Inelastic Behavior of Eccentric Braces in Steel Structure

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ABSTRACT

The performance of eccentric braces is to some extent considered as a new subject amongst Civil Engineers. In general, braces are the members that resist against lateral forces in a steel structure while the structures are under seismic excitation. Although the height of a structure and the structural system are the two parameters which can affect the inelastic behavior and response of the structure but these parameters have not been taken into consideration in the current design codes for designing of Eccentric Braced Frames (EBFs).

In this study nine frames were exerted which were braced with three different eccentric braces (V, Inverted-V and Diagonal) in three different heights (4, 8 and 12 story).Then the frames were assessed by nonlinear static (pushover) analysis mainly based on FEMA 440 (2005). As a result of these frame analysis, it can be observed that the plastic hinges firstly occur at the fuse section of braces and then at the compressive members of the eccentric braces. Regarding the analyses conducted on the 4 story frames it can be inferred that eccentric Diagonal braced frames have better performance than the eccentric Inverted-V braced frames and in the same way the Inverted-V braced frames have better behavior than the eccentric V braced frames. This is also true for the 8_story and 12_ story models in this study. Furthermore, a comparison relevant to the total weight of the frames has been conducted among the above mentioned frames and this lead to the conclusion that using eccentric diagonal bracing system for the 12 story model will yield to the heaviest frame which is not economical in terms of frame

weight. But on the other hand using the eccentric diagonal braces for low and medium rise structures is more logical and acceptable from economical point of view as this type of bracing system absorbs considerably more energy when compared with eccentric V and Inverted V bracing systems.

Keywords: nonlinear static analysis, Eccentric diagonal Bracing system, Eccentric Inverted-V bracing system , Eccentric V bracing system , Inelastic Performance

ÖZ

Dış Merkezli Bağların davranışı, inşaat mühendisler arasında her zaman artışmalı bir konudur. Genellikle bağlar, deprem esnasında uygulanan yatay kuvvetlere karşı direniş gösteren elemanlard. Genelde binanın yüksekliği ve yapı sistemi doğrusal olmayan davranışı ve yapının tepkisini etkileyen iki parametredirler, ama bu iki parametre güncel tasarım standardlarında Dısş Merkezli Bağlar'ın (DMB) tasarımında henüz dikkate alınmamaktadırlar.

Bu çalışmada, üç türlu dış merkezli bağ braces (V, Inverted_V and Diagonal) üç değişik yükseklikte 3 kat, 4 kat, 8 kat ve 12 kat) 9 tane çerçeve kullanılmıştır. Sonra bu çerçevelerin davranışı FEMA 440 (2005)'a doğrultusunda doğrusal olmayan statik itme analizi ile.Dogrusal olmayan analiz (push over analizi) sonucunda elde edilen sonuçlar ilk plastik mafsallarin önce sigorta noktalarında ve daha sonra da Dış Merkezli Bağların basınç elemanlarda oluşmaktadır. 4 katlı çerçeveler üzerinde yapılan benzer analizler sonucu gözlenenler Diagonal dış merkezli bağların davranışının ters çevrilmiş dış merkezli V bağlantısından daha iyi olduğu ve yine ters çevrilmiş V bağlantısının da dış merkezli V bağlantısından daha iyi olduğudur. Bu söylenenler, sekiz ve oniki kat çerçeveler için de geçerlidir. Çerçevelerin ağırlıkları göz önünde bulundurularak farklı çerçevelerde yapılan analiz ve tasarım sonucu en ağır çerçevenin Dış merkezli diagonal bağların kullanıldığı 12 katlı çerçeve olduğu saptanmıştır. Diğer yandan dış merkezli diagonal bağların dış merkezli ters çevrilmiş V ve V bağlantılı alçak ve orta katlı yapılarda daha çok enerji yuttuğu ve daha hafif olduğu gözlemlenmiştir.

Anahtar kelemiler: dogrusal olmayan statik analiz, Dış merkezli diagonal bağ sistemleri, Dış merkezli ters-V bağ sistemi, Dış merkezli V bağ sistemi, doğrusal olmayan performans

To My Family

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INTRODUCTION

1.1 Background

1.1.1 Preface

Every year, many people die because of earthquakes around the world. 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. This issue has been one of the important subjects for researchers during the last three decades. Finally they came up with suggesting concentric, such as X, Diagonal and chevron, eccentric and knee bracing systems and these were used in real life projects by civil engineers for several decades.

One of the principal factors affecting the selection of bracing systems is inelastic 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.

1.1.2 Literature Review

During the recent decades, nonlinear response of bracing systems has been studied and consequently parameters such as, seismic behavior factor, *R*, over strength factor, *W*, and displacement amplification factor, C_d, were introduced to loading codes of practice like UBC (Uniform Building Code) and IBC (International Building Code). These design codes are widely used in the USA and also throughout the world. Design engineers consider the handling of the actual performance levels as a difficult process. Therefore, these parameters were introduced in the new design codes in order to take the inelastic behavior of the bracing systems into account. In the process of the earthquake load calculation of a structure, seismic behavior factor is the parameter illustrating the impact of nonlinear performance of the bracing system that is fundamentally affected by the system ductility. The efficiency of bracing systems is influenced by these key parameters because they directly affect the reduction of the earthquake loads in the structure. In accordance with the loading codes, specific *R*, *W* and C_d factors were introduced for various structural systems (illustrating the distinction of their nonlinear behavior), such as concrete moment frame and steel moment frame with high, medium and low ductility, steel frames with concrete shear walls and steel braced frames.

The earthquake load applied on the structure is estimated by the equation written below**:**

$$C = \frac{A.B.I}{R} \qquad (1.1)$$

(Where A, B and I represent the values for site seismicity, soil type and importance factor of the structure, respectively)

The procedures of structural and seismic engineering have gone through great alterations since last decades. Changing the codes of practice and suggesting the new reports from Federal Emergency Management Agency (FEMA) manifest some of these changes. Despite the fact that the current design codes are based on the recent research findings, the fast improvement in nonlinear structural analysis procedures resulted in demand for more research based on the current analysis processes for the purpose of assessing the nonlinear behavior of structural systems.

1.2 Objectives of the Study

This study aims to do quantitative comparison amongst ductility levels of disparate steel bracing systems and compare the outcomes from the economical point of view by using mainly the most recent research findings in the field of nonlinear structural analysis. Simultaneously, by zeroing in on both weight and performance of the bracing systems, this study expresses a more realistic comparison among them.

1.3 Reasons for the Objectives

All of the steel braced framed structures 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 which are supposed to be used, especially the type of the 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 nonlinear performance of various structural systems engenders higher quality in their design.

1.4 Guide to the Thesis

This study is comprised of five chapters. Chapter two includes literature review, being divided into four sections. The first section (section 2.1) is devoted to a short background about pushover analysis. Section 2.2 explains about shape and geometry impact of frame on the total performance of frames. In section 2.3 evaluation of nonlinear static procedure is described then finally in section 2.4 a background on frame analysis is mentioned.

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. This chapter is divided into four sections. Design results are given in section 4.1., the results of the pushover analysis and its results are explained in chapter 4.2. The idealization of pushover analysis curvatures and the related information are detailed in section 4.3 and finally the assessment of the different bracing systems is given in section 4.4. Then the conclusion of thesis is in chapter 5.

Chapter 2

LITERATURE REVIEW

2. 1. A Short Background about Pushover Analysis

Nonlinear static pushover analysis procedure has been introduced to the Civil Engineering society and used simultaneously by the advent of designing on the basis of performance. A simple explanation of the Pushover analysis is: performing a static, nonlinear process in which the amount of the structural loading is boosted incrementally in accordance with a specific predefined pattern. Feeble links and failure modes of the structure can be figured out, while the amount of loading increases. The loading is continuous with and compatible to the outcome of the cyclic performance and behavior and also load reversals that are being figured out with the help of implementing altered continuous force-deformation provisions and with damping approximations.

Static pushover analysis can be defined as an effort by the structural engineering profession to assess the actual strength of the structure while it guarantees being as an effective and useful method for designing on the basis of performance . For Pushover analysis there are modeling processes, procedures of analysis and also acceptance provisions that are detailed in the ATC-40 and FEMA-273 documents. Force-deformation provisions are illustrated and explained for hinges that are introduced in pushover analysis.

As it can be observed from figure 1 below, in order to define the force deflection performance of the hinge, there are five points that have been designated and named as: A, B, C, D, E and also three points which are called as IO, LS and CP and identify the acceptance criteria of the hinge. The latter three points (IO, LS and CP) stand for Immediate Occupancy, Life Safety and Collapse Prevention respectively. The values assigned to each of these points vary depending on the type of member in use and also on the basis of many other parameters and elements which are explained in the ATC-40 and FEMA-273 documents. A variety of values are assigned to each of these points.

The steps used to perform a pushover analysis of a common three-dimensional building are described by Ashraf Habibullah, S.E.1, and Stephen Pyle, S.E.2, (Published in the Structures Magazine, Winter, 1998).

SAP2000 (a state-of-the-art, general purpose) or ETABS software, three-dimensional structural analysis program, is adopted as a medium of performing the pushover analysis. SAP2000 have the ability and potential to carry out nonlinear static pushover analysis as described in ATC-40 and FEMA-273 documents, for both two and three-dimensional structures. In figure 2.1 all plastic hinge stages are shown.

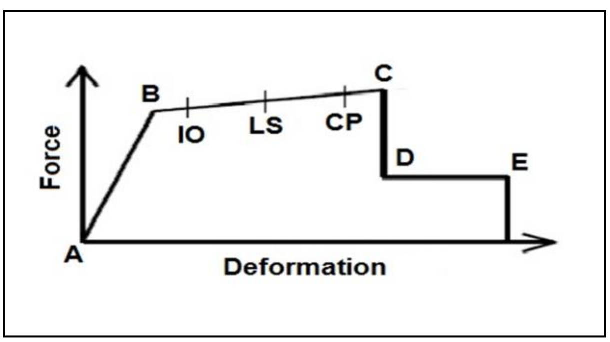

Figure 2.1: Force _Deformation for pushover hinge

2.1.1 Introduction to Inelastic Time History and Static Pushover Analysis

Inelastic time–history analysis can be considered as an effective approach for studying the structural responses to seismic forces. A set of meticulously opted ground motion records can lead to an exact assessment of the predicted seismic performance of structures. Furthermore, the authenticity and efficiency of the computational approaches have elevated significantly, there are still some uncertainty and doubt about the dynamic inelastic analysis method, that are basically relevant to its abstruse essence and being proper for pragmatic design applications. Furthermore, the calculated inelastic dynamic response is totally susceptible to the traits of the input information relevant to motions. As a result, choosing a suitable acceleration time–history is compulsory. This substantially causes the computational efforts to dramatically increase skyrocket. The inelastic static pushover analysis is a simple option for finding out the strength capacity in post-elastic range. This approach might be exerted in order to identify the probable weak areas in the structure.

This method is associated with implementing a predefined lateral load pattern that affects the building throughout its height. Then the lateral forces continuously are magnified in fixed ratio with a displacement control which is at the top of the building till they reach to a specific level of deflection. The target top displacement may be the deformation expected in the design earthquake in the case of designing a new structure, or the drift corresponding to structural collapse for assessment purposes. This approach allows tracking of the order of yielding and failure on the member, the structure levels and also the progress of the total capacity curve of a typical structure.

Over the past twenty years the static pushover procedure has been investigated and strengthened mainly by Saiidi and Sozen (1981) , Fajfar and Gaspersic (1996) and Bracci et al. (1997). This method is also explained and introduced as an approach for the purpose of designing and evaluating by the National Earthquake Hazard Reduction Program 'NEHRP' (FEMA 273) (1996), guidelines for the seismic rehabilitation of existing buildings. Furthermore, the so called method is taken into consideration by the Structural Engineers Association of California 'SEAOC' (Vision 2000), (1995), among the other analysis procedures with various level of complexity.

This analysis procedure is usually chosen because of it being applicable to performance-based seismic design methods and also it can be applied to different design levels for the determination of the performance targets. At last, it can be concluded from the recent discussions in code-drafting committees of Europe that this method is likely to be introduced in future codes.

2.1.2 Comparison between Inelastic Static Pushover and Inelastic Dynamic Analyses

As a result of the inelastic static pushover analysis simplicity in comparison with the inelastic dynamic analysis, investigation about inelastic static pushover method has become the subject of various scientists during recent years, (A.M. Mwafy, A.S. Elnashai),(2000). The veracity and the plausibility of putting inelastic pushover analysis into practice are evaluated by comparing it with 'dynamic pushover' idealized envelopes obtained from the incremental dynamic collapse analysis in a research (A.M. Mwafy, A.S. Elnashai), (2000). This is conducted by exerting natural and synthetic seismic and earthquake records used with 12 reinforced concrete (RC) structures as specimens with various characteristics by A.M. Mwafy, A.S. Elnashai, (2000). This involved intermittent scaling and implementation of each accelerogram followed by the evaluation of the utmost response, till the collapse occurrence in the structure.

The outcomes of more than one hundred inelastic dynamic analyses using a detailed 2D modeling method for each of the twelve RC buildings have been used to extend the dynamic pushover envelopes and also make a comparison amongst these with the static pushover results which have distinct patterns of loading. The study was conducted by A.M. Mwafy, A.S. Elnashai, (2000). Fine and proper correlation was acquired between the calculated idealized envelopes of the dynamic analyses and static pushover results of a designated and defined range of building. Furthermore comprehensive investigations in accordance with Fourier amplitude analysis of the response were conducted and as a result conservative assumptions were emphasized, when variances became explicit.

2.1.3 Results of the Comparison between Static Pushover and Dynamic Analyses Conducted by Mwafy and Elnashai 2000

1. In accordance with the structural modeling, prudent opting of the lateral loading distribution and finally a vivid interpretation of the outcomes, pushover analysis is suitable to provide insight into the elastic alongside the inelastic response of structures when subjected to ground motions of the earthquake.

2. For the framed structures that are low rise and have short period, it is more suitable to use static push over analysis. A well-designed building with structural irregularities is taken into consideration. Therefore the results of the procedure also illustrate fine correlation with the dynamic analysis. In the study conducted by (A.M. Mwafy, A.S. Elnashai), (2000), for a group of four 8-storey irregular frame buildings using an

inverted triangular lateral was used and the responses from the structure were identical to Inelastic time–history analysis.

3. The results obtained from previous studies can guide us to eliminate the variances amongst the two approaches i.e. static and dynamic analysis results for the structures that are special and having long seismic period. The confined and constrained capability of the fixed load distribution was used to find out the effects of higher mode in the post-elastic range is substantially the cause of these discrepancies. To overcome this problem and to guarantee the exact or trivial conservative anticipation of demands and capacities, more than one load pattern must be used.

4. The research was conducted on two sets of four 12- story frame structures and four 8-storey frame-wall structures illustrates that a conservative anticipation of capacity and a plausible estimation of deflection is acquainted by using the simple triangular or the multimodal loading distribution. The demand of some of the structures in the elastic range is trivially underestimated with the identical loading patterns. Contrary to this, a conservative anticipation of seismic demands in the range prior to collapse occurrence is provided by the uniform loading. Also just at the collapse limit state, it results in a plausible estimation of shear demands at the collapse limit state.

5. Comparison amongst the triangular and the multimodal distribution outcomes illustrate variances less than 4%, for the twelve story buildings, as the former acquaints the traits of the most significant mode of vibration. The load distribution from multimodal analysis merely conveys the distribution of inertia forces in the elastic range;

therefore effects of higher mode are not completely substantiated and authenticated in the post-elastic scope.

6. The extension in the basic period of building as a result of great cracking and yielding during earthquakes is dependent on the total stiffness of the structural system of the building. In this study, the noticed elongation ranges between 60% and 100% for the stiff frame-wall structural system and the most flexible irregular frame system respectively. Therefore, exertion of elastic periods in seismic code does not bode identical and constant levels of safety for various structural systems.

7. The outcomes of the dynamic collapse analysis illustrates that each earthquake record manifests its own characteristics and traits, imposed by the content frequency, duration, sequence of peaks and peaks' amplitude. The straggling and diverse results of various ground motions are depended on the traits and essence of both the structure and record. An effective method for studying the noticed variability of the outcomes and to recognize the extended inelastic periods of the structure is the Fourier spectral analysis.

2.2 Shape and Geometry Impact of Frame on the Total Performance of Frames under Earthquake Excitation:

2.2.1 Introduction

Amongst various kinds of loads, the most naturally uncertain one is the earthquake load. Earthquake develops and happens beneath the earth's crust as a result of the plate tectonics movement and this is why it is difficult to know the exactly time of occurrence and also the power and magnitude of energy released by it. Unlike the other kinds and forms of loads applied to the structures earthquake loads cannot be foreseen accurately. Besides the earthquake loads that are unpredictable, the responses of the structures which are in essence dynamic are also unforeseeable. The problems of relating to inaccuracy of structural responses are associated with and rooted in many variables and factors, such as: the materials used in the building, the geometry of the structure, the properties of soil in which the structure is erected, how and where the construction is built, the size and kind of frequency of ground motion, epicenter of earthquake, focal depth of the earthquake, etc. On the basis of the unpredictable essence and character of seismic loads, making a trustable skeleton or guideline in earthquake engineering has always been demanding task in front of civil engineers and those involved with structural analysis.

As a result of the efforts to find a system to resist the lateral forces in structures, the structural steel framing system has been evolved. Soon after 1906, it became clear that the performance of steel framed buildings is better in comparison with the masonry structures (FEMA-355e). There was no provision designated for earthquake loading in the building codes until the occurrence of San Francisco earthquake in that year. Later on, when the Santa Barbara earthquake happened in 1925, Uniform Building Code (UBC) became the first code which included the seismic provision in 1927. 32 years after that the Structural Engineers Association of California (SEAOC) published "lateral force recommendation" document. In 1961, UBC adopted it too. The demand for the steel moment resisting frame for tall buildings over 160ft was considered as the most important feature. Then American Institute of Steel Construction (AISC) in 1992 included seismic provisions in its specifications.

In accordance with each major earthquake the building code is to be modified (FEMA-355e). Until that time, the steel moment resisting frames were considered to dissipate and damp the earthquake energy properly and adequately because of the moment resisting connections which were presumed to behave ductile. In contrast with the first assumption, the Northridge earthquake in 1994 repudiated it, when many failures happened in beam-column connections because of a brittle behavior. This type of brittle failure causes little observable damage that is yielded by this kind of brittle failures and poses concerns about damages which were undiscovered in the past earthquakes. After Northridge earthquake, investigations have testified such type of damages in some of the buildings which were subjected to Loma Prieta (1989), Landers (1992), and Big Bear (1992) earthquake (FEMA-355f).

In September 2000, the FEMA-355f was published by Federal Emergency Management Agency which was prepared by the SAC joint venture, while it demonstrated and explained the prediction of performance and assessing approach of moment resisting frames as well as the processes of analysis and the seismic hazard status. It is noticeable that before the specifications of 1976, no kind of limitations had been designated for the seismic design in terms of lateral drift. One of the salient features of the so called process was the determination of the capacity and demand on the basis of story drift.

FEMA-356 categorized some typical drift measures to illustrate the total structural response. Collapse prevention level is assigned to a steel moment frame which includes 5% transient or permanent drift. For next level that is called life safety it splits into two groups which is 2.5% for transient and 1% permanent drift; for the consequent level that is immediate occupancy it was 0.7% transient in which permanent drift was negligible. But for steel frames that are braced, these drift measures for the so called levels that are in order as : (collapse prevention, life safety and immediate occupancy level), can be cited as 2% transient or permanent, 1.5% transient and 0.5% permanent; 0.5% transient with negligible permanent respectively.

Meanwhile it should be mentioned that these values were not requirements for drift limits. Some drift limitations and confinements were imposed Vision 2000_SEAOC to steel moment frames as 2.5% transient or permanent drift for collapse prevention, 1.5% permanent and 0.5% transient for life safety level and 0.5% transient with no permanent drift for operational level. As well, the requirements of connection in accordance with the Seismic Provisions of AISC 2005 demands that the beam to column connections should have the ability to bear minimum 0.04 radians of inter story drift angle.

2.2.2. Background of Moment Frames (MFs):

Moment resisting frames are described as the joining beams and columns in a rectangular shape. In this joining, the beams and columns should be spliced rigidly. The reason of resistance against lateral loads which accounts for the consequent shear forces and bending moments through frame members and joints is the rigidity of the frame behavior. It is impossible for a moment frame to move horizontally without deforming the beams and columns as a result of the beam-column connections rigidity. The lateral stiffness and strength of the entire structure is generated by the strength of the frames and bending rigidity. In the regions which have high seismicity steel moment resisting frames have been used much more in comparison to the other kinds of framing systems, because of many reasons. Steel moment resisting system is recognized as a system with a very high capacity of ductility amongst all the other structural systems. Meanwhile, there are large reduction factors which have been designated for designing seismic forces in codes of building.

No bracing members are present to block the wall openings which provide architectural versatility for space utilization. But, compared to other braced systems moment frames generally required larger member sizes than those required only for strength alone to keep the lateral deflection within code approved drift limits. Again, the inherent flexibility of the system may introduce drift-induced nonstructural damage under earthquake excitation than with other stiffer braced systems. When steel moment frames could not behave in a way that was expected after the occurrence of 1994 Northridge earthquake, even these concepts relevant to the expected performance of steel moment frames in energy dissipation under lateral loads was sacrificed. The assumption that the system is high in ductility is challenged by the brittle failures occurring at beam to column connection (Michel Bruneau et al. 1998).

Beam, column, and panel zone are the components of the moment frames. Panel zone is the part of the column included within the joint area of beam to column connection. In traditional analysis, moment frames were used to be modeled with nodes without dimensions, these nodes were as a matter of fact the intersection of beam to column members. The so called models are not taking panel zone into consideration. On the other hand in ductile moment frames, panel zone are clearly considered. Depending on the yield strength and the yield thresholds, the beam, column and even panel zone could contribute to the total plastic deformation at the joint. A structural component considerably weaker than the other framing into the joint will have to provide the needed plastic energy dissipation. Those structural components expected to dissipate hysteretic energy during an earthquake must be detailed to allow the development of large plastic rotations. Plastic rotation demand is typically obtained by inelastic response history analysis. Without considering panel zone plastic deformations it was expected that the largest plastic rotations in the beams are 0.02 radian (Tsai 1988, Popov and Tsai 1989).

