Design	HE100B
STORY3	C2	Column	Steel I/Wide Flange	Steel Frame Design	HE200B
STORY3	C3	Column	Steel I/Wide Flange	Steel Frame Design	HE200B
STORY3	C4	Column	Steel I/Wide Flange	Steel Frame Design	HE100B
STORY3	C5	Column	Steel I/Wide Flange	Steel Frame Design	HE180B
STORY3	C6	Column	Steel I/Wide Flange	Steel Frame Design	HE140B
STORY3	C7	Column	Steel I/Wide Flange	Steel Frame Design	HE140B
STORY3	C8	Column	Steel I/Wide Flange	Steel Frame Design	HE180B
STORY3	C9	Column	Steel I/Wide Flange	Steel Frame Design	HE180B
STORY3	C10	Column	Steel I/Wide Flange	Steel Frame Design	HE140B
STORY3	C11	Column	Steel I/Wide Flange	Steel Frame Design	HE140B
STORY3	C12	Column	Steel I/Wide Flange	Steel Frame Design	HE180B
STORY3	C13	Column	Steel I/Wide Flange	Steel Frame Design	HE100B
STORY3	C14	Column	Steel I/Wide Flange	Steel Frame Design	HE200B
STORY3	C15	Column	Steel I/Wide Flange	Steel Frame Design	HE200B
STORY3	C16	Column	Steel I/Wide Flange	Steel Frame Design	HE100B

Table A.13: Different sections that were used for column system in 4 story 3x3 EBF

Story no	Label	Design type	Section type	Design procedure	Design section
STORY4	C1	Column	Steel I/Wide Flange	Steel Frame Design	HE100B
STORY4	C2	Column	Steel I/Wide Flange	Steel Frame Design	HE120B
STORY4	C3	Column	Steel I/Wide Flange	Steel Frame Design	HE120B
STORY4	C4	Column	Steel I/Wide Flange	Steel Frame Design	HE100B
STORY4	C5	Column	Steel I/Wide Flange	Steel Frame Design	HE120B
STORY4	C6	Column	Steel I/Wide Flange	Steel Frame Design	HE100B
STORY4	C7	Column	Steel I/Wide Flange	Steel Frame Design	HE100B
STORY4	C8	Column	Steel I/Wide Flange	Steel Frame Design	HE120B
STORY4	C9	Column	Steel I/Wide Flange	Steel Frame Design	HE120B
STORY4	C10	Column	Steel I/Wide Flange	Steel Frame Design	HE100B
STORY4	C11	Column	Steel I/Wide Flange	Steel Frame Design	HE100B
STORY4	C12	Column	Steel I/Wide Flange	Steel Frame Design	HE120B
STORY4	C13	Column	Steel I/Wide Flange	Steel Frame Design	HE100B
STORY4	C14	Column	Steel I/Wide Flange	Steel Frame Design	HE120B
STORY4	C15	Column	Steel I/Wide Flange	Steel Frame Design	HE120B
STORY4	C16	Column	Steel I/Wide Flange	Steel Frame Design	HE100B