After the Northridge earthquake the required connection plastic rotation capacity was increased to 0.03 radian for new construction and for post-earthquake modification of existing building it was 0.025 radian (SAC1995b).

2.2.3 Pre-Northridge Design

The supposed ductility of the mentioned framing systems is on the basis of connection properties. Prior to Northridge earthquake moment, connections which were spliced by welding, were prevalent and common in practice through high seismic areas of North America. An alternative connection came into existence. In 1960s that was highly used in practice by construction industry, the so-called connection had a bolted web connection and flanges were completely welded. In order to compare and assess the nonlinear and plastic behavior of mentioned moment connections a number of tests were conducted by different people. The first one was made in 1960s. Popov and Pinkney (1969) worked on 24 beam column joints in their test. Those specimens which had welded flanges and bolted connections in comparison with the specimens with cover plated moment connection and completely bolted moment connection resulted in much better nonlinear behavior, as a result of slippage of the bolts that engenders visible pinching in the hysteresis loops under cyclic loading (FEMA-355e) (Michel Bruneau et al. 1998).

During 1970s, welded flanges-bolted web connections with completely welded connections were compared (Popov and Stephen 1970) and as a result the completed welded connection showed more ductile behavior. But in bolted webs four out of five failed suddenly. Popov and Stephen (1972) as well inferred that "The quality of workmanship and inspection is exceedingly important for the achievement of best results (Michel Bruneau et al. 1998)".

Popov et al. (1985) examined and worked on eight specimens. The tests were exclusively focused on the behavior of panel zone with W18 beams. As per the authors, during the welding procedure: "the back-up plates for the welds on the beam flange-to-column flange connections were removed after the full-penetration flange welding was completed and small cosmetic welds appeared to have been added and ground off on the underside (FEMA-355e)."

Tsai and Popov (1987) and Tsai and Popov (1988) conducted their own tests in which they illustrated some prequalified moment connections in ductile moment frames with W18 and W21 identical in concept to those examined by Popov and Stephen (1971), but did not have the ductility that was expected. Prior to development of sufficient plastic rotations, specimens that had welded flanges-bolted web connections would fail suddenly. Only four out of eight specimens achieve desirable beam plastic rotation. Authors realized that the quality control is an important factor (FEMA- 355e).

In order to examine the effect of the ratio of Z_f /Z on rotation capacity using W21 and W24 beams, where Z_f is the plastic modulus of the beam flanges and Z is the plastic modulus of the entire beam, Engelhardt and Husain (1993) performed 8 tests . They were not able to identify any relation between Z_f /Z with amount of hysteretic behavior developed before the failure occurrence. On the other hand, interestingly a couple of cases under this study resulted in ductility deficiency. Also past experimental information were compared with the result of this study. In accordance with the assumption that connections must have a beam plastic rotation capacity of 0.015 radian in order to endure and tolerate under severe earthquake, they came up with the fact that no single specimen was able to satiate that amount of rotation (Michel Bruneau et al. 1998).

Prior to the Northridge earthquake occurrence most of the beam-column connections in the moment resisting frame were defined to be able to convey plastic moment of the beams to the columns (Roeder and Foutch 1995). As a result, for the purpose of resistance against seismic forces, relatively lighter column and beam sizes were enough. Since then many engineers came up with the notion that it was economically advantageous to confine the number of bays in a frame that is designed as ductile moment frame. Before the Northridge earthquake occurrence even some engineers repeatedly designed structures with merely four single-bay ductile moment frames which had two in per main direction. This approach led to the loss in redundancy of the structure. As well as these single-bay moment frames demanded substantially deeper beams and columns with thicker flanges than the multi-bay ones exerted before in order to resist the identical lateral forces. This brought an opportunity to probe about potential size effects (Michel Bruneau et al. 1998). Roeder and Foutch (1996) consolidated all the results of test relevant to prenorthridge connections and came up with the tenet which illustrates that expected ductility subsides with the deeper sections. Bonowitz (1999a) reached to the identical results from the tests executed after Northridge earthquake (FEMA-355e).

2.2.4 Post-Northridge Design

Various criteria and elements have been recognized that were potentially resulted in the weak seismic behavior of steel moment connections relevant to the pre-Northridge. The failure was due to the combination of the following: workmanship and inspection quality; weld design; fracture mechanics; base metal elevated yield stress; welds stress condition; stress concentrations; effect of triaxial stress conditions; loading rate; and presence of composite floor slab (Michel Bruneau et al. 1998).

Many different solutions have been suggested to the problems of moment frame connection. Two important strategies have been developed to overcome the shackles. The solution is as follows: firstly strengthening the connection and secondly by weakening the beam ends that are framed into the connection. Both of the two strategies are able to move away the plastic hinges from the column face soundly (Michel Bruneau et al. 1998).

A performance that is satisfactory engenders a connection to be able to promote a beam plastic rotation of 0.03 radian with a minimum strength equal to 80 percent of the plastic strength of the girder. In SAC interim guidelines, these confinements can be found as the posed acceptance criteria (1995b). As the minimum requirement it was recommended that tentative authenticity of suggested connection to be done with the qualification testing in agreement with the ATC-24 loading agenda (ATC1992).

2.2.5 Semi –Rigid Connection

A connection in a moment frame will termed as partially restrained if it contributes to a minimum 10% of the lateral deflection or the connections strength is less than the weaker element of connected members (FEMA-356). It is presumed that in the Northridge earthquake, partially restrained connections could result in a better performance to provide flexibility in the structure. Proper placing of semi-rigid connections along with the rigid connection could improve the performance of moment frames.

Kasai et al. (1999) and Maison et al. (2000) studied the effect of semi-rigid connections within the SAC program. But, in those studies all the connections were considered as partially restrained (FEMA-355c). However, the knowledge about the effect of semi-rigid connections in a hybrid frame is limited. Built on the pioneer work of Radulova (2009), the study conducted by (S. M. ASHFAQUL HOQ), (2010), aims to study the seismic performance of fully rigid and hybrid rhombus and rectangular framing systems.

2.2.6 Background of Concentrically Braced Frames (CBFs)

The concentrically braced frame is a lateral force resisting system. This brace frame system is an efficient one that is known by its high elastic stiffness that is generally used to resist wind or earthquake loadings, figure 2 illustrates some kinds of CBFs. With the help of its diagonal bracing members that resist lateral forces, by utilizing higher internal axial actions and relatively lower flexural actions, the system is able to reach high stiffness. It is Diagonal bracing which forms the main units that generate the lateral stiffness. Braces may be made of different shapes and sections such as: I-shaped sections, circular or rectangular tubes, double angle attached together to form a T-shaped section, solid T-shaped sections, single angles, channels and tension only rods and angles. Generally the members of braces are joined to the other members of the framing system by gusset plates which are bolted or welded. The focus in CBF designing approach generally is on energy dissipation in the braces so that in accordance with the designing, the connection remains elastic at all stages during load administration. To maximize the energy dissipation, the braced connections must be designed to be stronger than the bracing members that, they are connected to, in order to make the maximum amount of energy dissipation, so that the bracing member can yield and buckle (Michel Bruneau et al. 1998). The typical CBFs are shown in figure 2.2 below.

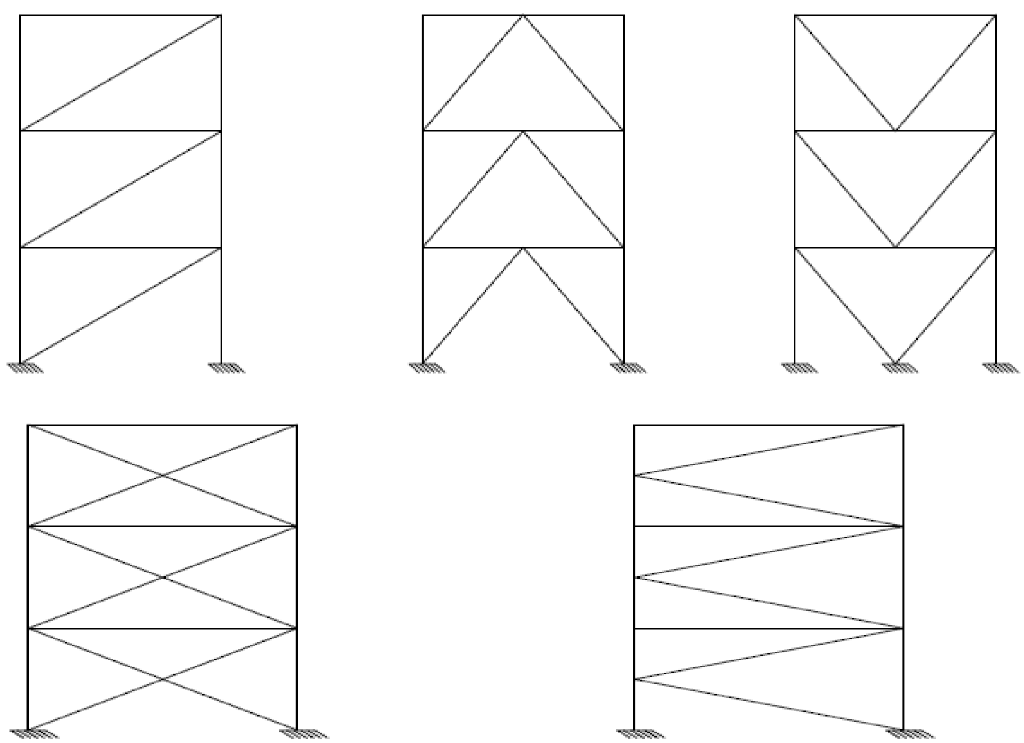

Figure 2.2: Typical CBF (Diagonal, Inverted _V, V,   Chevron and Knee bracing system) Configurations

Due to failure of the bracing members under large cyclic displacements CBF bracing systems are known to be less ductile seismic resistant structure when compared to other systems. These structures are apt to bear and endure large story drift after buckling of bracing members, which in turn may cause the fracture of bracing members. Recent analytical studies have illustrated that CBF bracing system that are designed by obsolete elastic design approach can undergo severe damage, under design level ground motions (Sabelli, 2000). Current seismic codes (ANSI, 2005a) have provisions to design ductile CBF that is also known as Special Concentrically Braced Frames (SCBFs).

2.2.7 Background of Eccentrically Braced Frames (EBFs)

The eccentrically braced frame is a combination of concentrically braced frame and moment resisting frame. The EBF accumulates specific positive characters of each frame and mitigates their respective disadvantages. The results of using EBF system are bringing high elastic stiffness for the system, inelastic responses which are stable under cyclic lateral loading and finally causing outstanding ductility and energy dissipation capacity (Michel Bruneau et al. 1998). To resist the lateral forces EBF systems exert and use both flexure of beam sections and the axial loading of braces. This bodes the need for a laterally stiff framing system with huge energy dissipation capabilities under large seismic forces. The most important distinguishing characteristic of an EBF is the disparted portion of a beam named as "link." A typical Eccentric braced frame consists of a beam, one or two braces and columns. The configuration of EBF systems is similar to the rest of the conventional braced frames. But the only thing which is different is that each brace must be connected to the frame eccentrically. Shear and bending in the beam close to brace are introduced by eccentric connection. The short portion of the frame where those forces are concentrated is termed as "link." All nonlinear behavior and activity is supposed to be confined to the link which is properly detailed. Links are considered as structural fuses that without rarefying much the stiffness and strength and as a result transferring less force to the surrounding columns and beams and braces and are able to dissipate seismic input energy. Common EBF arrangements are given in the figure 2.3 below:

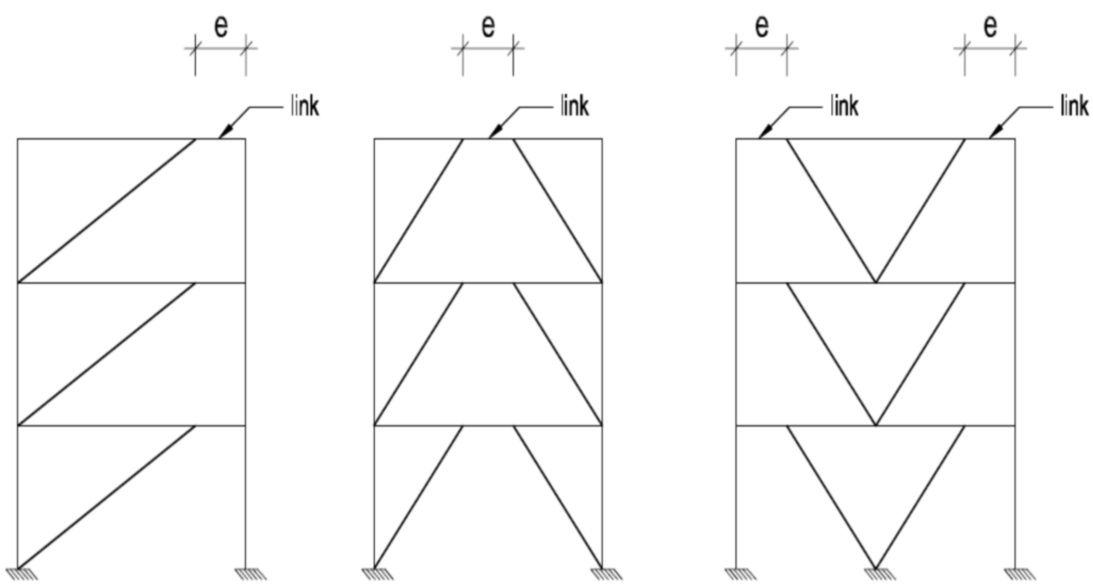

Figure 2.3: Typical EBF configuration

In order to clarify and explain the Lateral stiffness of the EBF it has been defined as a function of the ratio of link length to the beam length. By making link smaller the frame becomes stiffer till its stiffness becomes close the stiffness of CBF. On the other hand by making the link longer the frame will demonstrate more ductile behavior approaching to the stiffness of an ordinary moment frame.

2.2.7.1 Introduction to Eccentrically Braced Frames (EBFs):

Eccentrically Braced Frames are associated with desire of reaching to a bracing system which has laterally enough stiffness with significant dissipation of energy and also the ability of adjustment to huge seismic forces (Charles WRoeder & P.Popov),(1978). Generally an ordinary EBF includes one column, a beam, and one or two braces. The configurations and body of EBFs are exactly like the traditional ones with the only difference which dictates that at least one end of each brace has to be spliced to the frame eccentrically. Bending forces as well as shear forces are introduced in the beam adjacent to the brace by the eccentric connection. Link of an eccentric brace is in fact the short portion of the frame where the so called forces are focused on.

EBF lateral stiffness is primarily a function of the ratio of the link length to the beam length (Egor p.popov,Kazuhiko Kasai& Michael D, p. 44) (1987). As the link becomes shorter, the frame becomes stiffer, approaching the stiffness of a concentric braced frame. As the link becomes longer, the frame becomes more flexible approaching the stiffness of a moment frame.

Producing a frame that can remain substantially elastic outside a well-defined linkage is the most important factor that EBF designing is done on the basis of it. While being undergone huge loading, it is foreseen that the link will be distorted inelastically with great ductility and dissipation of energy. The provisions of codes are provided so that they guarantee the beams, braces, columns and their connections to stay and remain in elastic phase and also the links remain stable. In an earthquake that is considered as a huge one, structural damage and perennial deformation yielded by the link should be expected.

2.2.7.2 Three Important Variables in the Designing of EBF Bracing System

The important variables are as follows:

1) Bracing configuration

2) The link length

3) The link section properties

When these elements are taken into consideration, then the rest of the designing process of the frame can be executed with minimal effect on the link size, configuration or link length.

Designating a systematic procedure to assess the effect of the prominent variables is crucial to EBF design. If attention is not paid identify their effect, then the designer may have to iterate through a myriad of probable combinations. The strategy suggested by (Roy Becker & Michael Ishler,1996) in their guide is as follows:

1) Establish the design criteria.

2) Identify a bracing configuration.

3) Select a link length.

4) Choose an appropriate link section.

5) Design braces columns and other components of the frame.

Designing EBF system is accompanied by series of problems, identical to the most design problems, it is a repetitive process. Most designers will make an initial configuration, link length and link size combination on the basis of approximations of the design shears. Reasonable estimation for braces and columns can simply follow.

When initial configurations and sizes are designated, it is expected that the designer is able to have access to elastic analysis computer software to refine the analysis of the building period, the base shear, the shear distribution through the structure, the elastic deflection of the building and the distribution of forces amongst frame members.

2.2.7.3 Bracing Configuration

The selection of a bracing system configuration is related to various elements. These factors encompass the size and position of required open areas in the framing elevation and the height to width proportions of the bay elevation. These constraints may substitute structural optimization as designing criteria. UBC 2211.10.2 requires at least one end of every brace to frame into a link. There are many frame configurations which meet this criterion.

2.2.7.4. Frame Proportions:

In designing EBF systems, the proportions of frames are typically opted to increase the application of the high shear forces in the link. Frame properties of typical eccentric braces are shown in figure 2.4 below. Shear yielding is very ductile and its capacity for inelastic behavior is very high. This characteristic, as well as the benefits of frames with high stiffness, generally make short lengths desirable.

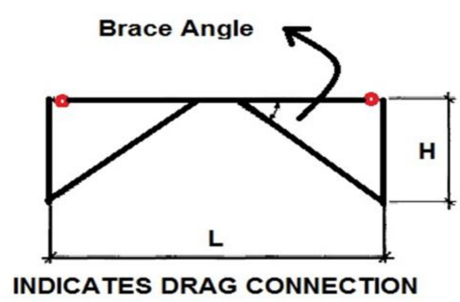

Figure 2.4: Frame proportions

The desirable angel of the brace as shown in the above picture should be kept between 35° and 60°. If the angle is beyond or below this range, then it will result in awkward details at the brace- to- beam and brace-to-column connections. Furthermore for specific gusset plate configurations, it is very sophisticated to align actual members with their analytic performance points. Meanwhile small angles are also apt to result in a huge axial force member in the link beams (Michael D.Engelhardt,and Egor p.popov, p. 504) (1989).

In some frames, if a small eccentricity is introduced at the brace, then the brace connection at the opposite end from the linkage is easier. The mentioned eccentricity can be acceptable, with the conditional assumption that the designing of connection is in a way that it will remain in the range of elastic state at the factored brace load.

In order to optimize designing of the link some flexibility in opting of the link length and its configuration is required. Generally, accommodating architectural features is easier in an EBF system when compared with the concentrically braced frame. There must be a close cooperation and coordination between the architect and engineer for the purpose of optimizing the structural behavior with the architectural requirements.

2.2.7.5. Link Length

The inelastic performance of a link is a great deal affected by its length. If the link length becomes shorter, then the inelastic behavior will become greater as a result the influence of shear forces. Shear yielding has a tendency to occur uniformly alongside the link. Shear yielding has a high ductility and also considerable inelastic performance capacity which is more than that predicted by the web shear area, if the web is braced enough against buckling. (Michael D.Engelhardt,and Egor p.popov, p. 499,1989).

Often the behavior of the links are like short beams which are exposed to equal shear loads applied in opposite directions at the ends of the link. According to this style of loading, the moments produced at both ends are identical and also in the same direction. The shape of the link deformation is like the letter (S), which is distinct at mid span by a point of counter flexure. The measure of moment is equal to 1/2 the shear times the length of the link. Details relevant to the typical loading of the link beam in an eccentric bracing system are shown in figures 2.5 to 2.7.

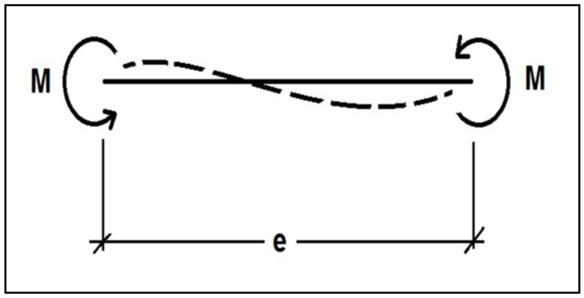

Figure 2.5: Typical loading

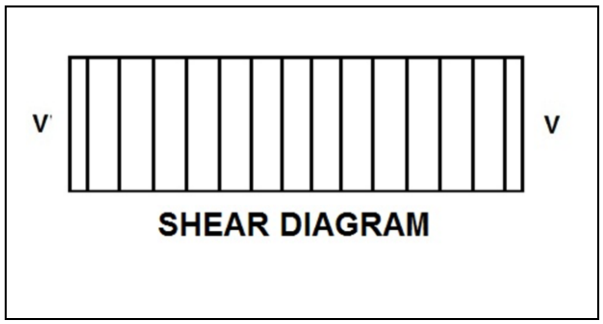

Figure 2.6: Shear Diagram for typical loading

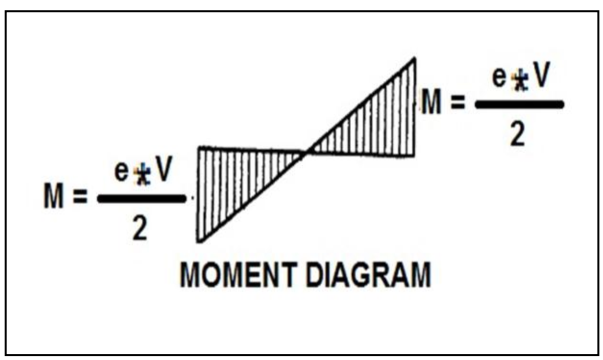

Figure 2.7: Moment Diagram for typical loading

Link lengths generally behave as follows:

If $E< 1.3\ M_s/V_s$

Guarantees shear performance, and are recommended as upper limit for shear links (Egor p. popov, Kasai,and Michael, p. 46) ,(1978)

If $E< 1.6M_s/(V_s)$

Link post - elastic deformation is controlled by shear yielding. UBC2211.10.4 rotation transition. ("Recommended lateral force requirements and commentary", p. 331, C709.4) (1996)

If $E=2M_s/V_s$

Theoretically, the behavior of Link is balanced between shear and flexural yielding .

If $E<2M_s/V_s$

Link behavior considered to be controlled by shear for UBC 2211.10.3 (("recommended lateral force requirements and commentary", p. 330, C709.3)(1996)

If $E>3\ M_s/V_s$

By flexural yielding, Link post-elastic deformation is controlled. UBC2211.10.4 rotation transition. ("Recommended lateral force requirements and commentary", p. 331, C709.4).(1996)

Note: Most of the recent researches have been done on link lengths less than 1.6 (M_s/V_s). Generally these kinds of links behave well, illustrating high ductility with good stability in their hysteretic response.

When a link length becomes shorter then rotation of the link will be greater. UBC 2211.10.4 puts some limits on these rotations. If these limits are exceeded, then the lateral deflection ought to be cut down or on the other hand link length be elevated.

In most design cases, link lengths of about 1.3 M_s/V_s perform well (Egor p.popov, Kazuhiko Kasai and Michael D. 8, p. 46)(1987). This matter facilitates the designer's work and gives them some flexibility in order to modify member sizes and link lengths while they are designing, since the ratio still remains below the 1.6 M_s/V_s code cutoff relevant shear links. Keeping link lengths near the upper limit of shear governed behavior generally leads to plausible rotation of link.

Choosing of link length is often confined by architectural or other configuration restrictions. When there is no such restraints are taken into account, then the initial link length estimates of 0.15L for chevron configurations are reasonable.

The outstanding ductility of shear yielding stimulates most designers to exert shear links in their design. When the minimum link length is restricted, cover plates are probable to be added to the flanges for the purpose of making an increase in the flexural capacity and alter a moment link into a shear link, or the link beam is able to be fabricated as a built up section from plates. A discontinuity in the deflection curvature of the beam will be engendering by Plastic deformation of the link. This is probably due to the concentration of the structural and non-structural damage around the link.

2.2.7.6 Link Beam Selection

Link beams are typically opted to satiate the minimum web area demanded to resist against the shear from an eccentric brace. In general, optimizing the link opted to meet the required dt_w is desirable but it should not exceed this quantity. Extra web area in the link will demand over sizing the other elements of the frame, as they are designed to surpass the strength of the link.

Deformation caused by shear in the link usually causes a moderate contribution to the elastic deformation of a frame. Elastic deflection is caused by the bending of the beams and columns and also by axial deformation happening of the columns and braces. Inelastic deformation of the frame is dominated by rotation of the link caused by its shear deformation. Consequently, the link beams, which appear as the stiffest in an elastic analysis do not necessarily have the greatest ultimate shear capacity.

Generally the design of a link beam is optimized by selecting a section with the minimum required shear capacity and the maximum available bending capacity. The most efficient link sections are usually the deepest sections with the minimum required shear area which comply with the compact web requirements of UBC Chapter 22, Division IX, Table B5.1, and meet the flange width-thickness ratio, $b/2t_f$, not exceeding $52/\sqrt{F_y}$. When the depth or flange size is restricted, the designer may wish to select a section which complies with the shear requirements and add cover plates to increase the flexural capacity. Cover plates may also be used to increase the flexural capacity and transform a bending link into a shear link when nonstructural restrictions prevent reduction of the link length. The designer may customize the section properties by selecting both the web and flange sizes and detailing the link as a built up section.

2.2.7.7 Link Beam Capacity

Since the link portion of the beam element is the "fuse" that determines the strength of other elements, such as the braces and columns, then its capacity should be conservatively determined based on the actual yield strength of the material.

Based on current mill practices, the yield strength of A36 material is approaching 50 ksi, and it will exceed 50 ksi if it is produced as a Dual Grade Steel meeting both A36 and A572 Grade 50 requirements.

Thus, it is now recommended that the capacity of the link beam should be based on yield strength of 50 ksi for A36, A572 Grade 50 and Dual Grade Steels. Although the actual yield point may somewhat exceed 50 ksi, this has been accounted for in the over-strength factors of 1.25 and 1.50 required for the columns and braces, respectively, of the EBF frame.

2.3 Evaluation of Nonlinear Static Procedures

In the study conducted by (H.S. Lew and Sashi K. Kunnath) the effectiveness of nonlinear static procedures for seismic response analysis of buildings were examined. Nonlinear static procedures are recommended by FEMA 273 document in assessing the seismic performance of buildings for a given earthquake hazard representation. Three nonlinear static procedures specified in FEMA 273 are evaluated for their ability to predict deformation demands in terms of inter-story drifts and potential failure mechanisms. Two steel and two reinforced concrete buildings were used to evaluate the procedures. Strong-motion records during the Northridge earthquake are available for these buildings. The study has shown that nonlinear static procedures are not effective in predicting inter-story drift demands compared to nonlinear dynamic procedures. Nonlinear static procedures were not able to capture yielding of columns in the upper levels of a building. This inability can be a significant source of concern in identifying local upper story failure mechanisms.

The American Society of Civil Engineers (ASCE) is in the process of producing an U.S. standard for seismic rehabilitation existing buildings. It is based on Guidelines for Seismic Rehabilitation of Buildings (FEMA 273) which was published in 1997 by the U.S. Federal Emergency Management Agency. FEMA 273 consists of three basic parts: (a) definition of performance objectives; (b) demand prediction using four alternative analysis procedures; and (c) acceptance criteria using force and/or deformation limits which are meant to satisfy the desired performance objective. FEMA-273 suggests four different analytical methods to estimate seismic demands:

- **(I)** Linear Static Procedure (LSP)
- **(II)** Linear Dynamic Procedure (LDP)
- **(III)** Nonlinear Static Procedure (NSP)
- **(IV)** Nonlinear Dynamic Procedure (NDP)

Given the limitations of linear methods and the complexity of nonlinear time-history analyses, engineers favor NSP as the preferred method of analysis.

Following the analysis of a building, the safety and integrity of the structural system is assessed using acceptance criteria. For linear procedures acceptance criteria are based on demand-to-capacity ratios and for nonlinear procedures, they are based on deformation demands. In the research done by (H.S. Lew and Sashi K. Kunnath) , they examined the ability of the FEMA 273 nonlinear static procedures to predict deformation demands in terms of inter-story drift and potential failure mechanisms in the system.

2.3.1 Nonlinear Static Procedures for Seismic Demand Estimation

There are several procedures that can be adopted for conducting a nonlinear static analysis. While the fundamental procedure for the step-by-step analysis is essential and identical, the different procedures vary mostly in the form of lateral force distribution to be applied to the structural model in each step of the analysis. FEMA- 273 recommends the following three procedures:

2.3.1.1. Inverted Triangular Pattern (FEMA-1):

A lateral load pattern represented by the following FEMA-273 equation:

$$\boldsymbol{F_x = \frac{w_x h_x^k}{\sum_{i=1}^{n} w_i h_i^k} V} \qquad (2.1)$$

Where: F_x = lateral load at floor level x

$W_{x,i}$= weight at floor level x, i

$h_{x,i}$= height from base to floor level x,i

k = 1.0 for T < 0.5 seconds;

k = 2.0 for T > 2.5 seconds,

With linear interpolation for intermediate values

V = total lateral load (base shear) to be applied to the building

This load pattern results in an inverted triangular distribution across the height of the building and is normally valid when more than 75% of the mass participates in the fundamental mode of vibration.

2.3.1.2 Uniform Load Pattern (FEMA-2):

A uniform load pattern based on lateral forces that are proportional to the total mass at each floor level.

$$\boldsymbol{F}_{\boldsymbol{x}}=\frac{\boldsymbol{w_x}}{\sum_{\boldsymbol{i}=\boldsymbol{1}}^{\boldsymbol{n}}\boldsymbol{w_i}}\boldsymbol{V} \tag{2.2}$$

(This pattern is expected to simulate story shears.)

2.3.1.3 Modal Load Pattern (FEMA-3):

A lateral load pattern proportional to the story inertia forces, consistent with the story shear distribution calculated by a combination of modal responses is considered as the modal load pattern.

In the study conducted by (H.S. Lew and Sashi K. Kunnath), each of the above procedures was evaluated using four sample structures: 6- story steel frame building; 13-story steel frame building, 7-story concrete building, and 20-story concrete building. These buildings have strong-motion records from the Northridge earthquake. The strong motion records at the roof level were used to calibrate the building models.

2.3.2 Experimental Evaluation of Nonlinear Static Procedure Conducted by H.S. Lew and Sashi K. Kunnath

In the study conducted by (H.S. Lew and Sashi K. Kunnath), they examined the effectiveness of nonlinear static procedures for analysis of inelastic response of buildings. Specifically, the FEMA 273 procedures are evaluated to see whether nonlinear static procedures can predict deformation demands in terms of inter-story drift and potential failure mechanisms in the system.

1) Six-Story Steel Moment-Frame Building,

2) Thirteen-Story Steel Moment- Resisting Frame Building,

3) Seven-Story Reinforced Concrete Moment Frame Building, and

4) Twenty-Story Concrete Moment Frame Building

A frame model of each of the above buildings was first calibrated against observed instrument data. Then, each of the building models was analyzed using a detailed nonlinear time-history analysis followed by a series of nonlinear static pushover procedures. They were:

I) A lateral load pattern represented by an inverted triangular load (FEMA-1).

II) A uniform load pattern based on lateral forces that are proportional to the total mass at each floor level (FEMA-2).

III) A lateral load pattern proportional to the story inertia forces consistent with the story shear distribution calculated by a combination of modal responses using a response spectrum analysis (FEMA-3)

2.3.2.1 The Following Conclusions were Drawn from the Study

Nonlinear static procedures are generally not effective in predicting inter-story drift demands compared to nonlinear dynamic procedures. Drifts are generally under-estimated at upper levels and sometimes over-estimated at lower levels.

2. The peak displacement profiles predicted by both nonlinear static and nonlinear dynamic procedures are in agreements. This suggests that the estimation of the displacement profile at the peak roof displacement by nonlinear static procedures is reasonable so long as inter-story drifts at the lower levels are reasonably estimated.

3. Nonlinear static methods did not capture yielding of columns at the upper levels.

This inability can be a significant source of concern in identifying local upper story mechanisms.

2.4 Background to Frame Analysis

2.4.1 Introduction

The laws of physics and mathematics are implemented in structural analysis to figure out the performance and behavior of structures. The real performance of a structure is complicated, but disparate level of idealization is able to cut down the complexity. In this chapter the term analysis is basically dealt with the processes and guidelines in order to provide the member strength and deformation demands of a building while under seismic load excitation. Firstly, different analysis methods proposed by Federal Emergency Management Agency (FEMA), National Earthquake Hazards Reduction Program (NEHRP) are cited. Then there is a discussion over some analysis processes which are given briefly and finally some solution techniques are again discussed.

2.4. 2 Analysis Methods

Various levels of complexity relevant to geometry of the structure as well as the material behavior are involved in the structural response under seismic excitation. Depending on the required soundness different kinds of idealizations are suggested in the assessment of structural response. FEMA-355f took four elastic and three inelastic analysis processes into consideration exerted by FEMA-273, NEHRP Guideline (1997) for performance assessment of steel moment resisting frames.

I) Elastic Analysis Methods:

The suggested elastic analysis procedures are: equivalent lateral force and modal analysis by FEMA-302, FEMA-273, linear static and linear dynamic methods and linear time history analysis procedures (FEMA-355f).

Base shear is calculated in accordance with seismic response coefficient and total dead load by plausible portion of other loads, in the equivalent lateral force method. This base shear is disseminated to disparate floor levels and the response is to be calculated on the basis of static analysis (FEMA-355f).

FEMA-273 linear static procedure implements the identical background as the equivalent lateral load approach for calculating the seismic load. On behalf of designing the base shear this approach introduces the term “pseudo lateral load” that is the final product after implementing various modification factors. The “pseudo lateral load” is opted in a way so that the response relevant to this load will be to some extent identical to that reached by nonlinear time history analysis (FEMA-355f).

Linear time history procedure exerts two approaches for calculation of structural response. In the first method, modal analysis and mode superposition is used which is clarified later in this chapter. In the second method, this procedure exerts direct integration technique for calculation of seismic response. Some prevalent consolidation techniques namely Newmark and Wilson approaches are discussed at the end of this chapter.

II) Inelastic Analysis Methods:

For inelastic analysis procedures that are considered, these methods can be mentioned: FEMA-273 nonlinear static procedure, capacity spectrum procedure (Skokan and Hart, 1999) and nonlinear time history analysis (FEMA-355f).

The other name of FEMA-273 nonlinear static procedure is the static pushover analysis, which in academic settings is well known. Inelastic material behavior is included in the static pushover analysis method by considering P-Δ effects. These effects are concisely discussed later. In this approach a target displacement is designated at any point of the structure and then the building is pushed with an incremental lateral load till, the target displacement is reached to that point or in other words the structure collapses (FEMA-355f).

Capacity spectrum approach is substantially plausible for reinforced concrete structures (ATC, 1996) and as a result is out of the scope of this article. It should be realized that the nonlinear time history analysis method is same as the linear time history analysis, but merely material nonlinearity as well as the geometric effects should be taken into account in order to evaluate the structural response. Some important terms of these analysis procedures are detailed in the subsequent sections.

2.4.2.1 First Order Elastic Analysis

The structural behavior is considered as linear under any kind of loading by the first order elastic frame analysis. This analysis approach does not take into account the geometric effects of members and the structural deflections (P-Δ and P-δ effects) as well as the material nonlinearity. It supposes that the displacement is to be minute and thus the second order effects as a result of geometrical changes are ignored. Therefore, the matrix of stiffness is stable for the members that are not dependent on the applied axial forces. The deflection is symmetrical with the applied load, it means that by incrementing the quantity of loads then the displacement will also expand that can be expressed as a straight line correlation as shown in figure 2.8 below.

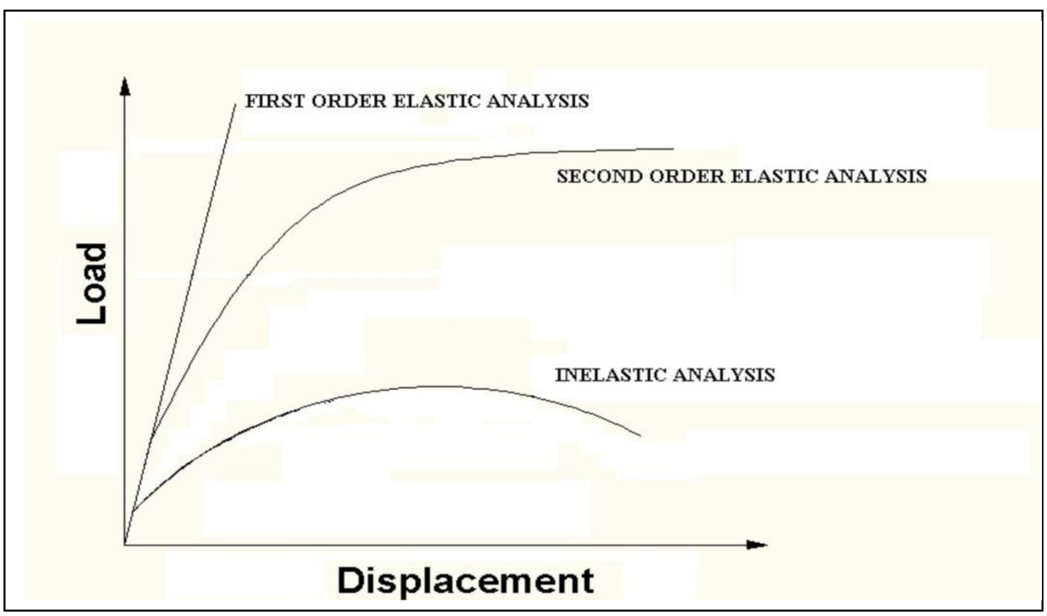

Figure 2.8: Generalized Load-Displacement curve for different types of analysis

The primary slopes of other kinds of analyses are coincided with it. It can be justified as at lower loads the structures do not cause any subsequent impacts in geometry and material properties.

2.4.2.2 Second Order Elastic Analysis

The geometric impacts that are considered in the second order elastic analysis due to the member and structural deflections that are named as P-δ and P-Δ effects respectively. Due to the second order elastic analysis, the structural response is illustrated in the load-displacement curve shown in Figure 8. Primarily it succeeds the path of linear analysis, but as the loading became greater, to produce enough geometric effect, it commences to serve from linear analysis to demonstrate the effect of geometric nonlinearity. Figure 2.9 shows P-δ and P-Δ effects, which is the reason for this geometric nonlinearity.

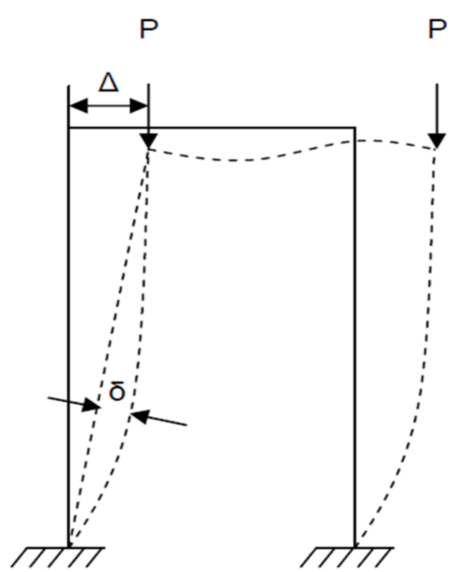

Figure 2.9: P-δ and P-Δ effects

These geometric impacts engender higher internal forces because of axial loads. The matrix of stiffness needs to be set to echo these impacts and the corrections promote extra deflection. The system reaches to equilibrium in a repetitive process in order to solve this problem. Principle of superposition is not plausible in second order elastic analysis, as the stiffness matrix and consequently the structural response are dependent on the deflected shape of the frame. This method over predicts the collapse load since the material nonlinearity is not taken in to account.

P-δ EFFECT:

When deformation happens in a member, this would have some effect on the stiffness of that member and extra moment will be generated in that member. This second order impact, which is due to deflection through a member and the axial force, is termed as P-δ effect.

P-Δ EFFECT:

If a structure deflects substantially, then the primary geometry of the structure cannot be used for formulating the transformation matrix because of alteration in nodal coordinates. It is termed as P-Δ effect amongst Engineers.

2.4.2.3 Inelastic Analysis

Some elements, such as yielding of Material and instability of structural members, have important impact on controlling the final load. If the material nonlinearity is included in the second order elastic analysis then it would become the inelastic analysis. Nonlinearity in the inelastic analysis splits into two kinds – geometric nonlinearity and material nonlinearity.

Material commence yielding at the outer fiber of the section while the elastic moment becomes close to the yield moment point M_y. At this point material nonlinearity comes into effect and then by applying extra load, yielding will disseminate through the section from outer fiber to plastic neutral axis. The yielding of the section will continue until the thorough section is yielded which leads to the development of full plastic moment M_p. This nonlinearity can be consolidated to the analysis substantially by two methods. Plastic hinges in the first method are supposed to form at the extreme ends of a member i.e. all the material nonlinearity is substantially lumped at the two extreme ends of a member. It is known as concentrated plasticity (plastic hinge, lumped plasticity) approach. The other approach is on the basis of the assumption that the plasticity is over the whole member and known as the distributed plasticity (plastic zone) approach (Chan and Chui, 2000).

2.4.2.4 Concentrated Plasticity Approach

The progressive yielding along the member length is overlooked by concentrated plasticity approach. This method presumes that the material nonlinearity is accumulated and consolidated in a small region with zero length (Yau and Chan, 1994). It is associated with the plastification of cross section that commences at the outer most fiber of the section and winds up to the occurrence of a hinge at a given point. The assumption here is that the hinges will occur merely at the ends and the rest of the member will stay in elastic phase. Mashary and Chen (1991) and Yau and Chan (1994) simulated the so called material nonlinearity with the help of exerting zero length spring at the ends of the member. Many other approaches are suggested for computer modeling relevant to this hinge property. This approach is easier than the the distributed plasticity approach and it saves computation time. Prevalent implemented approaches for simulating plastic hinges are as following: a) Elastic-plastic hinge method, b) Column tangent modulus method, c) Beam-column stiffness degradation method, d) Beam-column strength degradation method and e) End spring method.

2.4.2.5 Distributed Plastic Approach

This approach assumes yielding will be distributed over the length of the member and the cross section. This method discretized structure into many elements. In order to try to observe stress and strain for all of the members each section is further divided into smaller fibers. Primary defects and residual stresses can be included by assigning stresses to each fiber before loading, which can be varied along the side and thickness of the section (Chan, 1990). The distributed plasticity method is more precise when compared with the concentrated plasticity approach since substantial stress-strain correlation is directly applied for the computation of forces. This approach demands great amount of time for computation and also requires huge memory capacity to store data. Therefore, this method is proper for analyzing structures that are simple. Prevalent implemented methods for this approach are traditional plastic zone method and simplified plastic zone method.

2.4.3 Dynamic Analysis of frame

Structural dynamics is concerned with the performance of structure under dynamic loading. While a static load is defined as loading that does not vary over a period of time, the dynamic load is defined as any load that alters its magnitude, direction or position over a period of time. If the change happens very slowly, then the response of the structure maybe determined exerting static analysis. On the other hand if the loading changes quickly by time (corresponding to the structures time period), then the response of the structures must be determined implementing dynamic analysis. In the study done by (S. M. Ashfaqul Hoq,May 2010) ,dynamic term was used for seismic loads.

Structural dynamic analysis is different from the static analysis in two ways. First of all, for a dynamic problem both the resulting response and the applied force response in the structure are considered as variants of time, i.e. function of time, and there is no single solution like the static problem. In order to accomplish the assessment of structural response one has to probe the solution during a specific interval of time. The second one is considered as the most prominent characteristic in dynamic analysis, the inertia force act. If a dynamic load is applied to structure, there will be time variant deflection in the structure that will engender acceleration and therefore as a result inertia force will be inferred. The acceleration and mass characteristics of the structure are two parameters that magnitude of the inertia force is depended on.

Contrary to the static analysis, dynamic problems greatly depend on damping and mass. For the purpose of writing the equations of motion there are three components or parameters, namely mass, damping and the stiffness characteristics that are required. Mass is obtained and calculated from all the loads that the structure bears and also the self-weight of the members. This mass is able to be consolidated and accumulated at the joints or disseminated upon the member. Substantially structural components of a system can provide the stiffness and the last element known as damping parameter which bodes the properties of energy dissipation of a material or system. It is a procedure that a free vibration by its execution could constantly subside in amplitude and at last come to rest. Various mechanisms can be implemented in order to dissipate the energy in vibrating system and in many cases more than one mechanism can be illustrated simultaneously.