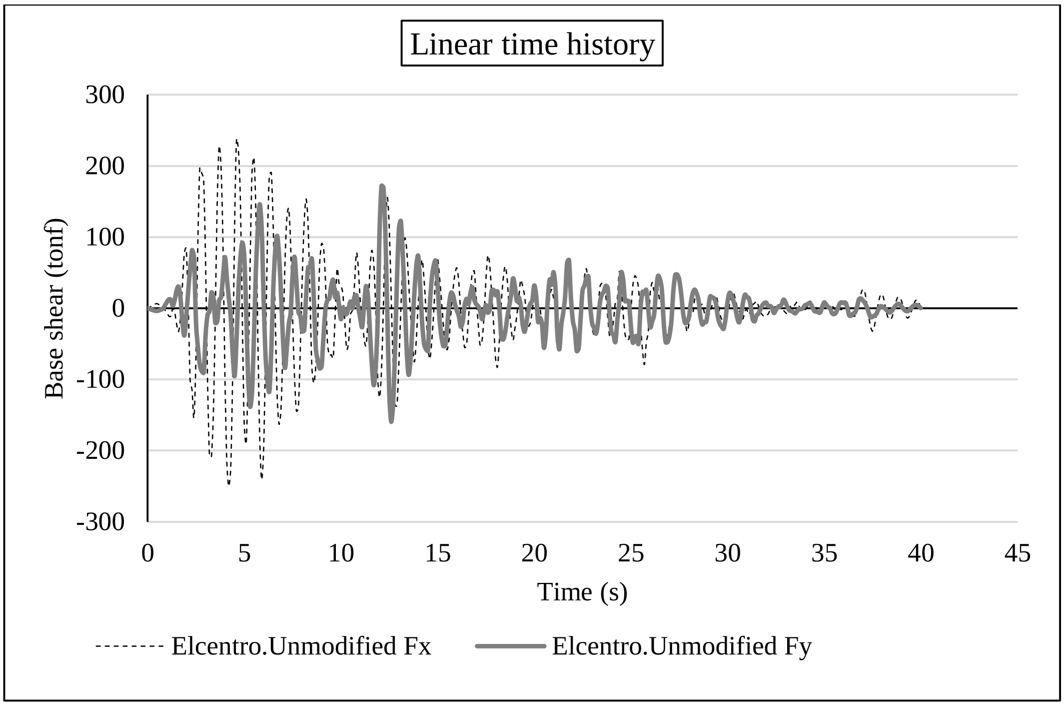

Figure A.1: History of unmodified accelerogram base shear of Elcentro earthquake in linear analysis

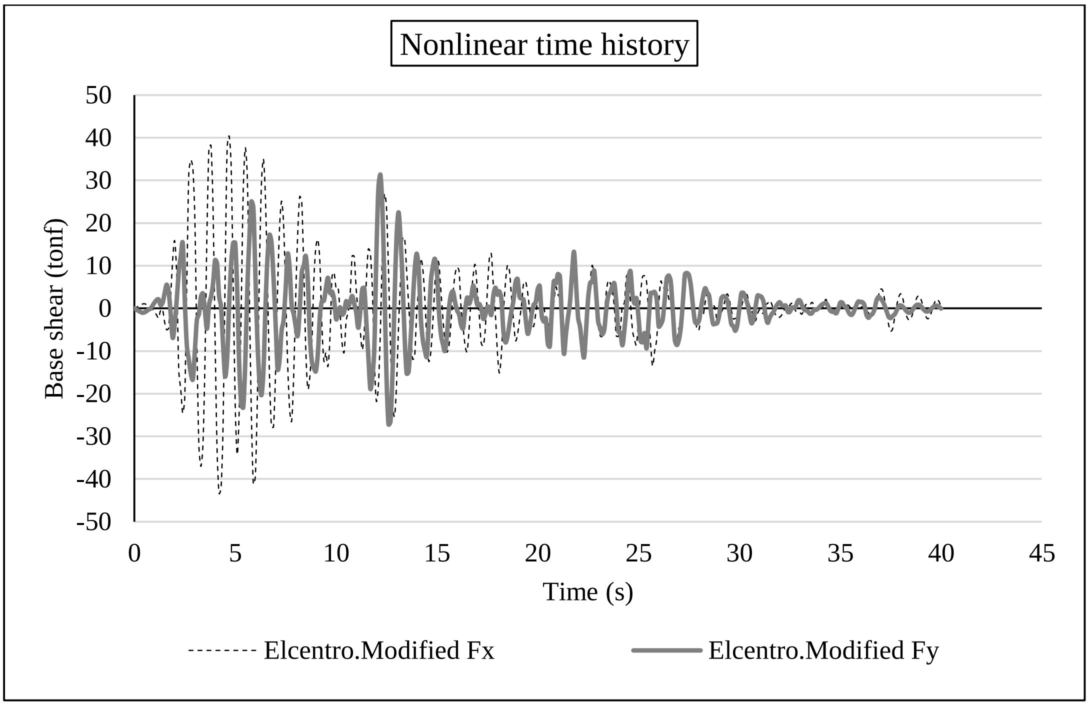

Figure A.2: History of modified accelerogram base shear of Elcentro earthquake in nonlinear analysis