The equivalent lateral load method cited before alters dynamic force into static forces. It cannot reflect the true dynamic response, but because features of resonance cannot be explained in a static approach then it cannot bode and echo the true dynamic response of the building. Mode superposition and modal analysis is a renowned accepted method for linear systems, for the purpose of considering all dynamic impacts in the analysis. Disparate kinds of direct integration methods are implemented in order to reach to the numerical solution to both linear and nonlinear dynamic problems. Different methods are briefly discussed in the following sections.

2.4.3.1 Modal Analysis

Modal analysis method is exerted in structural dynamics in order to specify the natural mode shapes and frequencies of the structure. It is a comfortable method of computing the dynamic response relevant to a linear structural system. The response of a MDF system under externally applied dynamic load can be explained by N disparate equations as follows,

$$[m]\{\ddot{U}\} + [c]\{\mathring{u}\} + [k]\{u\} = \{p(t)\}, \text{ where} \quad (2.3)$$

[m] is the mass matrix,

[c] is the damping matrix,

[k] is the stiffness matrix of the system,

{p(t)} is the externally applied dynamic force matrix, and

$\{u\}$, $\{\mathring{u}\}$, $\{\ddot{u}\}$ denotes displacement, velocity and acceleration matrix.

The prominent strategy of this approach for dynamic analysis is to alter N set of coupled equations of motion into N uncoupled equation for a multiple-degree-of-freedom system. Based on the number of DOF, a MDF system has multiple characteristic deflected shapes. Each characteristic deflected shape is called a natural mode of vibration of the MDF system denoted by $ø_n$.

By means of the superposition of modal contributions the displacement {u(t)} of the system can be determined. i.e. {u (t)} $=\sum_{n=1}^{N} q_n$ (t) $ø_n$, where q_n (t) = modal coordinates. Deflected shape $ø_n$ does not change during the passage of time. The equation, $[k]ø_n = \omega_n^2[m]ø_n$, is the matrix eigen value problem where ω_n is the natural frequency and $ø_n$ is the natural modes of vibration of the system (Chopra, A.K. 1995). This equation possesses a non-trivial on the provision that solution,

$$\left| [k] - \omega_n^2[m] \right| = 0, \tag{2.4}$$

It is recognized as the frequency equation since after expanding the determinant then it engenders a polynomial of order N in ω_n^2 . This equation possesses N number of roots for ω_n^2 that are positive and real numbers for N number of natural vibration frequencies, commenced with $\omega_1$as the smallest and ω_n the largest. If applied force {p (t)} can be written as $[s]_p$ (t) with spatial distribution is determined with [s], then the spatial distribution is expanded to components of its modal $\{S_n\}$. Where, $\{s_n\} = \Gamma_n$ $[m]ø_n$. Then the equation could be converted to disported equations in modal coordinates and the solution for the modal coordinate is, q_n(t)= $\Gamma_n D_n$(t), where D_n is controlled by the equation of motion for nth-mode SDF system of the nth mode of the MDF system. The contribution from this mode to modal displacement is, $\{u_n$ (t)$\}$ = $\phi_n q_n$(t) = $\Gamma_n \phi_n D_n$(t). And the identical static force relevant to the nth mode response is, $\{f_n$(t)$\}$ =$\{s_n\}\{A_n$(t)$\}$, where A_n (t) = $\omega_n^2 D_n$ (t), is the pseudo acceleration. The n^{th} mode contribution to any response is defined with the static analysis for force $\{f_n\}$. Consolidating all the response contributions derived from all the modes results in the total dynamic response (Chopra, A.K. 2007).

2.4.3.2 Step-by-Step Integration

Analytical solution of a dynamic problem is not probable in cases where the physical properties such as geometry and elasticity of material do not stay fixed. The coefficient of stiffness can be altered with yielding of materials or by great variance regarding to axial force that will engender the alteration in the coefficient of geometric stiffness. The approach which can be applied for the analysis of nonlinear system is the numerical step-by-step integration, which is also applicable to linear systems. The principal strategy of this method is to split the response history into short time increases and then the response is calculated during each increase presuming it as a linear system with the characteristics defined at the beginning of each increase. The characteristics are brought up to date at the end of each interval on the basis of deformation and stress, and therefore, the nonlinear system is idealized as a collection of changing linear systems. In 1959, Newmark N.M. created a method of calculation associated with dynamic problems in structure. Later revisions were made to this method in accordance with stability, accuracy etc. Some famous methods are briefly discussed in the following sections.

2.4.3.3 Newmark's Method

In this method it is presumed that at time (i), the values of displacement, velocity and acceleration is known and by numerical integration it can be appraised for time(i+1), if the time increase, Δ_t is very minute. Newmark proposes two parameters γ and β in order to signify the proportion of acceleration that will enter into the equations for displacement and velocity (Newmark N.M. 1959). The adopted equations are as following:

$$u_{i+1} = u_i + (\Delta_t)\, \mathring{u}_i + [(0.5 - \beta)(\Delta t)^2]\, \ddot{u}_i + [\beta(\Delta t)^2]\, \ddot{u}_{i+1} \tag{2.5}$$

$$\mathring{U}_{i+1} = \mathring{u}_i + [(1-\gamma)\, \Delta_t\,]\, \ddot{u}i + (\gamma\, \Delta_t)\, \ddot{u}_{i+1} \tag{2.6}$$

The two parameters γ and β account for the stability and preciseness of the system. If γ is assumed to be zero, a negative damping will happen, which will lead to an auto vibration from the numerical approach. On the other hand if we assume γ is greater than 1 ⁄ 2, then a positive damping as well as the real damping will be introduced to cut down the magnitude of the response. Consequently, in general γ is assumed as equal to 1 ⁄ 2. The better results are obtained with measures of β in the range of 1⁄6 to 1⁄4 (Newmark, N.M. 1959). Two well-known special cases for Newmark's approach are as following:

2.4.3.4 Average Acceleration Method

If there is no disparity of acceleration through a time step and the value is stable and, equal to the value of medium acceleration, then,

Ü (τ) =1/2 (üi+1+ü$_i$). If (τ) = Δ_t, this will yield

$\mathring{U}_{i+1}$= $\mathring{u}_i$ +1/2 Δ_t (ü$_i$ +ü$_{i+1}$), and u_{i+1}= u_i + (Δ_t) $\mathring{u}_i$ +1/4 (Δ_t) 2 (ü$_i$ + ü$_{i+1}$).

If γ = 1/2 and β =1/4 then the above two equations will be identical to equation 2.5 and 2.6.

For this method, utmost velocity response is not wrong whether the value of β other than 1/4 will engender some error (Newmark N.M. 1959). From stability point of view Newmark's method is stable if,

$$\frac{\Delta t}{Tn} < \frac{1}{\pi\sqrt{2}}\frac{1}{\sqrt{\gamma - 2}} \qquad (2.7)$$

Where, T_n is the natural time period of the system.

For, $\gamma = \frac{1}{2}$ and $\beta = \frac{1}{4}$, $\frac{\Delta t}{Tn} \leq \infty$ average acceleration method is steady under all circumstances.

2.4.3.5 Linear Acceleration Method

If the acceleration fluctuation performance is linear along a time step, then,

Ü (τ) = ü$_i$ +τ/Δ_t (ü$_{i+1}$–ü$_i$). If (τ) =Δ_t, this will yield

$\mathring{U}_{i+1}$= $\mathring{u}_i$ + 1/2 Δ_t (ü$_i$+ü$_{i+1}$), and

U_{i+1}= $u\ _i$+ (Δ_t) $\mathring{u}_i$ + (Δ_t) 2 (1/3 üi + 1/6 üi+1).

The above two equations are identical from equation 2.4 and 2.5, if γ = 1/2 and β =1/6.

Equation 2.3 illustrates that the linear acceleration approach will remain fixed, if Δt/T_n≤ 0.551. Then as a result this approach is conditionally steady. On the other hand the criteria associated with stability have not compelled any provision in choosing time step. Generally by taking short integration interval, a good authenticity can be obtained from unconditionally stable linear acceleration method.

2.4.3.6 Wilson θ Method

E.L.Wilson altered the conditionally stable linear acceleration approach into unconditionally stable. His suggested approach is famous as Wilson θ Method. In this approach it is presumed that the acceleration will change linearly through an extended interval, $\delta_t = \theta\Delta_t$.

The parameter θ in this approach designates the exactness and the steadfastness traits of the numerical analysis. If θ = 1, therefore this approach will shift to Newmark's standard linear-acceleration method. But if θ ≥ 1.37, Wilson's method becomes fixedunder all circumstances. The details are shown in figure 2.10.

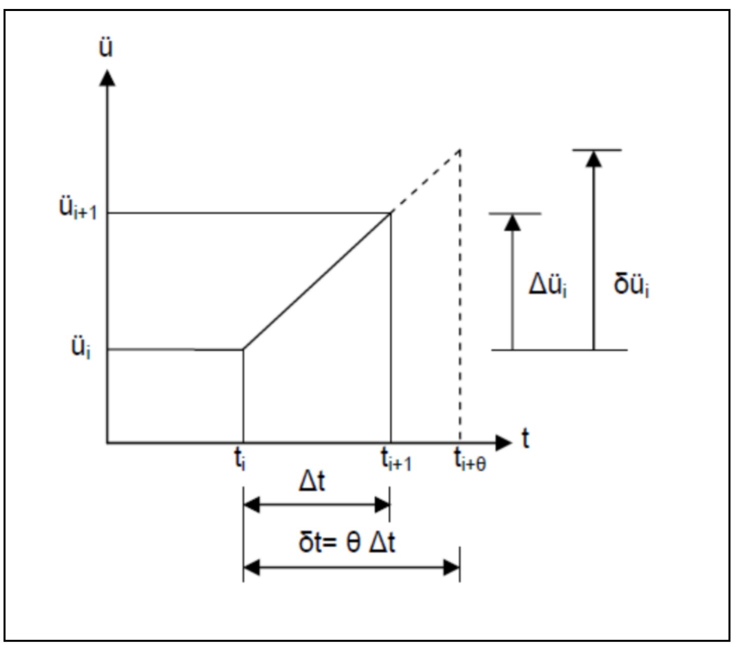

Figure 2.10: Linear variation of acceleration over extended time times

2.4.3.7 Hilber-Hughes-Taylor Method

For the purpose of defining and introducing the damping in numerical form into Newmark's method which does not affect the accuracy Hilber, Hughes and Taylor proposed the α parameter Where,

$$\gamma = \frac{(1-2\alpha)}{2} \quad \text{And} \quad \beta = \frac{(1-\alpha)^2}{4} \tag{2.8}$$

The parameter is in the range of -(1 ⁄ 3) to (0). If α = 0, then this method shifts to Newmark's medium acceleration approach. This matter would lead to the higher preciseness but it may produce extra vibrations while in the higher modes. If the value of α decreases then the amount of numerical damping will increase, that will fundamentally damp the modes with higher frequency. In some cases it is required for a nonlinear solution to meet with exertion of negative α measure (SAP2000).

Chapter 3

DESIGN OF MODEL STRUCTURES

This chapter includes only one section. The methodology of design is described in section 3.1. Then on the basis of the designing in this chapter, the results and discussions (design sections, weight of the sections and the total Weights of the frames, making comparison amongst the different kinds of braced frames in their performance and also from economical view) are given in consequent sections. The units of Kg, Kgf and meter are used in this study for mass, force and distance respectively.

3. 1. Methodology of Design

The geometry of the frames that are going to be designed and analyzed in this study is defined in section 3.1.1. An economical comparison of bracing systems is also given in this section 3.1.2. Choice of 2-D versus 3-D models are done in section 3.1.3. The criteria of design are chosen in section 3.1.4. Design software is selected and introduced in section 3.1.5. The design materials and the steel sections which are going to be used for designing the frames are given in sections 3.1.6 and 3.1.7. And finally Sections 3.1.8 and 3.1.9 are devoted to connections, loading of the frames respectively

3.1.1 Frame Geometry

In order to evaluate different bracing systems, prior to going into any action for assessment, models with different bracing systems must be designed. In this regard the types of opted models, their shape and sizes are significant as they have influence on the nonlinear behavior outcomes of the frame models (Maheri & Akbari, 2003, Kappos, 1999, Assaf , 1989, Tremblay, 2002, Kim & Choi, 2005, D. Ozhendekci & N. Ozhendekci, 2008) and also on the economic aspects (Kameshki & Saka, 2001, Tremblay, 2002, Maher & Safari, 2005, D. Ozhendekci & N. Ozhendekci, 2008, Richards, 2009). Therefore, the frame geometry selected was to some extent identical to the previous researches done about similar subjects.

Frames similar in some ways to those used by Arash Farzam are introduced in this study. The aim was to accomplish results that were in A.Farzam work and it was the future suggestion of his work. The details of his frames details imitated the study done by Maheri and Akbari (2003) amongst the whole available literature relevant to this subject, therefore this study is also similar to that of Maheri and Akbari (2003). The frames geometry were originally used by Mwafy and Elnashai (2001).Prior to Maheri and Akbari these frames were suggested by Mwafy and Elnashai (2001) who made a comparison between the outcomes of nonlinear static (pushover) and dynamic procedures done on these frames and came up with the fact that NSP is an outstanding method in order to assess the nonlinear behavior of the so-called frames. For the purpose of giving veracity and authenticity to the application of nonlinear static procedure and the models exerted in this study, similar models and analysis approaches as in the above mentioned studies have been applied and adopted.

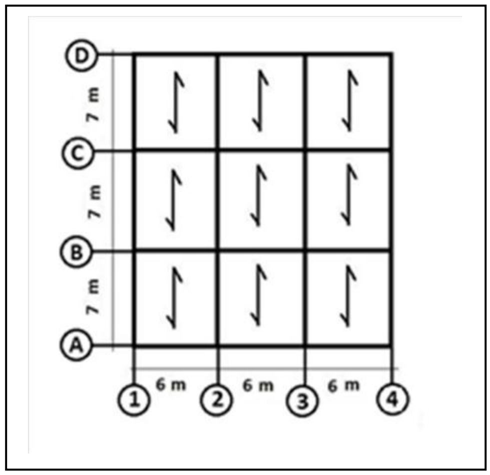

Figure 3.1: The Same assumed Plan for 4, 8 and 12

The details of frames geometry and location are as following**:**

1) In accordance with (Maheri & Akbari, 2003 and Mwafy & Elnashai, 2001) three different eccentric types of braces have been implemented in nine frames with various stories (4-, 8- and 12-storey frames) so that these frames delegate low to medium rise structures.
2) Each frame has three bays with 6 meter span length somewhat similar to that of Maheri and Akbari (2003) because the span was 5 meter in their study, although the original frames had five bays with span lengths of 5 meters (Mwafy and Elnashai, 2001). The

former had reduced the number of bays since according to Assaf (1998), number of spans has little effect on nonlinear response of the frames but reduction of bays increases the speed of frame analysis by computer. The use of three bays is a very good choice for the efficient placement of the X-, V-, and inverted V- bracing systems within the frame central bay. Moreover, Diagonal bracing system can be located within the two perimeter bays at the right and left corner.

3) Height of all stories is 3.2 meters, except for the first floor which is 2.8, it should be beckoned that the heights are a little different from the above mentioned references as in them all heights had been considered 3 meter.

4) Each frame is braced against lateral loading with three different eccentric bracing systems (V-, inverted V- and diagonal).The type of the ground in this study is type II and the site of the construction is in Mashhad(A big city in Iran)

3.1.2 Calculation of the Entire Frame Weight

In order to reach an economical comparison of the different braced frame types and in accordance with the approach which had been used and introduced by Kameshki and Saka (2001) and D. Ozhendekci and N. Ozhendekci (2008), the whole weight of the frames with different bracing systems are calculated and compared with each other.

3.1.3 2-D versus 3-D Models

There is a dilemma about choosing one of the following for the analysis and design; 2-D or 3-D. These are the factors that may influence the computer analysis time. Disparate tenets exist amongst various scientists relevant to the options. Basically, the decision is to be made on the basis of the regularity degree of the structures.

Mwafy and Elnashai (2001), who were the first researchers exerting these approaches, acclaim that as a result of regularity of the frames, 2-D frames are apt to show clearly

The behavior of the structure and exerting 3-D models are not needed as they decrease the analysis speed and also are time consuming. Also Maheri and Akbari (2003) have introduced their frame models in two dimensional forms, authenticating their style by quoting and referring to the previous reference. Also in this research two dimensional modeling has been considered in order to imitate the two above mentioned studies while extra verification and veracity has been brought by using and the implementation of FEMA 356 rules. This method has been considered since it has been widely used by researchers. According to the above mentioned rules of FEMA 2 dimensional models are allowed for structures, if the structure has the following 2 provisions:

1) If the diaphragms of the structure are rigid
2) If the effects of horizontal torsion have been taken into consideration in the model

Since these two conditions exist in the frames of this study, then the model 2-D of 2-D is selected rather than 3-D models and this is also in line with FEMA 356.

The geometries of the 4-story model are given in the Figures 3.2 to 3.4. The 8- and 12-story model geometries are similar to the 4-story frames with the only difference in the number of stories.

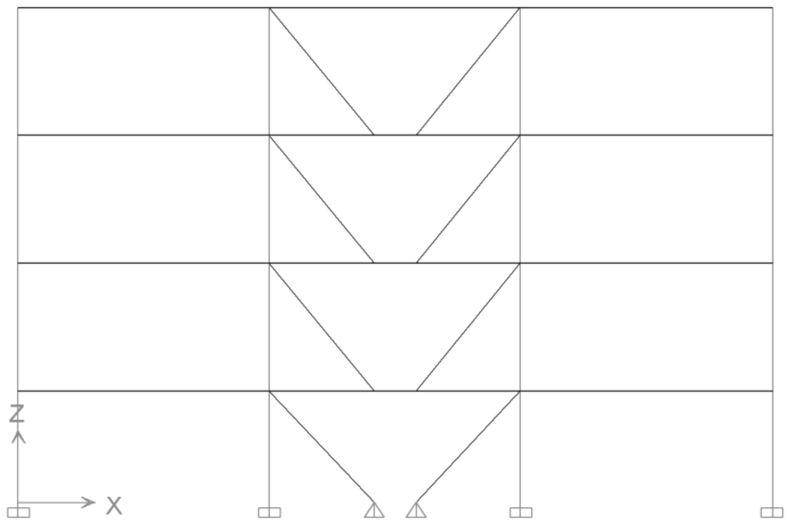

Figure 3.2: 4 story eccentric inverted V braced frame

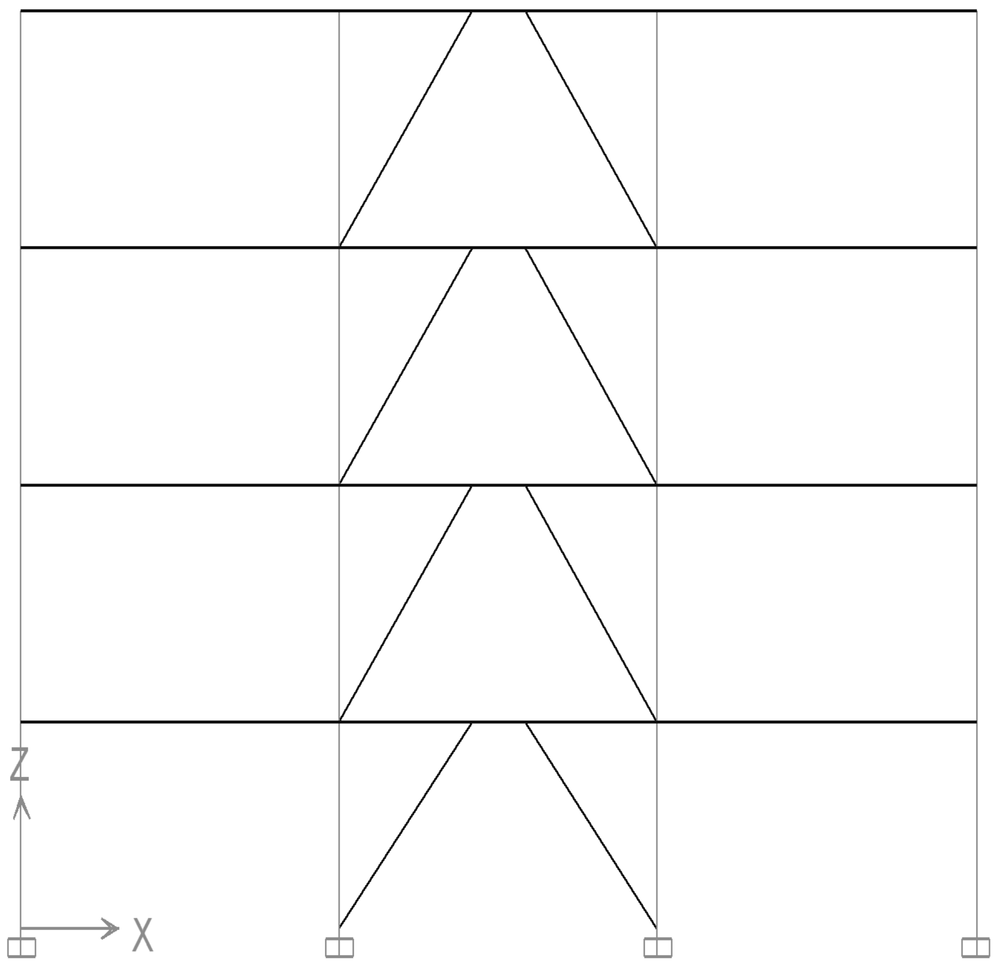

Figure 3.3: 4 story eccentric inverted V braced frame

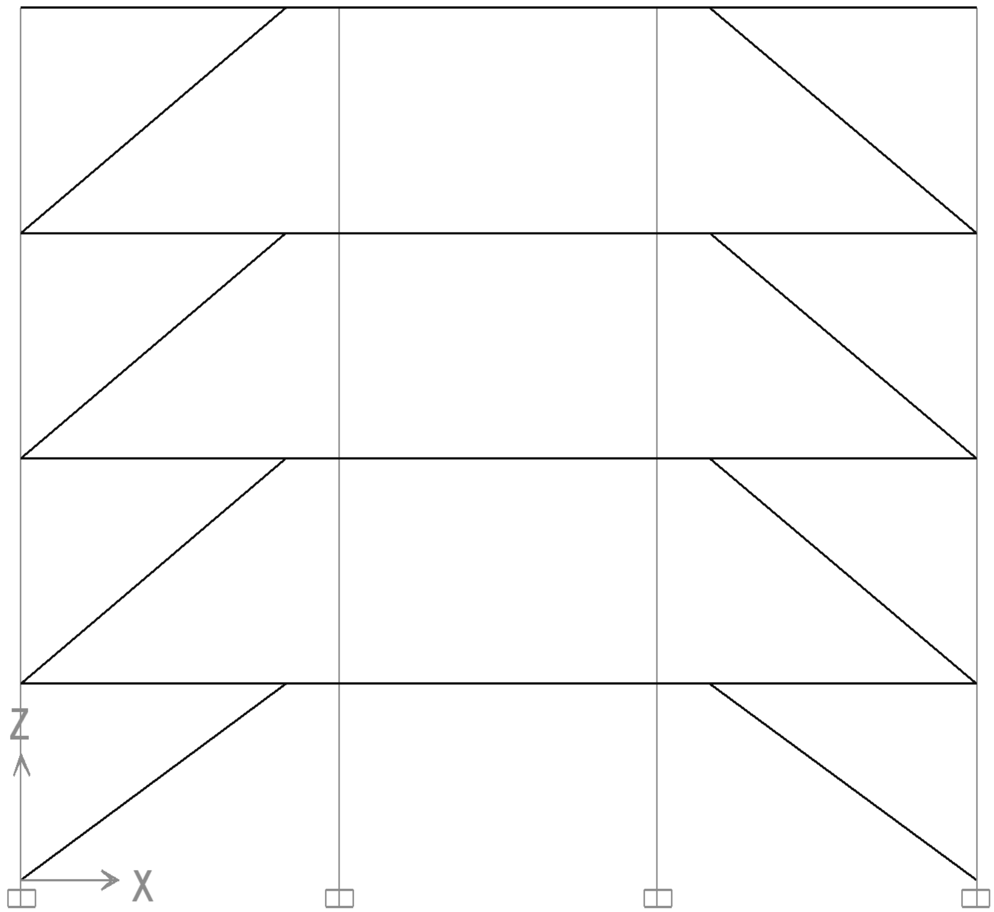

Figure 3.4: 4 story eccentric Diagonal braced frame

3.1.4 Design Criteria

AISC LRFD (1999) is used as steel design code. Sixth volume of National provision of Iran and 2800 code (Third ed.) were used as loading codes. Fifth volume of National provision of Iran is used for the properties and characteristics of materials such concrete, steel, cement, brick, mortar, ceramic, tile and so on.