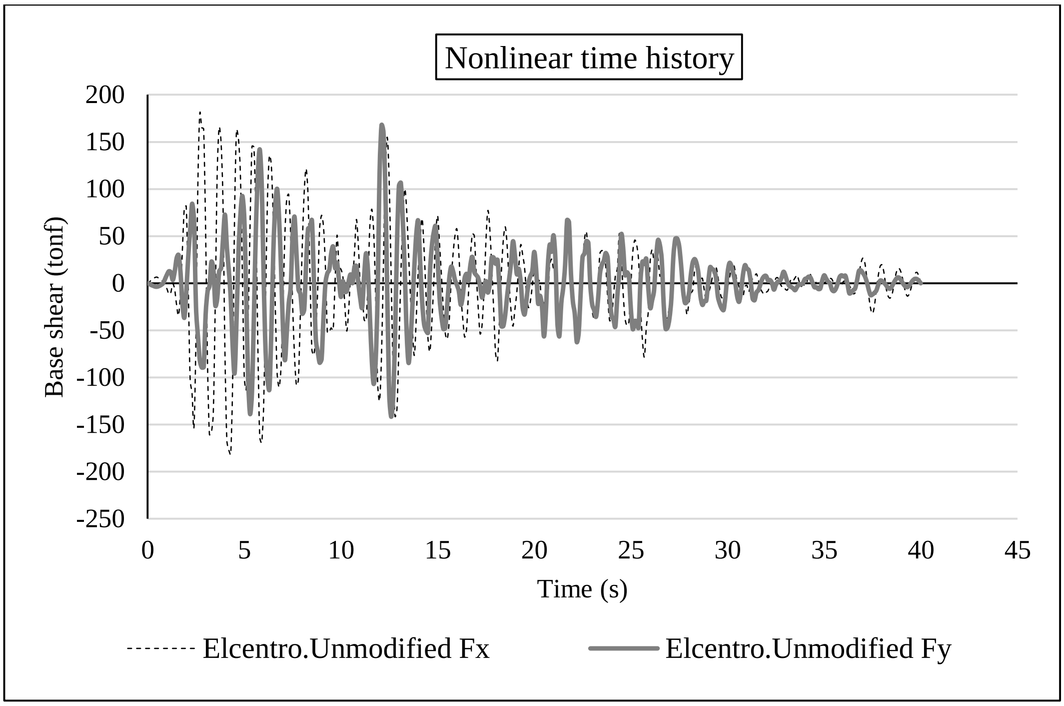

Figure A.3: History of unmodified accelerogram base shear of Elcentro earthquake in nonlinear analysis

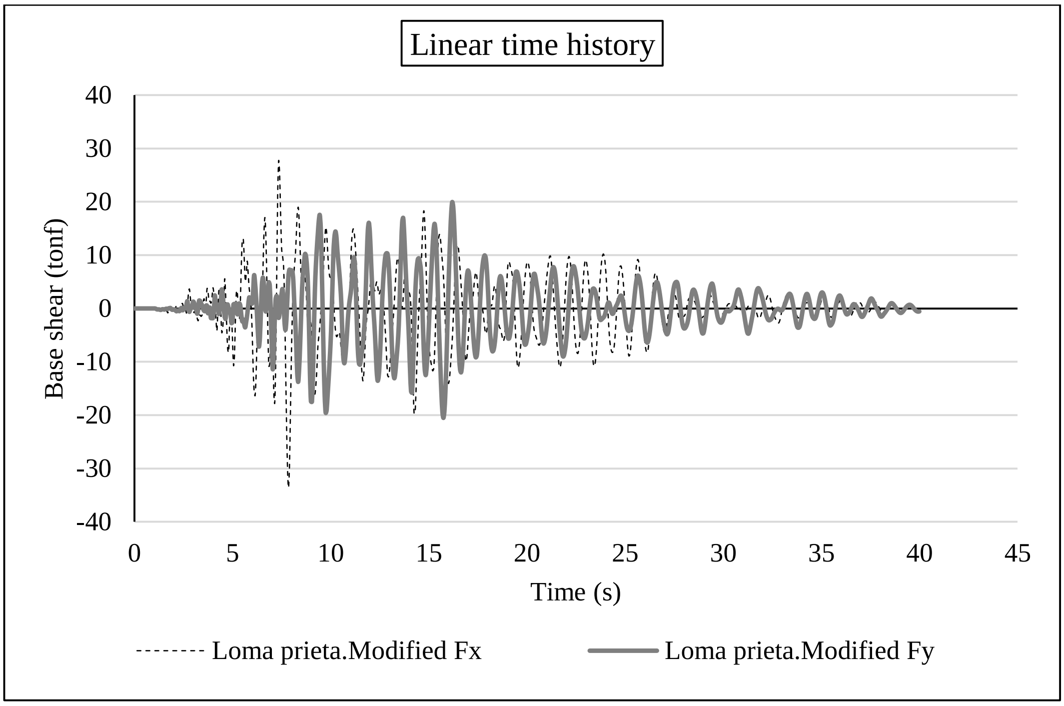

Figure A.4: History of modified accelerogram base shear of Loma Prieta earthquake in linear analysis

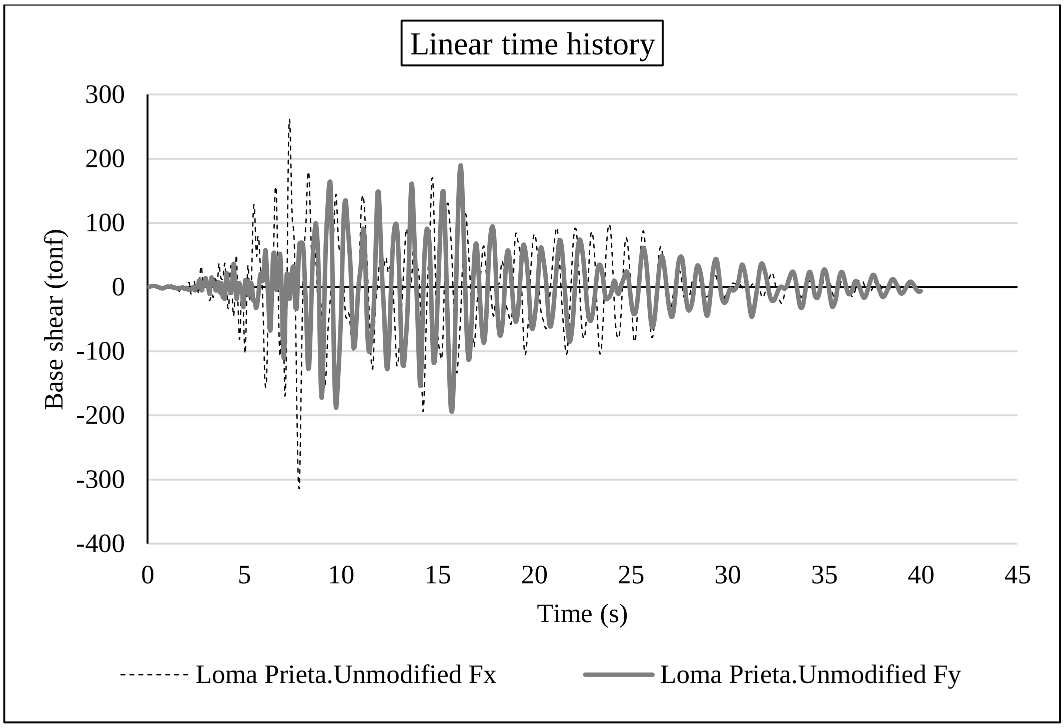

Figure A.5: History of unmodified accelerogram base shear of Loma Prieta earthquake in linear analysis