3.1.5 Design Software

ETABS 9.7.1 is used for this study since this software is widely used amongst Civil Engineers and also it is powerful enough to perform nonlinear static analysis as well as linear, static and dynamic analysis.

3.1.6 Design Material

In this study the steel material properties are opted in accordance with Fifth volume of national provisions of Iran,

The steel properties are detailed below:

1) Modulus of Elasticity: $E = 2.04 \times 10E10$ kg/m^2

2) Poisson's Ratio: $\vartheta = 0.3$

3) Weight per Unit Volume: 850kgf/m^3

4) Mass per Unit Volume: 850kg/m^3

5) Minimum Yield Stress: 3700kgf/m^2

6) Effective Tensile Stress: 5200kgf/m^2

3.1.7 Design Sections

IPE sections are used for beams and Boxed Column-sheet and boxed sections are used for the columns and bracing members respectively, the so called sections are all produced in Isfahan steel company in Iran [Appendix F]. This is in line with most of the practical works in Iran. It should be noted that I-sections (IPE and IPB from DIN Sections) for beams and columns and rectangular hollow sections for braces had been used by Moghaddam, Hajirasouliha and Doostan(2005) in their study.

For a structure in which I sections are used for the columns, the horizontal loads (earthquake load) become more critical, if the columns are set in the direction where weak axis bending occurs. In order to consider this matter, while drawing the structure; rotate the columns 90 degrees so that the weaker axes are parallel to the lateral load direction. But in this study the cross-sections are square and therefore it is needless to rotate the columns as the results are identical.

3.1.8 Connections

The connections of beam-column have been assumed to be pinned. Also the connections of braces were assumed to have pinned joint. The column base plate connections are fixed in order to reduce the column sections.

3.1.9 Loading

The loading defined in this study splits into two different types:

1) Gravity Load

2) lateral (earthquake loads)

Gravity loads are defined in accordance with the sixth volume of national provisions of Iran, and also the lateral loads are calculated in accordance with (2800 earthquake code) of Iran.

In this study the amount of Dead load and Live load calculated are respectively are given below:

Dead load: 700 $\frac{\mathrm{Kg}}{\mathrm{m}^2}$

Live load (for residential buildings): 200 $\frac{\mathrm{Kg}}{\mathrm{m}^2}$

Therefore according to its regulation (national provision of Iran), the participation coefficient of Dead load and Live load have been exerted as 100 percent and 20 percent respectively (1 for Dead load and 0.2 for live load). On the basis of the fact that the tenth volume of national provision of Iran is not defined in ETABS software, according to acclaimed and issued agenda by the National Engineering Organization of Iran , in the design of the structure we are allowed to use the pre-defined codes in ETABS software but on condition which dictates that the design must be on the basis of ASD or LRFD method, then for designing of the structures, AISC LRFD 99 was used.

The calculation of coefficient is as following:

$$C = \frac{ABI}{R} \tag{3.1}$$

1) **A** = variable A is the ratio of base plan acceleration which is different in disparate regions with specific seismic characteristics. The regions are categorized and named in four different regions with very high relative danger, high relative danger, medium relative danger and low relative danger with participation ratio of 0.35, 0.30, 0.25, and 0.2 respectively. (2800 earthquake code, page 26).

2) **B** = variable B is identified as the reflection coefficient of the structure, this number elaborates the type of structure response to the earth's movement and can be calculated according to the below formula which can be found in 2800 earthquake code(page 26)

$$B = 1 + S\frac{T}{T_0} \qquad 0 \le T \le T_0 \tag{3.2}$$

$$B = S + 1 \qquad T_0 \le T \le T_s \tag{3.3}$$

$$B = (S+1)\ \left(\frac{T_s}{T}\right)^{\frac{2}{3}} \qquad T \ge T_s \tag{3.4}$$

3) **I** = variable I describes the importance of the structure and this variable again splits into four different segment that are as following from most important to least important (2800 earthquake code (pages 18 and 31)) :

Table 3.1: Importance factor of Buildings (I)

I factor groups	Group 1	Group 2	Group 3	Group 4
Quantity	1.4	1.2	1	0.8

4) **R** = variable R describes the behavior of the structure and its quantity varies on the basis of structure height and the different frame systems that are used in the structure (2800 earthquake code (page 34)).

Table 3.2: Behavior factor

Earthquake Coefficient of Stories	C Factor
4 _ Stories	0.107
8_Stories	0.099
12_ Stories	0.0806

3.1.10Typical load combinations used for linear analysis

I) 1 Dead load + 0.75 live + 0.8 Elx

II) 1 Dead load + 0.75 live _ 0.8 Elx

III) 1 Dead load + 0.75 live _ 0.8 Elx and, IV) 1 Dead load + 0.75 live + 0.8 Elx

Chapter 4

RESULTS AND DISCUSSION

4.1 Design Results

The process of frame modeling and their analysis were finished, and then they were designed according to AISC LRFD (1999) and therefore the outcomes are rendered below in three disparate segments for 4, 8 and 12 story structures. The results below are relevant to the three different types of 4, 8, 12 -story braced frames as following:

4.1.1 Design Results of 4 Story frames

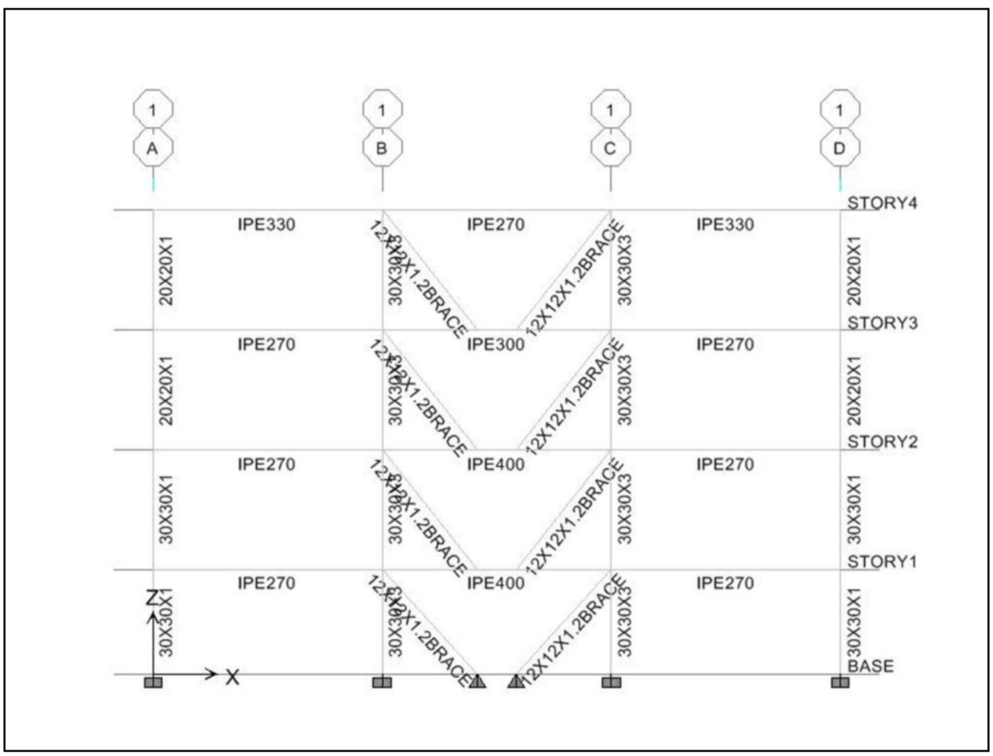

Figure 4.1: 4 story structure with Eccentric V bracing system

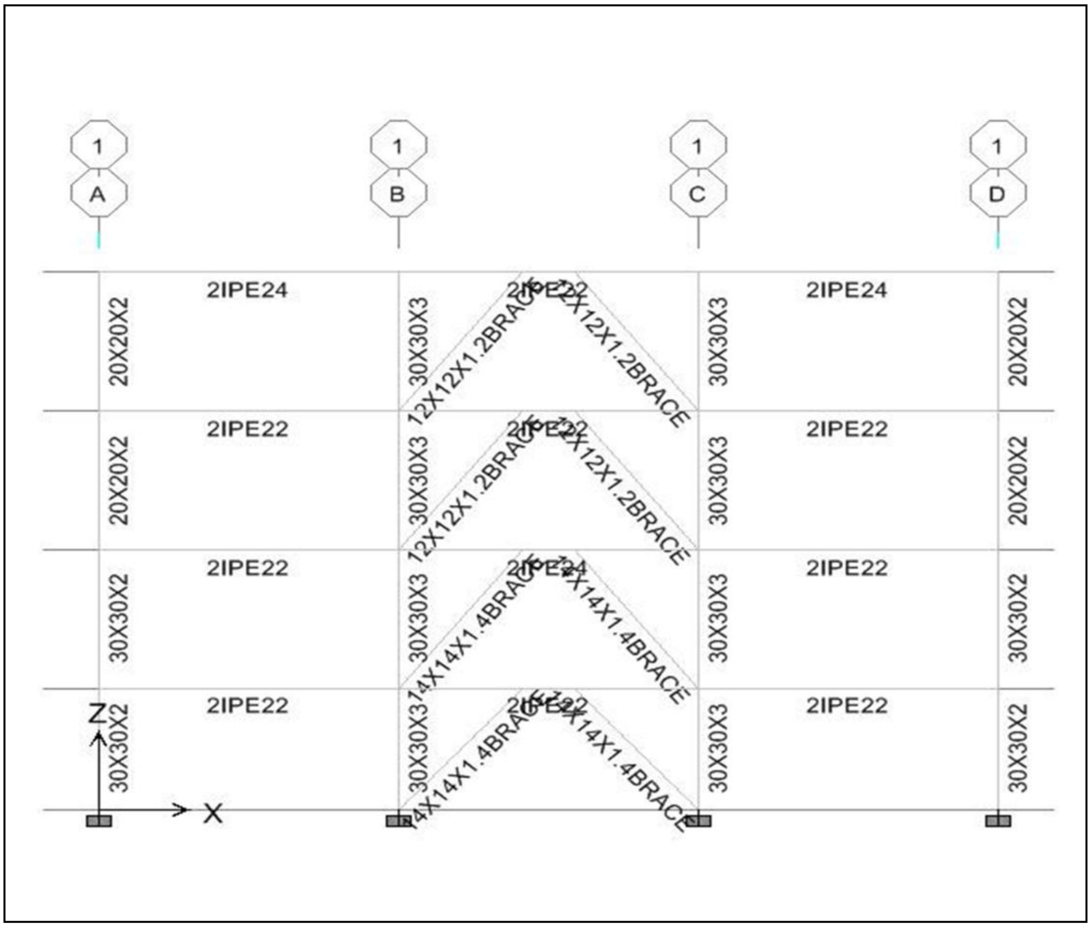

Figure 4.2: 4 story structures with Eccentric inverted _V bracing system

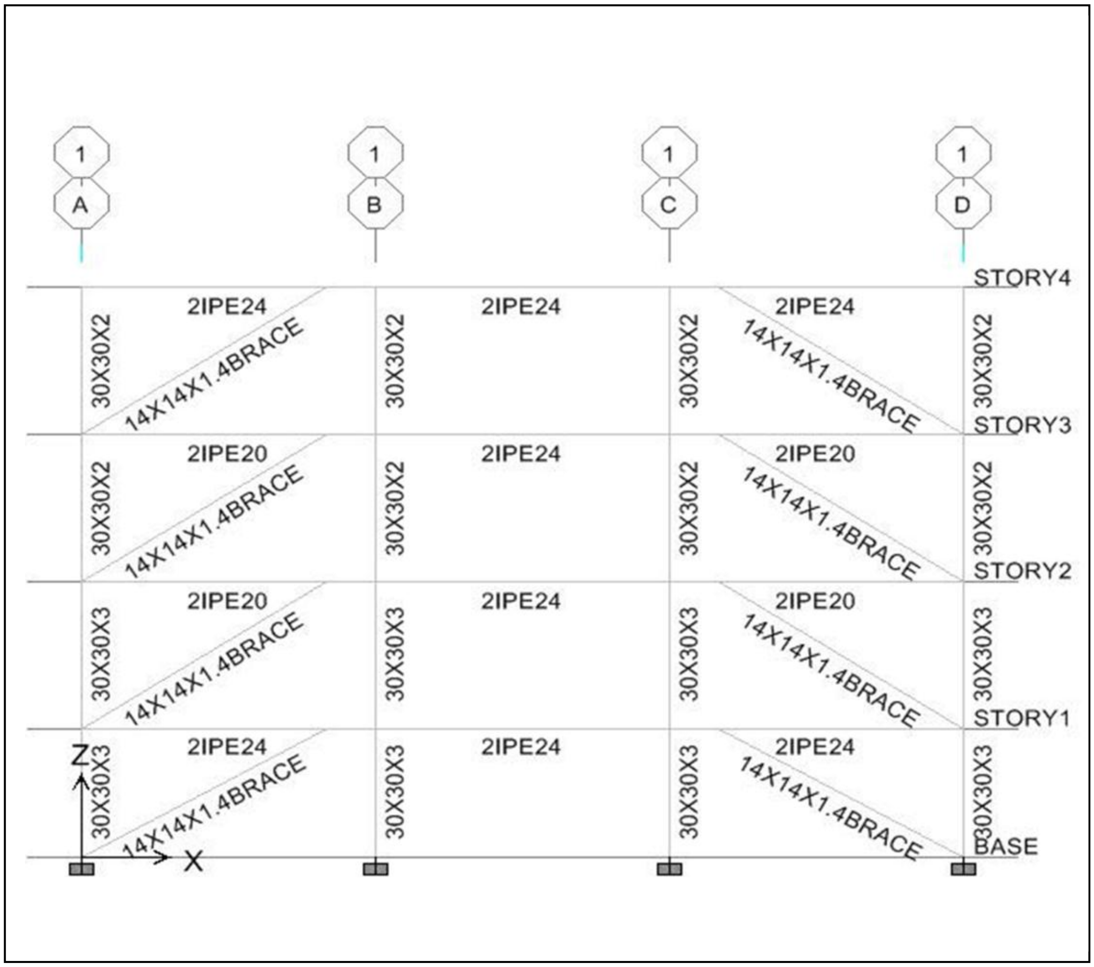

Figure 4.3: 4 story structure with Eccentric Diagonal bracing system

4.1.2 Design Results of 8 -story frames

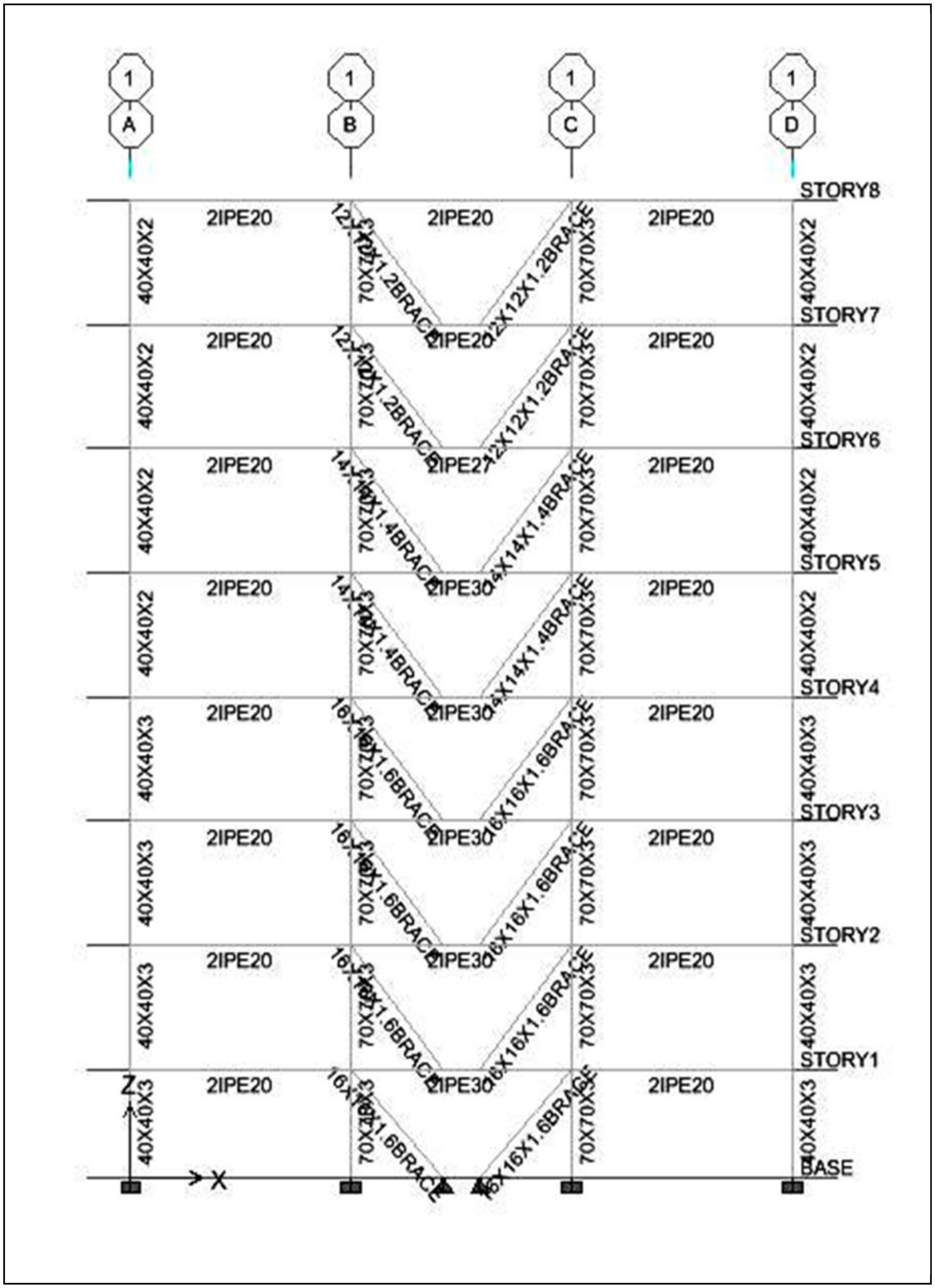

Figure 4.4: Performance of the 8 story structure with Eccentric V bracing system until target displacement