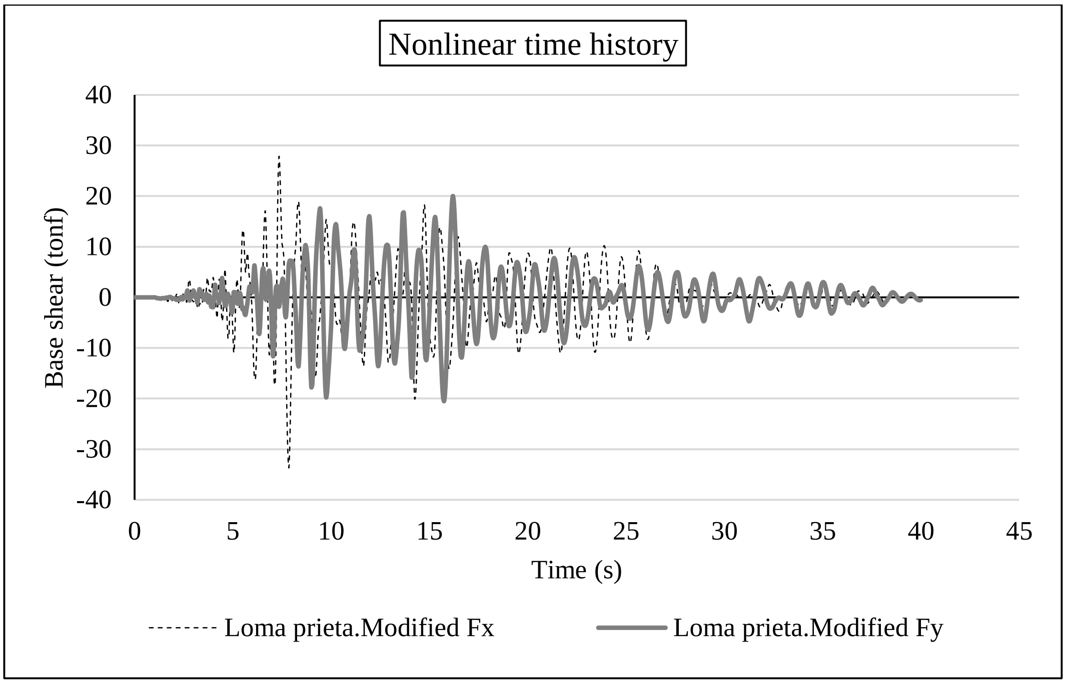

Figure A.6: History of modified accelerogram base shear of Loma Prieta earthquake in nonlinear analysis

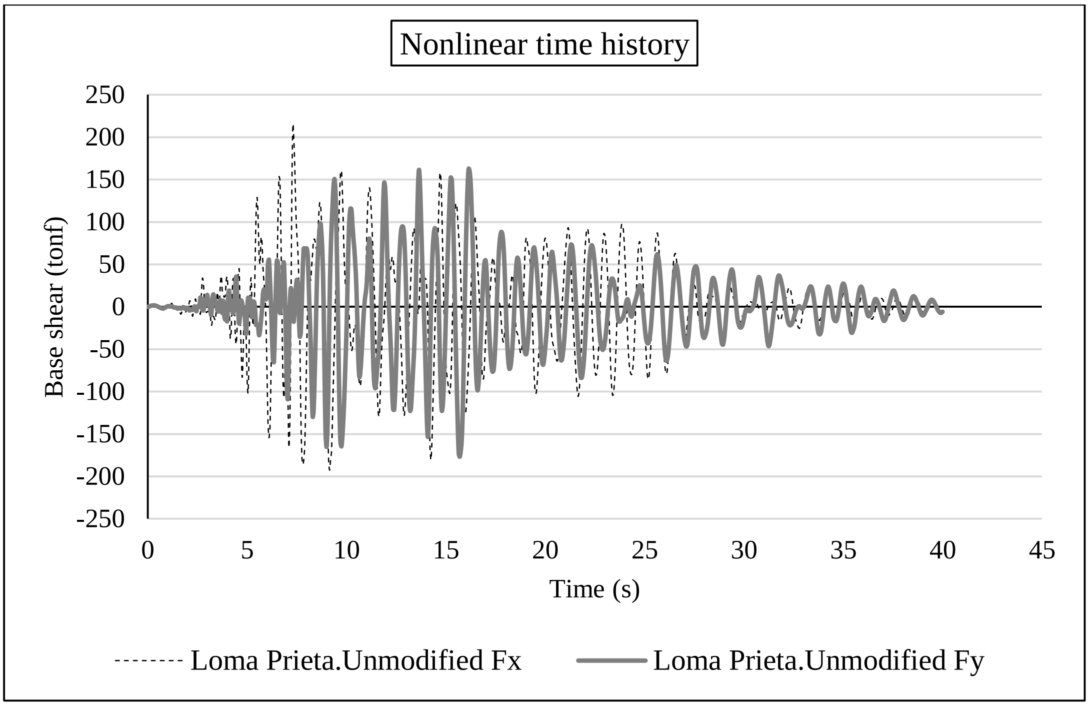

Figure A.7: History of unmodified accelerogram base shear of Loma Prieta earthquake in nonlinear analysis

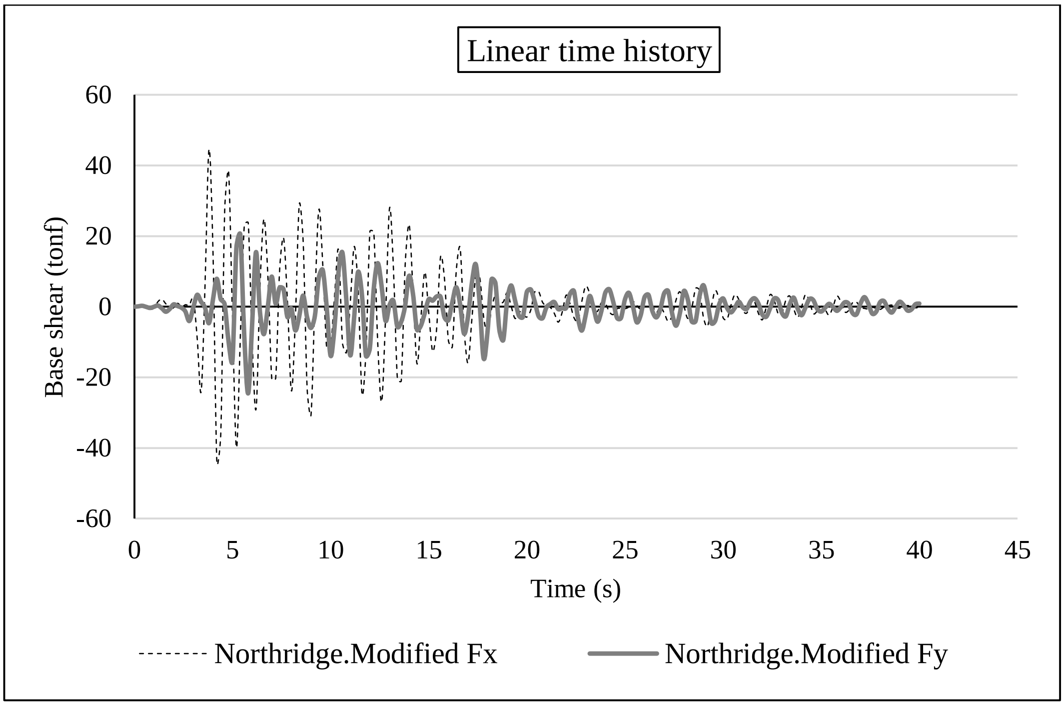

Figure A.8: History of modified accelerogram base shear of Northridge earthquake in linear analysis