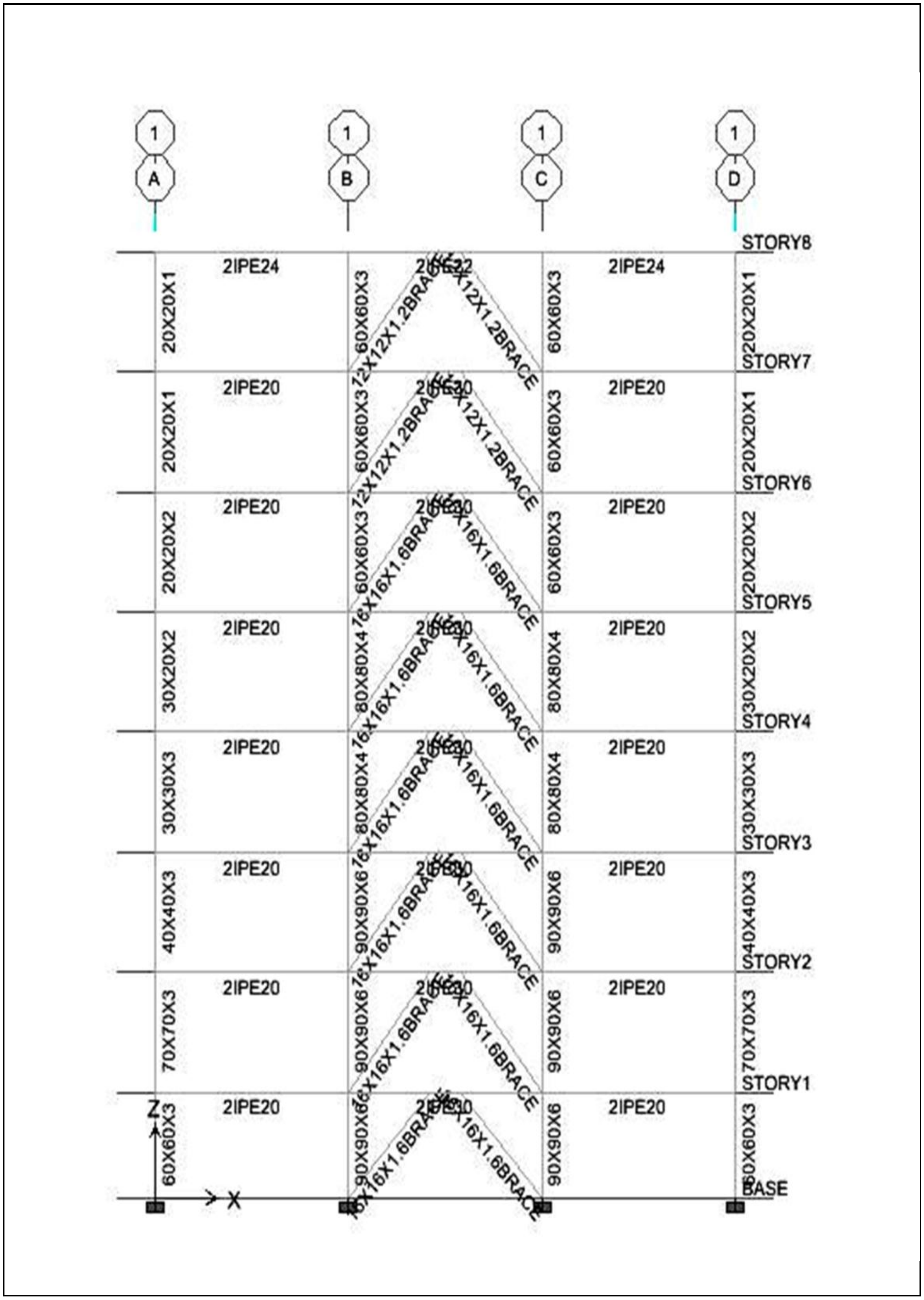

Figure 4.5: 8 story structure with Eccentric inverted _V bracing system

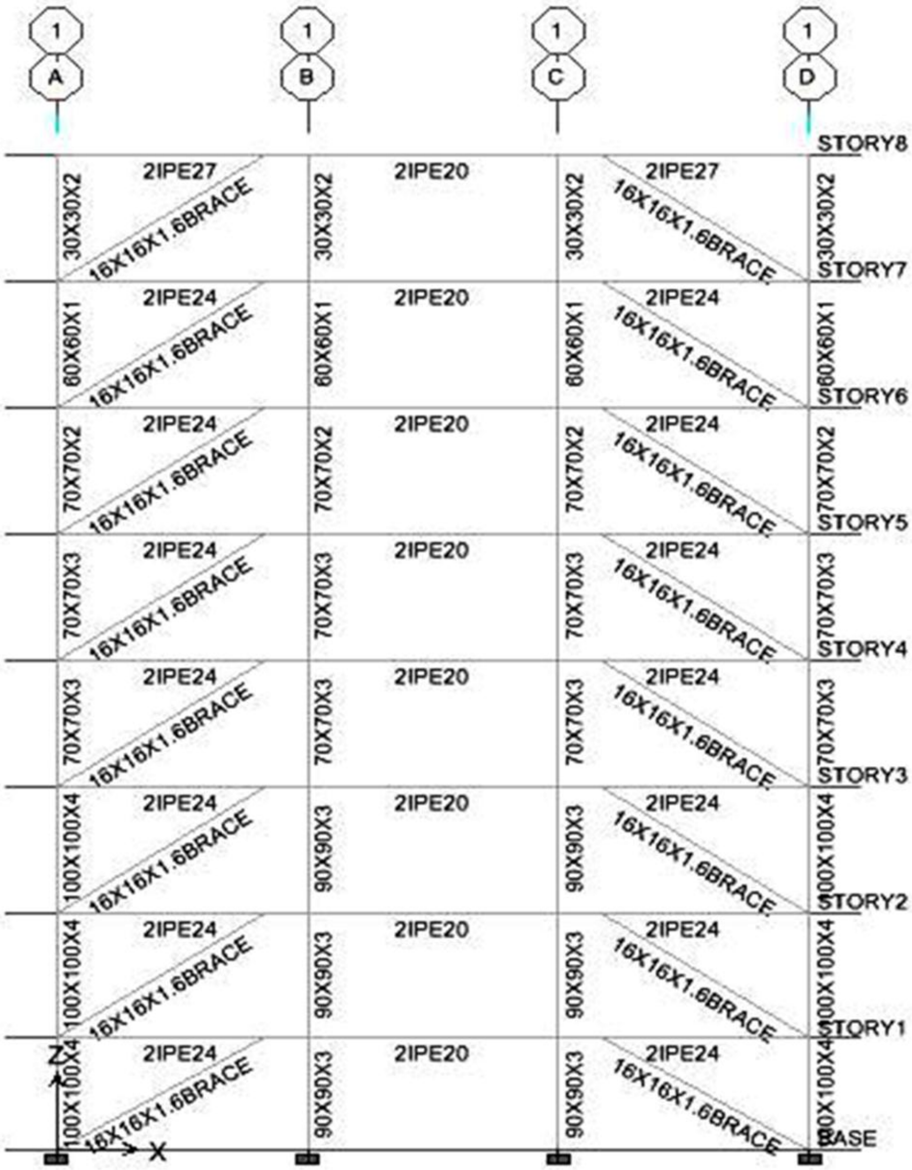

Figure 4.6: 8 story structure with Eccentric Diagonal bracing system

4.1.3 Design Results of 12 -story frames

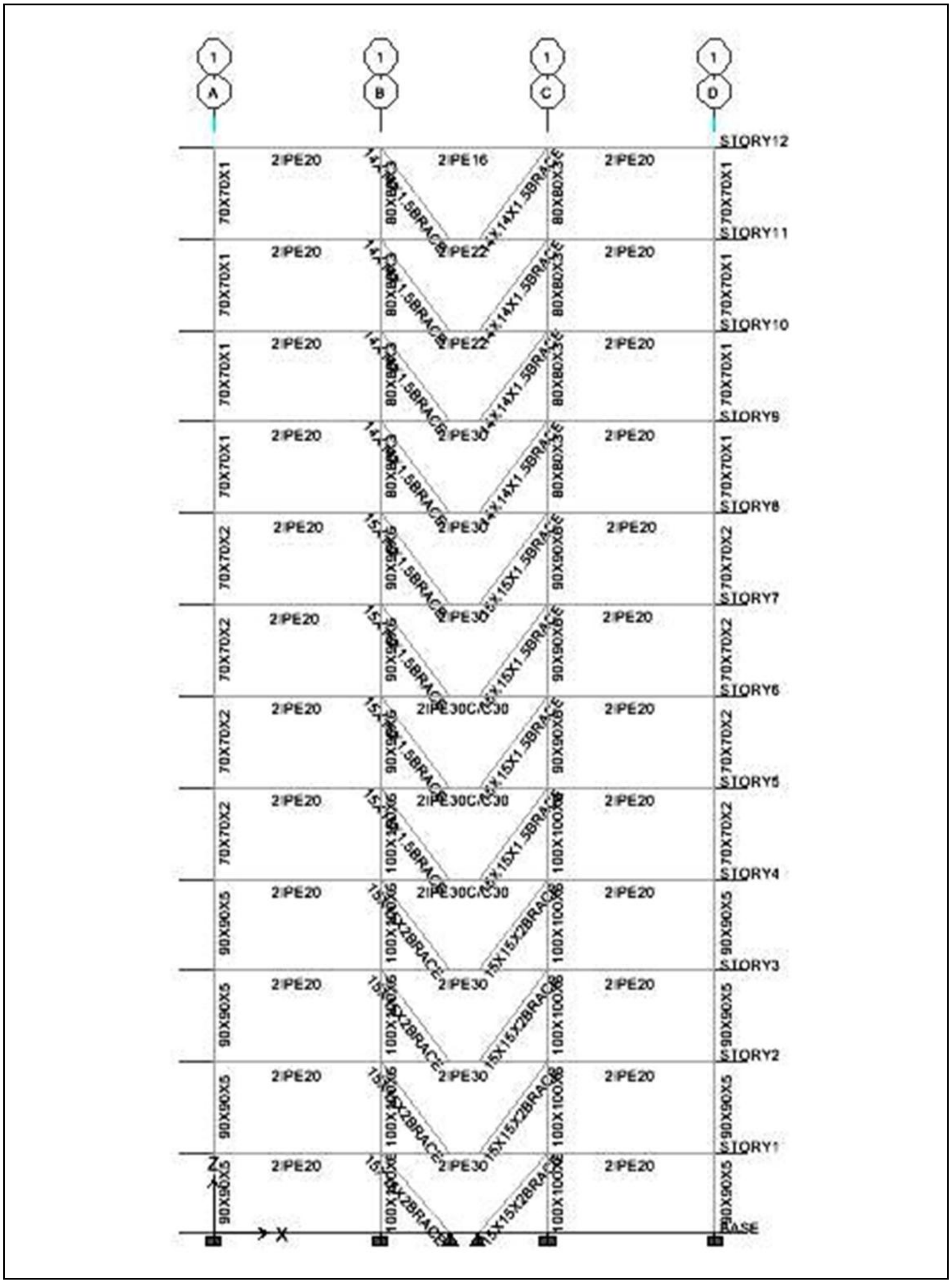

Figure 4.7: 12 story structures with Eccentric V bracing

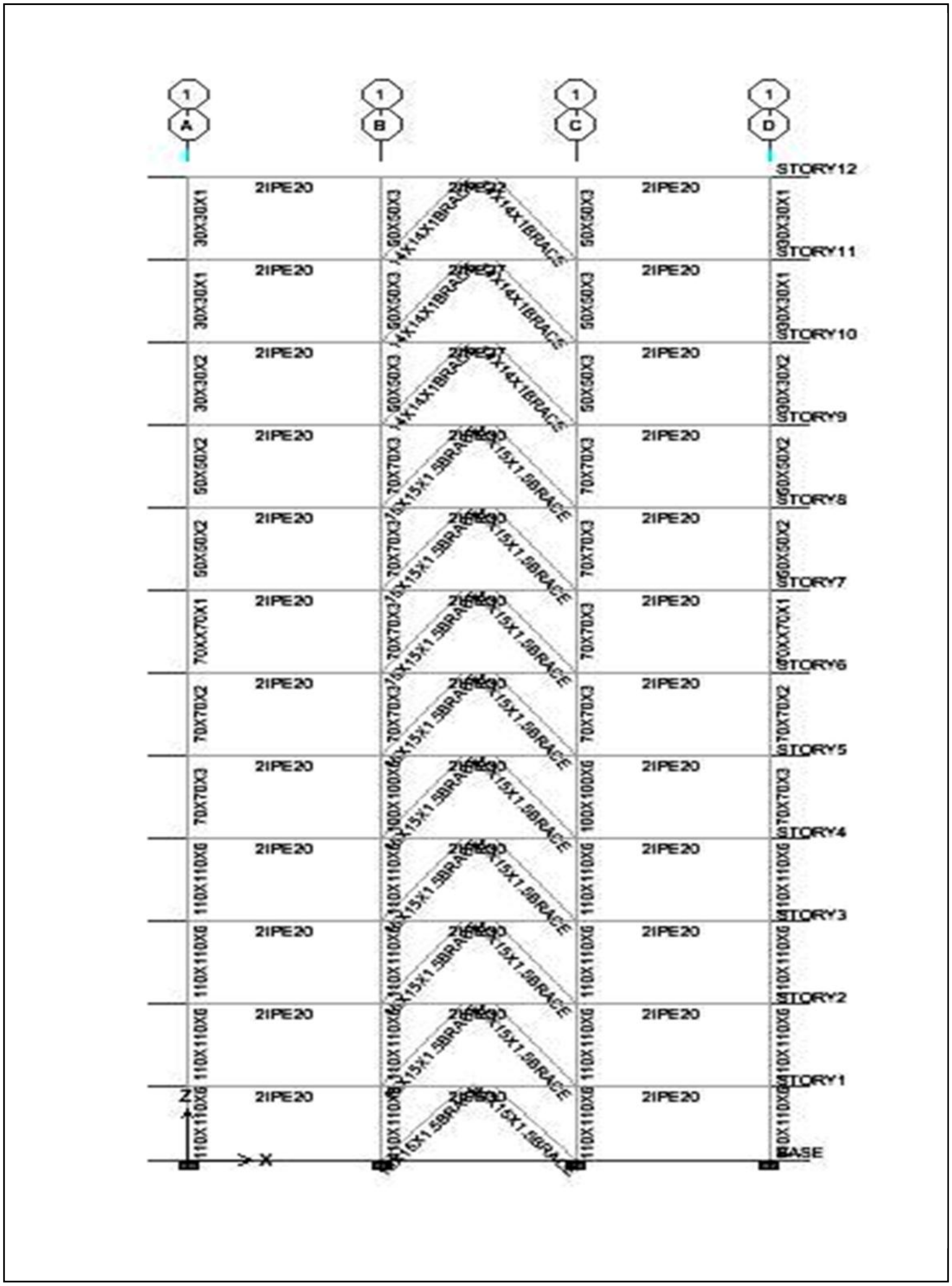

Figure 4.8: 12 story structure with Eccentric inverted -V bracing system

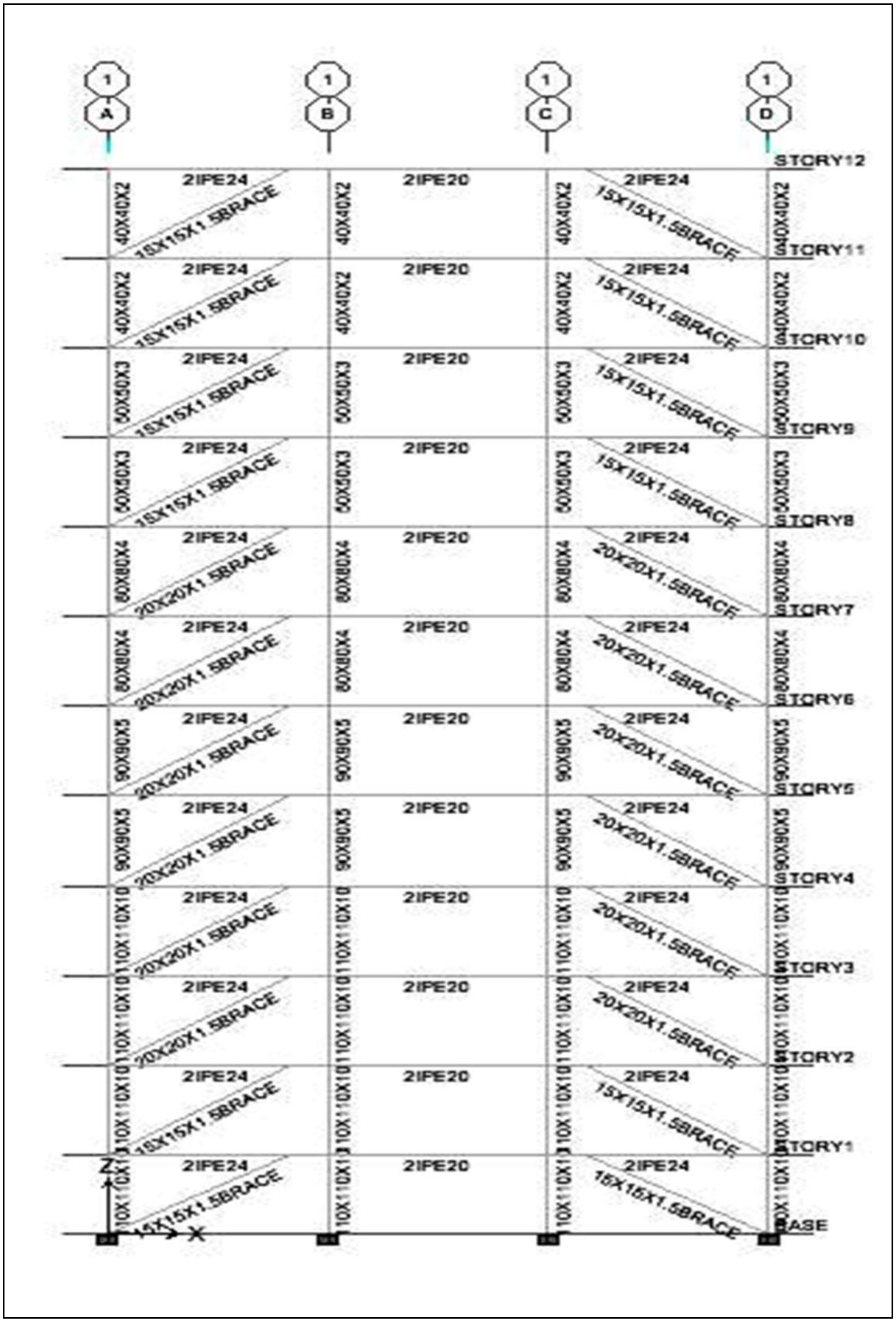

Figure 4.9: 12 story structures with Eccentric Diagonal bracing system

4.2 Pushover Analysis

4.2.1 Assessment of Nonlinear Behavior

Structural response curve is the key point that is considered as a paragon to evaluate the nonlinear parameters of a structure. These parameters, such as response modification factor, over strength factor and displacement amplification factor, can all be extracted from the pushover curve of the frame with mathematical equations.

4.2.2 Choice of the Method of Analysis

Among the four methods of linear static, linear dynamic, nonlinear static and nonlinear dynamic analysis, the third one, nonlinear static (pushover) procedure, has been chosen. The reasons for this choice are explained below.

Linear procedures, either static or dynamic are not suitable for this study since they directly deal with the nonlinear behavior of the frames for ductility assessment. In accordance with FEMA 356, linear procedures can be used only with acceptable accuracy while the structure behaves completely elastically.

Since linear procedures cannot be used for this research, the option remains between the two procedures i.e. nonlinear static (pushover) and dynamic procedures. The former one takes less computing time for analysis, if the site characteristics of a special place are not taken into consideration directly, while the second one takes more computing time for analysis where the characters such as: site, fault and earthquake are taken into consideration. It should be emphasized that the target of this research is to evaluate the eccentric bracing systems, without considering the site, fault and earthquake characteristics at a specific area. 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) but only with some alterations used for the analysis. 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.

4.2.3 Software selection for Computer Analysis

Different kinds of computer programs have the ability to perform pushover analysis on the structures and frames. ETABS and SAP 2000 are both the most well-known and widely accepted software for doing nonlinear analysis and they are powerful enough to do such analysis and provide reliable results.

4.2.4 Pushover Load Pattern

The target displacement has been opted at the highest point of the structure which is the roof by exerting the inverted triangular form of loading to the structure, the frames is pushed until they reach the pre-determined target displacement.

4.2.5 Displacement-Based Pushover Analysis

There are two methods for pushover analysis; Force-based and Displacement-based approaches. Between these two approaches, the second one is more accurate for frames that have high ductility. If the frame has low ductility and it is considered not ductile then, the second approach cannot be used for pushover analysis by using computer. As a result the push over analysis should be done with force-based approach even if it has little accuracy. It is good to add and emphasize that the ETABS is able to adjust the displacement increments for the purpose of minimizing the disparity of the pushover curve in comparison with the real nonlinear structural response. In this study the displacement-based pushover analysis is used.

4.2.6 Nonlinear Material Property

The nonlinear material property has been selected as the ETABS default because it bodes a ductile (and thus proper for nonlinear analysis) and widely used behavior in accordance with 1994 AISC LRFD Manual of Steel Construction. This is also in line with Inel and Ozmen (2006). According to Kim and Choi (2005) P-Delta effects were taken into consideration in order to get more accurate results.

4.2.7 Failure Criteria

According to FEMA 356, 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) and Arash Farzam (2009) in their study. In this study, the same failure mode has been exerted.

4.2.8 Plastic Hinge Properties

Comprehensive and complete information about plastic hinge properties of all of the structural segments are rendered by Federal Emergency Management agency in their Table, that are fulfilled by engineers throughout the world. All the information relevant to this table are at disposal as default hinge properties in ETABS software.

4.2.9 Column Hinge Properties

In accordance with FEMA 356, occurrence of a plastic hinge in a column is as a result of the interaction amongst axial force (P), moment in the stronger (M2) and weaker (M3) direction of the section. Therefore, interaction of P-M2-M3 is exerted to illustrate plastic hinges at the two ends of the columns (beginning and ending positions) that are in fact considered as the junction points with the other structural elements (Table 5-6 of FEMA 356).

4.2.10 Brace Hinge Properties

Nonlinear behavior of brace elements can be best modeled by assuming a hinge (being made under pure axial load) in the middle of the element. An axial load plastic hinge is modeled in the 0.5 relative distances of all bracing elements as per Table 5-6 of FEMA 356 [Appendix] in this study.

4.2.11 Beam Hinge Properties

Considering the fact that the beam to column connections is rigid, two plastic hinges (one at the beginning and the other one at the end) will be obtained. But for the beams that are braced with eccentric braces, the plastic hinges will occur at the place of fuses. For these kinds of beams the M_3 and V_2 are taken into consideration.