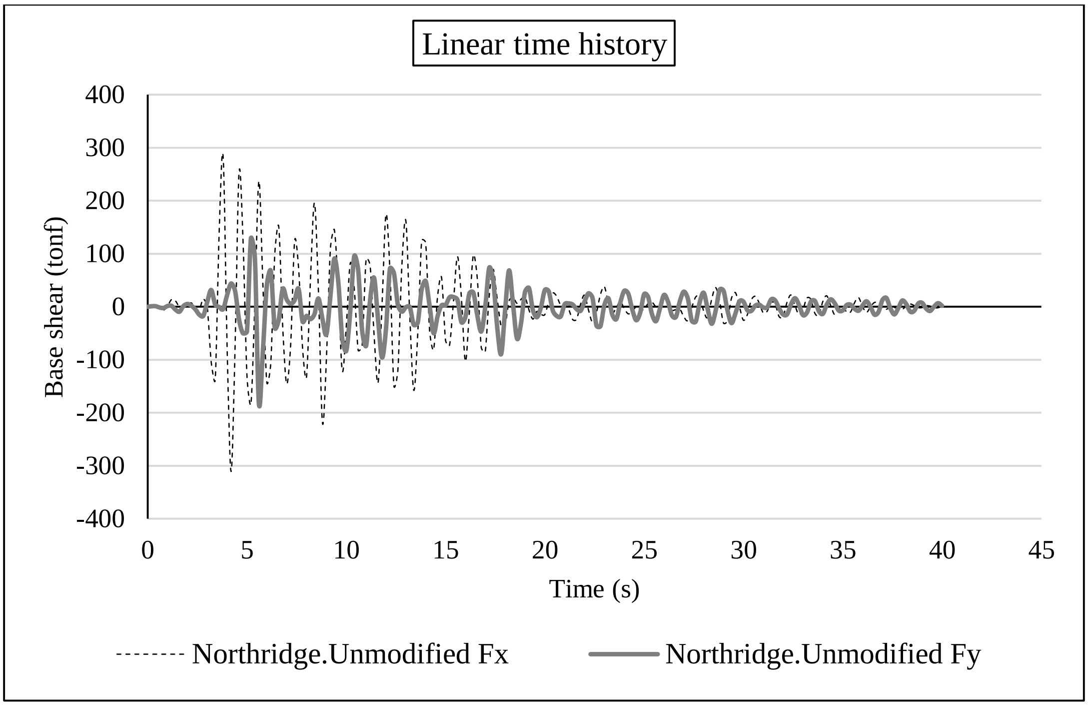

Figure A.9: History of unmodified accelerogram base shear of Northridge earthquake in linear analysis