4.3 Idealization of Pushover Curve

In order to find the parameters of the nonlinear behavior of the frame, according to Kim and Choi (2005) and Maheri and Akbari (2003) the virtual push over curve by a bilinear curve. Inclusive information is available associated with idealization in FEMA 273, 356 and 440 coefficient based approach of performance-based process. Kim and Choi (2005) had exerted FEMA 356 approach in their research. On the other hand, as FEMA 440 has later altered this approach, then the modified coefficient-based approach of FEMA 440 is exerted in this thesis. Figure 4.10 illustrates the idealized bilinear response curvature against virtual pushover curve of a frame.

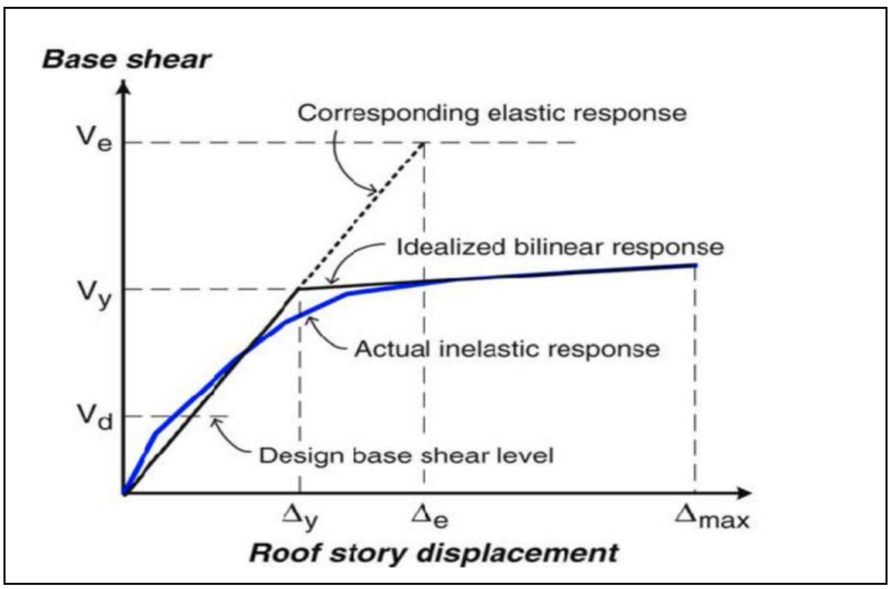

Figure 4.10: Idealization curve

Table 4.1: Plastic hinge levels

●	Minor
▲	Immediate Occupancy
✚	Life Safety
◆	Collapse Prevention
★	Collapse

4.3.1 Target Displacement

As it was explained in the previous sections, the estimation of target displacement point plays a significant role in the coefficient based performance-based method. Different documents and studies have mentioned various relationships for the calculation of target displacement point. FEMA 356 introduced the equations 4.1 to 4.7 for this task and they are summarized in FEMA 440:

$$\text{Target Displacement } (\delta) = C_0 \; C_1 \; C_2 \; C_3 \; S_a \; \frac{{T_e}^2}{4 * \pi^2} \, g \tag{4.1}$$

Where:

C_0: "Modification factor to relate spectral displacement of an equivalent SDOF system to the roof displacement of the building MDOF system". There are several ways of calculating C0, the easiest of which is using the appropriate value from Table 3-2 of FEMA 356.

C_1: "Modification factor to relate expected maximum inelastic displacements to displacements calculated for linear elastic response" This coefficient should be calculated from the following equations 4.2 to 4.5.

a) $= 1$ $\qquad T_e \geq T_s$ (4.2)

b) $= [1 + (R - 1)\,T_s / T_e]/R$ $\qquad T_e < T_s$ (4.3)

But not greater than:

a) $= 1$ $\qquad T_e \geq T_s$ (4.4)

b) $= 1.5$ $\qquad T_e < T_s$ (4.5)

And, nor less than 1.

Te: "Effective fundamental period of the building in the direction under consideration, in seconds" which should be calculated in compliance with section 3.3.3.2.5 of FEMA 356.

Ts: "Characteristic period of the response spectrum, defined as the period associated with the transition from the constant-acceleration segment of the spectrum to the constant- velocity segment of the spectrum."

R: "Ratio of elastic strength demand to calculated yield strength coefficient"

$$R = \frac{S_a}{V_{y/W}} * C_m \tag{4.6}$$

C_2: "Modification factor to represent the effect of pinched hysteretic shape, stiffness degradation and strength deterioration on maximum displacement response." C2 value should be obtained from table 3-3 of FEMA 356.

C_3: "Modification factor to represent increased displacements due to dynamic P-Δ effects."

$$C_3 = 1 + \frac{|\propto|(R-1)^{3/2}}{T_e} \tag{4.7}$$

S_a: "Response spectrum acceleration, at the effective fundamental period and damping ratio of the building in the direction under consideration"

g : "acceleration of gravity"

$\propto$: "Ratio of post-yield stiffness to effective elastic stiffness, where the nonlinear force displacement relation shall be characterized by a bilinear relation."

Table 4.2: All the calculated criteria associated with the target displacement

Target Displacement criteria	C_0	C_1	C_2	C_3	S_a	T_e (sec)	δ (cm)
4Story,V_braced	**1.35**	**1.15**	**1**	**1**	**0.75**	**0.52**	**8.10**
4story,In_V_braced	**1.35**	**1.05**	**1**	**1**	**0.75**	**0.64**	**11.10**
4 story,Dia_braced	**1.35**	**1.00**	**1**	**1**	**0.75**	**0.46**	**5.50**
8 Story,V_ braced	**1.45**	**1.00**	**1**	**1**	**0.69**	**0.77**	**15.10**
8story,In_V_braced	**1.45**	**1.00**	**1**	**1**	**0.69**	**0.81**	**16.60**
8 story,Dia_braced	**1.45**	**1.02**	**1**	**1**	**0.69**	**0.64**	**10.66**
12 Story,V_ braced	**1.5**	**1.08**	**1**	**1**	**0.56**	**1.06**	**26.00**
12story,In_V_braced	**1.5**	**1.00**	**1**	**1**	**0.56**	**1.16**	**28.54**
12 story,Dia_braced	**1.5**	**1.00**	**1**	**1**	**0.56**	**0.86**	**15.63**

4.4 Assessment of Bracing systems

The plastic hinges of all frames until reaching to the target displacements for each frame are shown in Figures 4.11 to 4.19 Also the level of the plastic hinges in which they happened in accordance with FEMA-356. Then the Idealized curve of each frame has been demonstrated and also a comparison amongst identical bracing system in different height has been conducted, as well as the same comparison amongst disparate bracing systems with the same height. All the comparison is conducted on the basis of each frame's Idealized curve as the paragon of the study.

4.4.1 Frames Behavior until Target Displacement

In this frame (Figure 4.11), three plastic hinges happened in the range of collapse level at the compressive member of the bracing system of the first, second and third floor. One plastic hinge occurred before IO level in the fuse segment of beam of the second floor. The target displacement point in this frame is 8.1 cm with 9.42×10^3 Kgf as force.

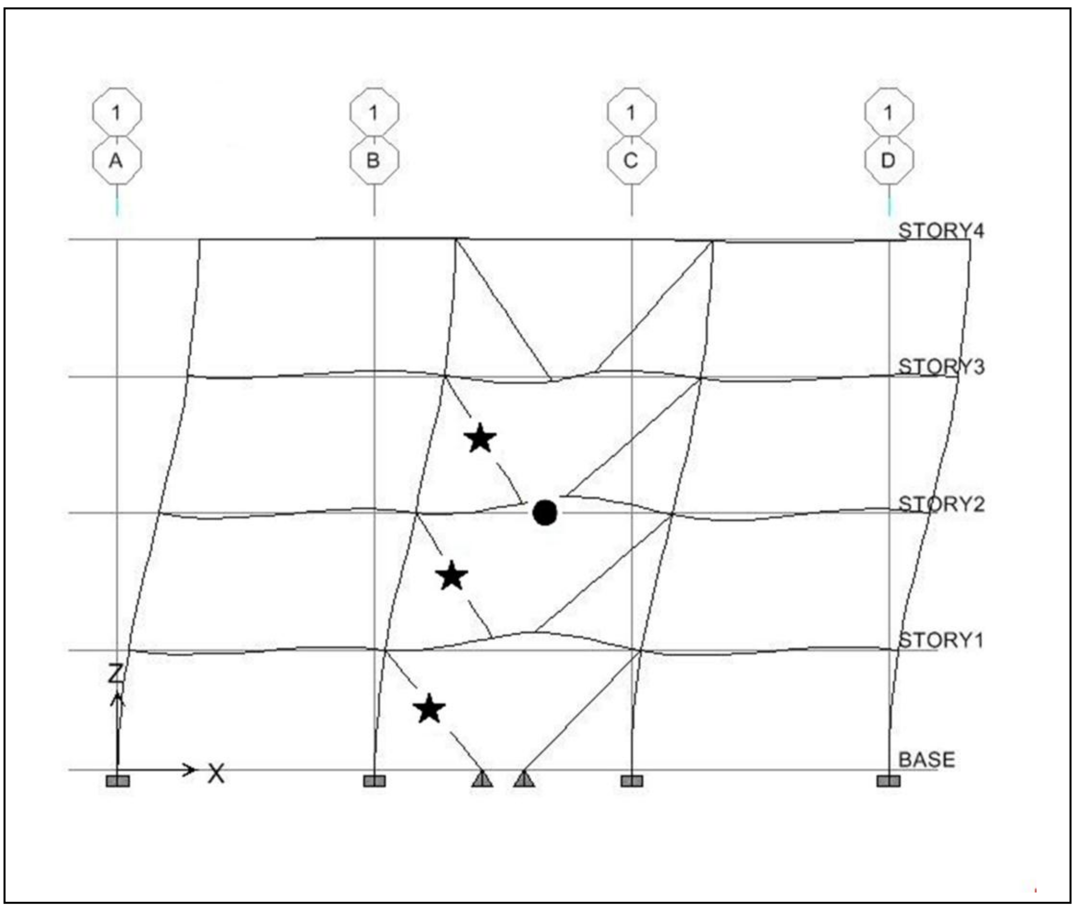

Figure 4.11: Performance of the 4 story structure with Eccentric V bracing system until target displacement

In this frame (Figure 4.12), one plastic hinge occurred before IO level in the fuse segment of beam in first floor. Two plastic hinges happened in the range of B_IO level in the fuse segment of second and third floor. The target displacement point in this frame is 11.1 cm with 14.2× 10^3 Kgf as its force.

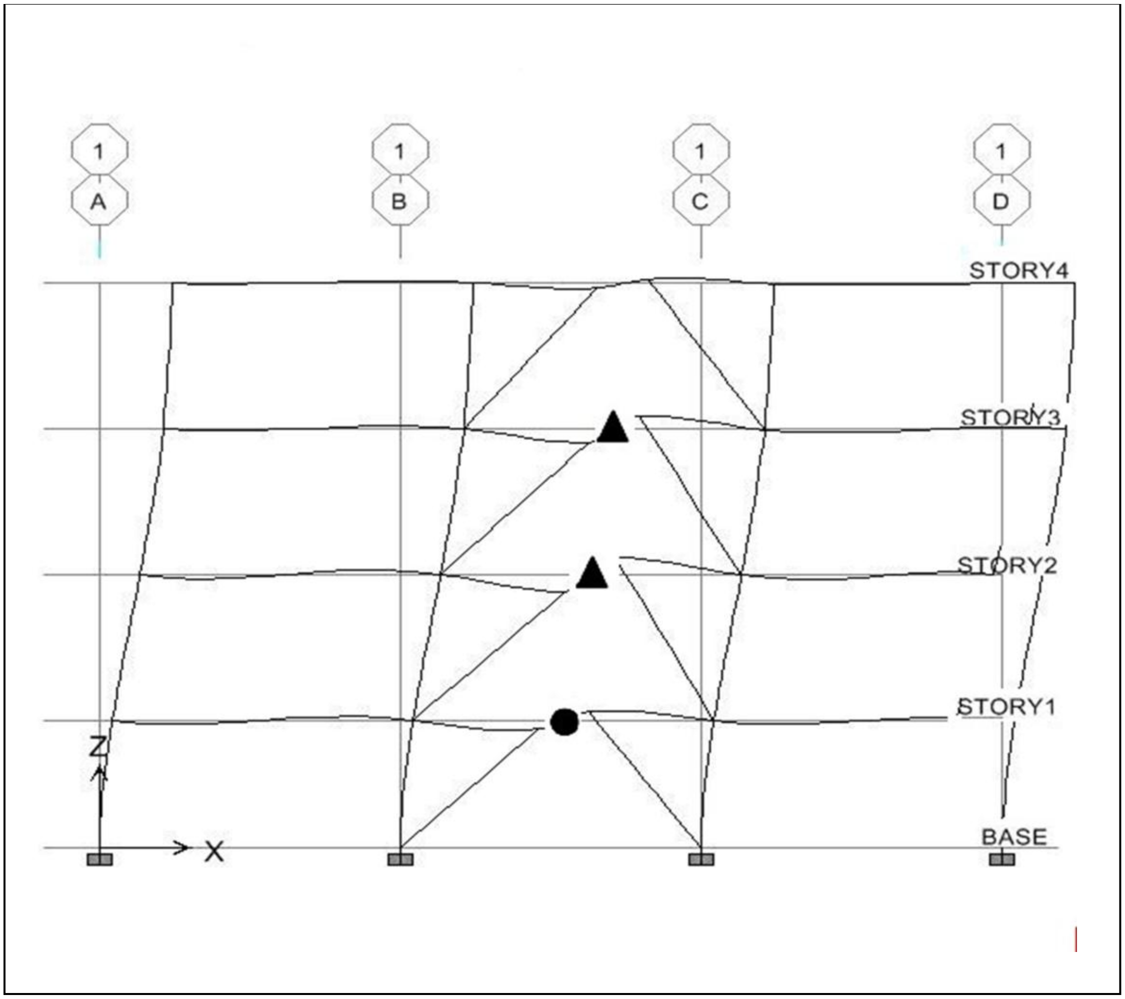

Figure 4.12: Performance of the 4 story structure with Eccentric inverted _V bracing system until target displacement.

In this frame (Figure 4.13), six plastic hinges that are before IO level happened which 4 of them are in the fuse of beams of first and third floor and the other two happened in the braces of first and second floor. Two plastic hinges happened in the range of B_IO level in the fuse segments of second floor beams. The target displacement point in this frame is 5.5 cm with 8.24× 10^3 Kgf as its force.

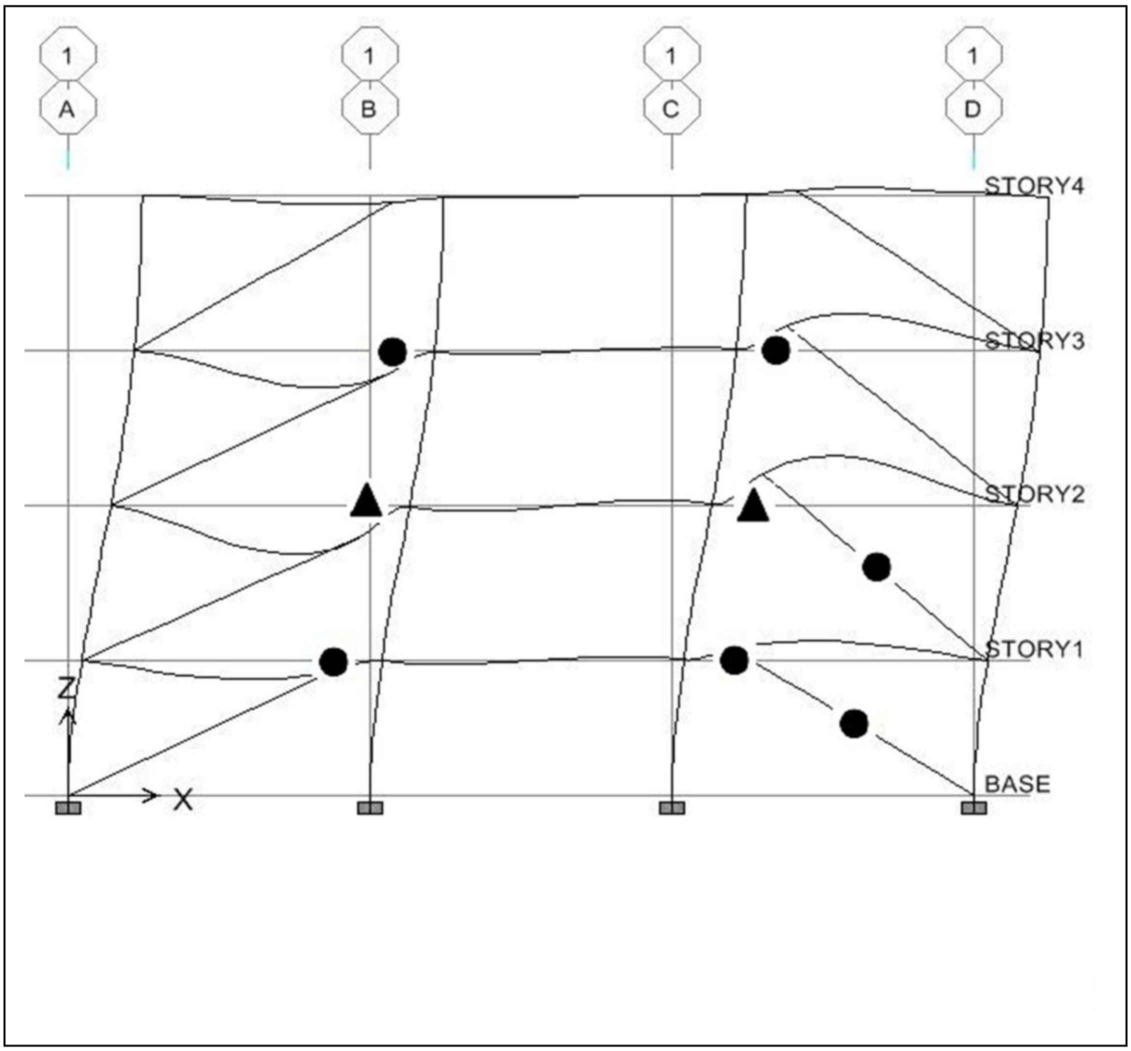

Figure 4.13: Performance of the 4 story structure with Eccentric Diagonal bracing system until target displacement

In this frame(Figure 4.14), 4 plastic hinges that are before IO level happened, which 3 of them are in the fuse segments of beams in first, fourth and fifth floor and the other one happened in the brace of seventh floor. Two plastic hinges happened in the range of B_IO level in the fuse segments of second and third floor beams. The target displacement point in this frame is 15.1 cm with 31.2× 10^3 Kgf as its force.

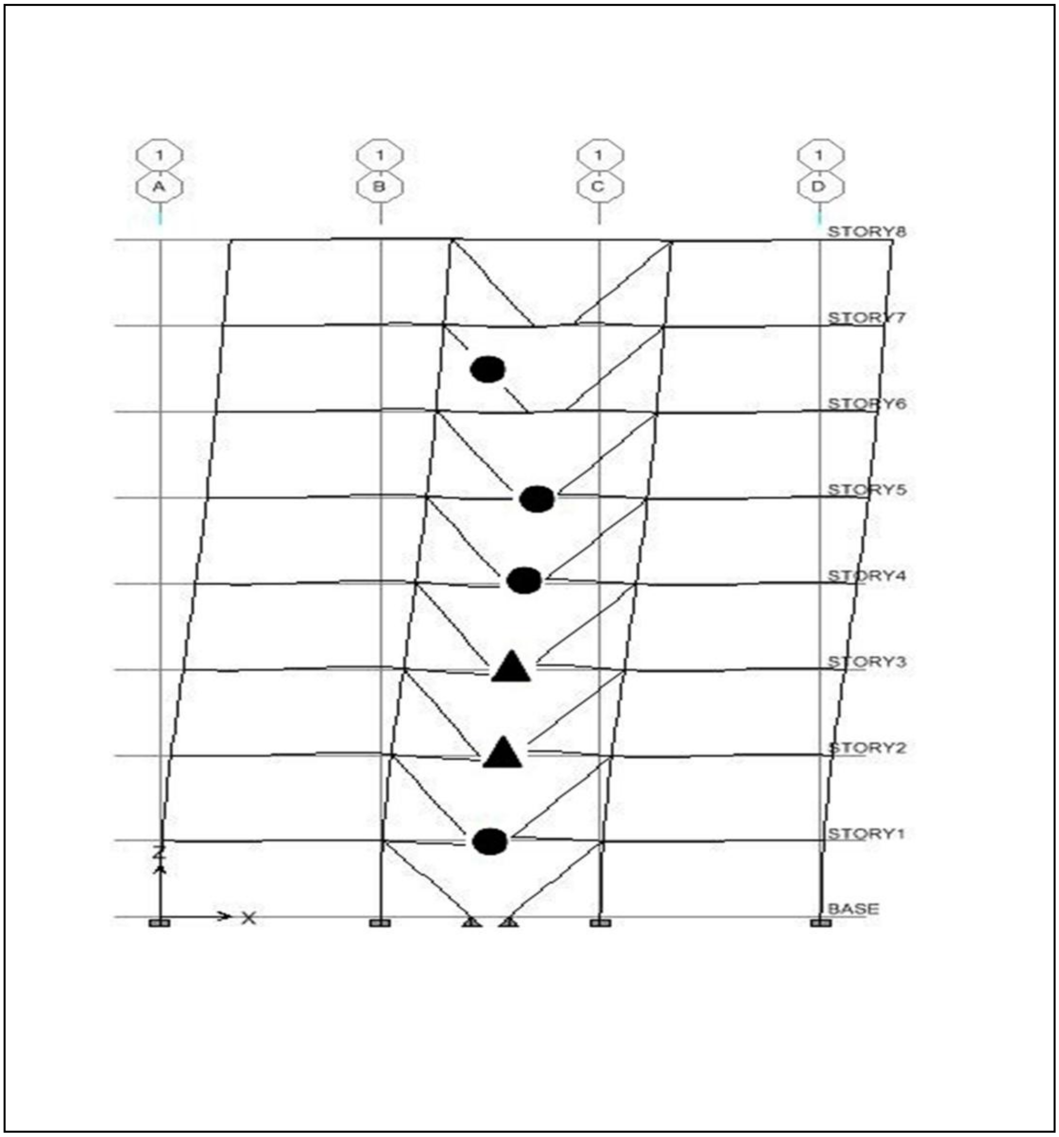

Figure 4.14: Performance of the 8 story structure with Eccentric V bracing system until target displacement

In this frame (Figure 4.15), 3 plastic hinges that are before IO level happened, which are in the fuse segments of beams in second, third and eighth floors. 3 plastic hinges happened in the range of B_IO level in the fuse segments of fourth, fifth and sixth floor. And finally one plastic hinge occurred in the range of collapse level in the seventh storey brace. The target displacement point in this frame is 16.6 cm with 32.83× 10^3 Kgf as its force.