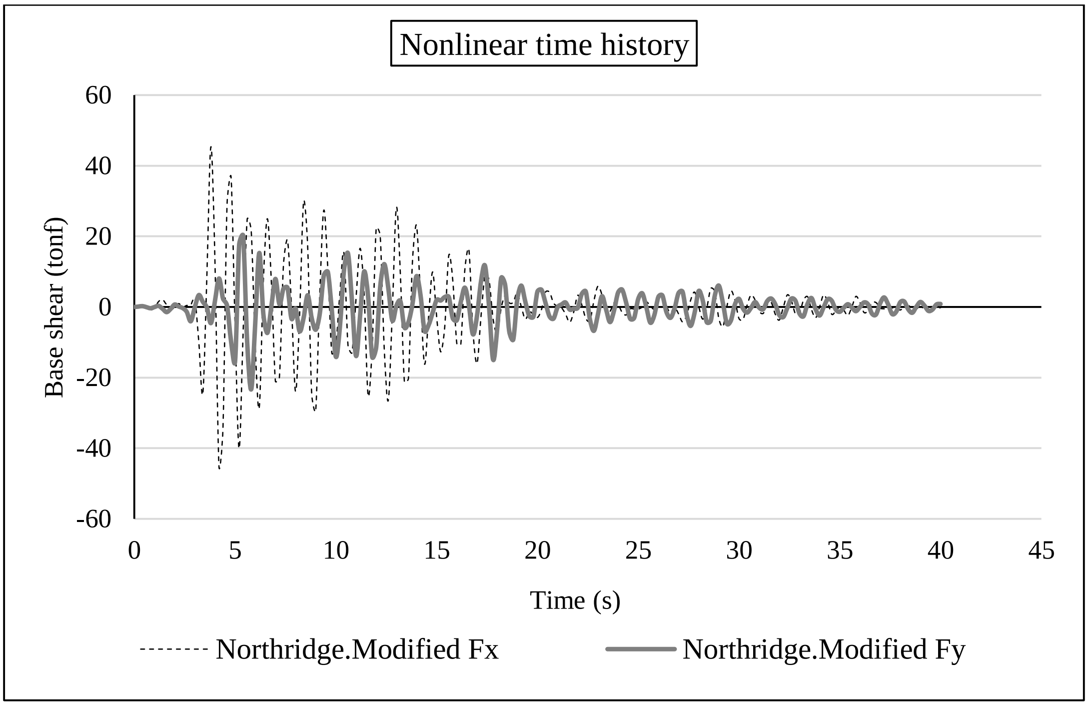

Figure A.10: History of modified accelerogram base shear of Northridge earthquake in nonlinear analysis

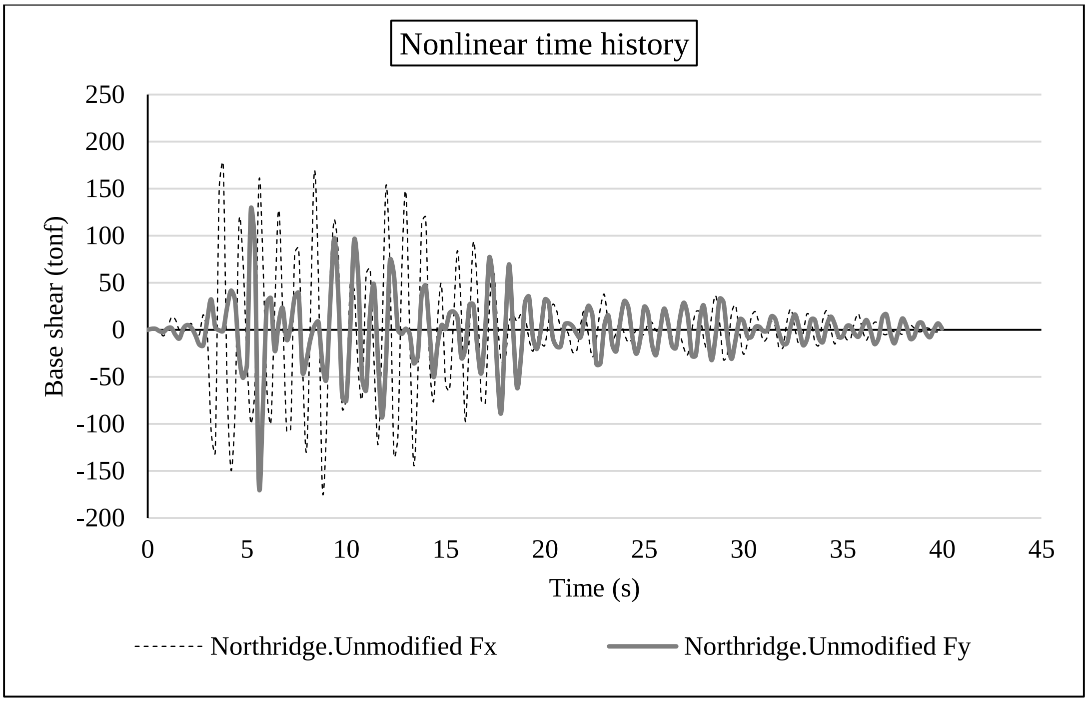

Figure A.11: History of unmodified accelerogram base shear of Northridge earthquake in nonlinear analysis