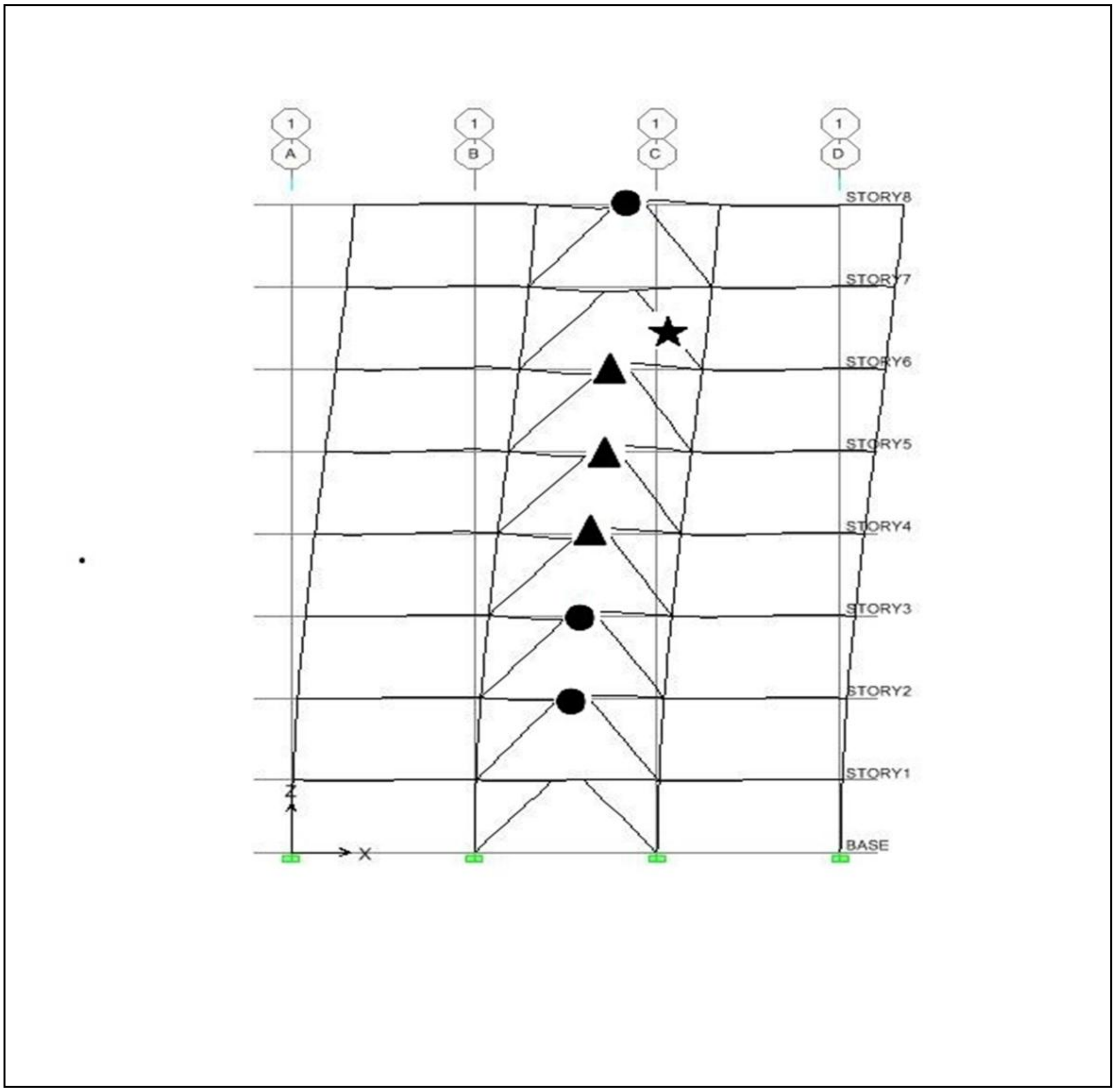

Figure 4.15: Performance of the 8 story structure with Eccentric inverted _V bracing system until target displacement

In this frame (Figure 4.16) 10 plastic hinges that are before IO level happened, which are in the fuse segments of beams in second, third, fourth, fifth and sixth floors. The target displacement point in this frame is 10.66 cm with 21.13×10^3 Kgf as its force.

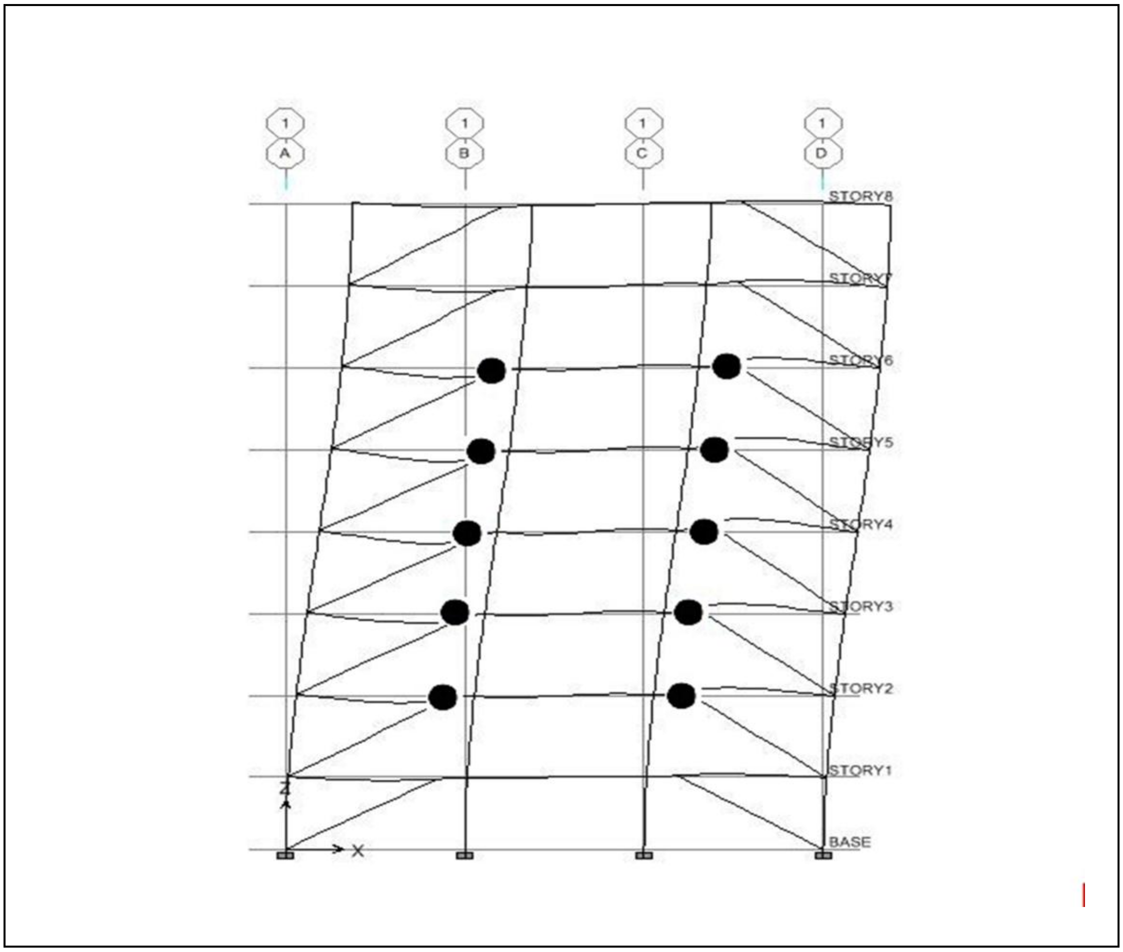

Figure 4.16: Performance of the 8 story structure with Eccentric Diagonal bracing system until target displacement

In this frame (Figure 4.17), 8 plastic hinges that are before IO level happened, which are in the fuse segments of beams in second till ninth floors. The target displacement point in this frame is 26 cm with 50.33× 10^3 Kgf as its force.

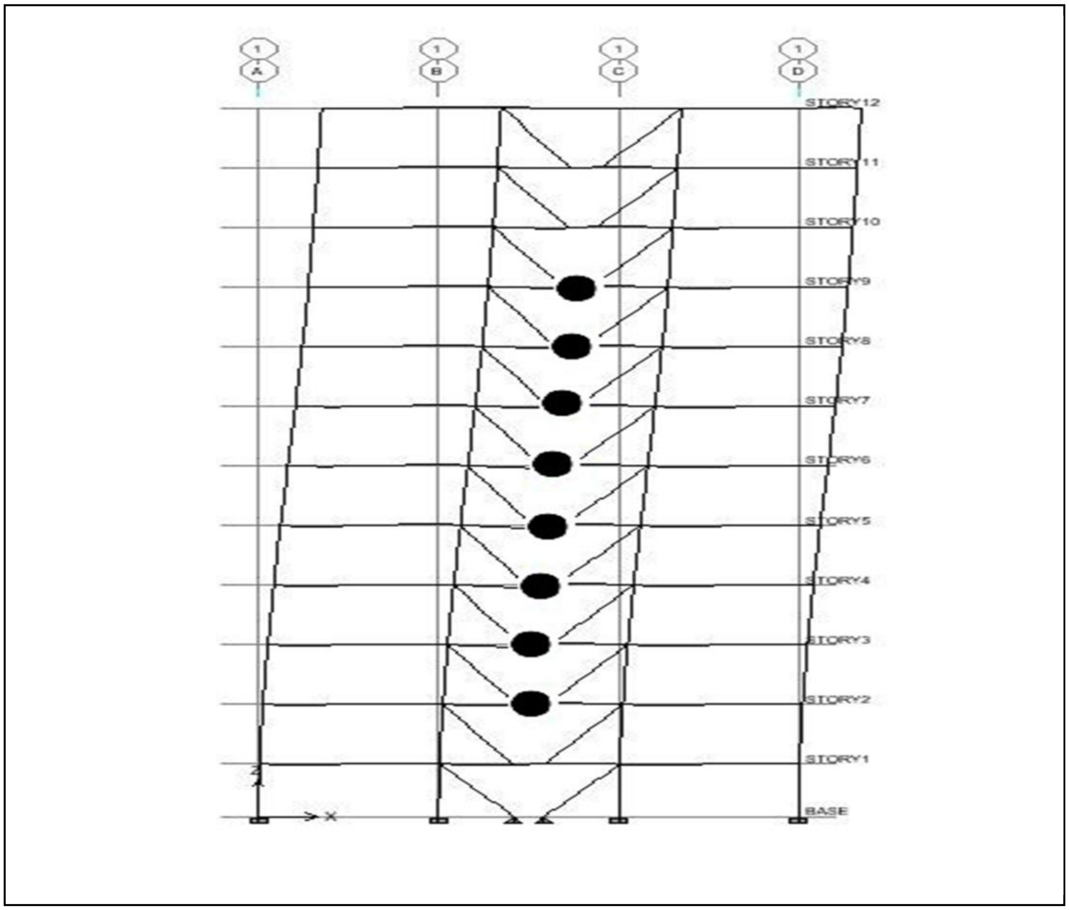

Figure 4.17: Performance of the 12 story structure with Eccentric V bracing system until target displacement

In this frame (Figure 4.18), 4 plastic hinges that are before IO level happened, which 3 of them are in the fuse segments of beams in third, fourth, eleventh floors and the other one happened in the bracing of tenth story. 6 plastic hinges happened in the range of B_IO level in the fuse segments of fifth to tenth floors. The target displacement point in this frame is 28.54 cm with 63.2× 10^3 Kgf as its force.

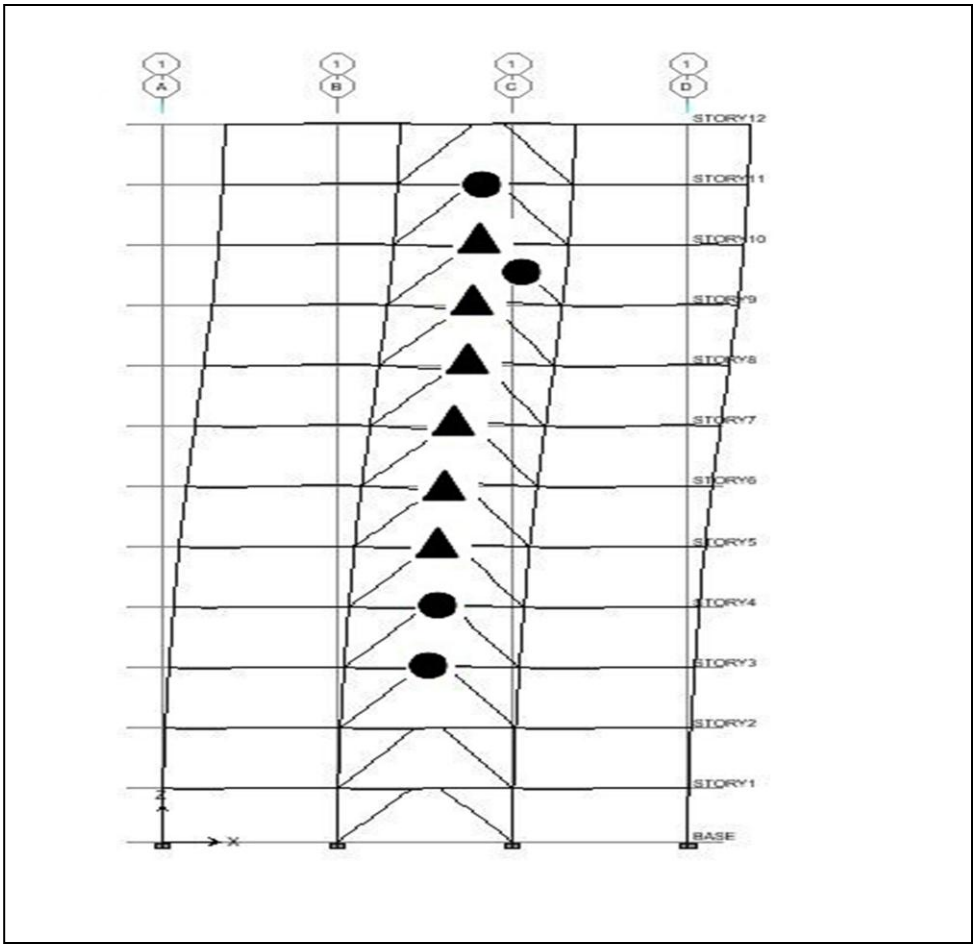

Figure 4.18: Performance of the 12 story structure with Eccentric inverted _V bracing system until target displacement

In this frame (Figure 4.19), 16 plastic hinges that are before IO level happened, which are in the fuse segments of beams in third to tenth floors. And one plastic hinges happened The target displacement point in this frame is 15.63 cm with 31.75×10^3 Kgfvin the range of C_D level in bracing system of ninth floor. The target displacement point in this frame is 15.63 cm with 31.75×10^3 Kgf as its

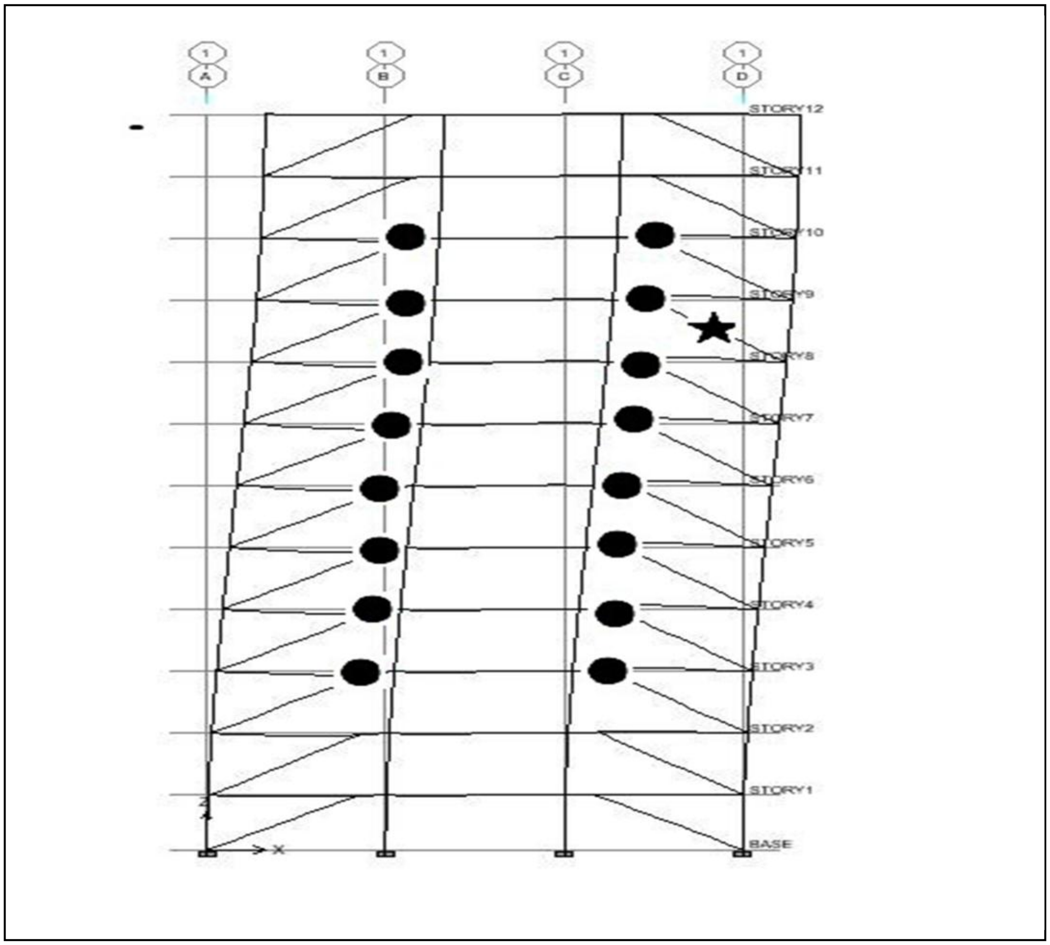

Figure 4.19: Performance of the 12 story structure with Eccentric Diagonal bracing system until target displacement

4.4.2 Comparison Among Idealized Curvatures:

According to the curves drawn in figures 4.20 to 4.22 it can be noted that the initial slope of elastic zone relevant to eccentric diagonal bracing system is more than the initial slopes of elastic zones for eccentric V and inverted-V bracing systems. Also the amount of the area under the curves in figures 4.20 to 4.22 is arranged respectively from greater to lower amount as the following: eccentric inverted-V, eccentric V, eccentric diagonal. The slope of the second linear part of the idealized curves in figures 4.20 to 4.22 bodes the capacity of energy absorption in the plastic zone. Then it can be inferred that the capacity of energy absorption for eccentric diagonal bracing system is more than eccentric V and eccentric inverted-V bracing system.

Regarding the curves in figures 4.23 to 4.25, which illustrate the behavior of each eccentric bracing system (eccentric V, eccentric inverted-V, eccentric diagonal) in different heights, it can be inferred that by increasing the number of stories then the indeterminacy degree becomes more and as a result the number of plastic hinges increased. So it can be observed that the more stories a structure has, the more energy will absorb. By comparing the idealized curves in figures 4.23 to 4.25 it can be induced that the initial stiffness of the four-story structure is more than eight-story structure and also in the same way the initial stiffness of eight-story structure is more than twelve-story structure. In accordance with the second part of the idealize curves it can be concluded that the ductility of the twelve-story building is more than eight-story building and also for the eight-story building is more than four-story building.

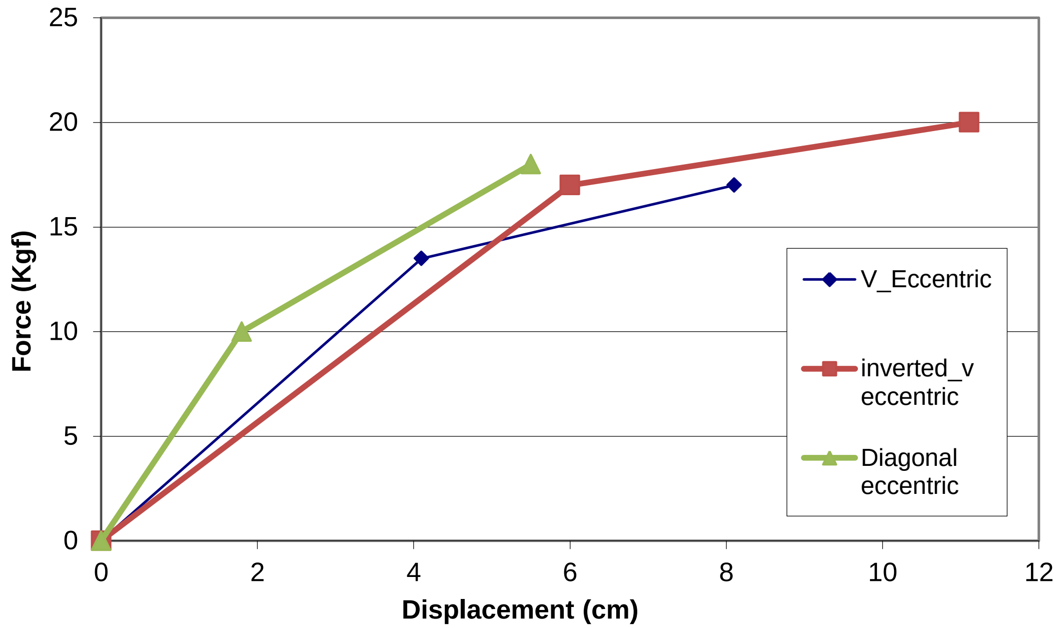

Figure 4.20: Comparison amongst the 3 three different kinds of bracing system of the four story structure

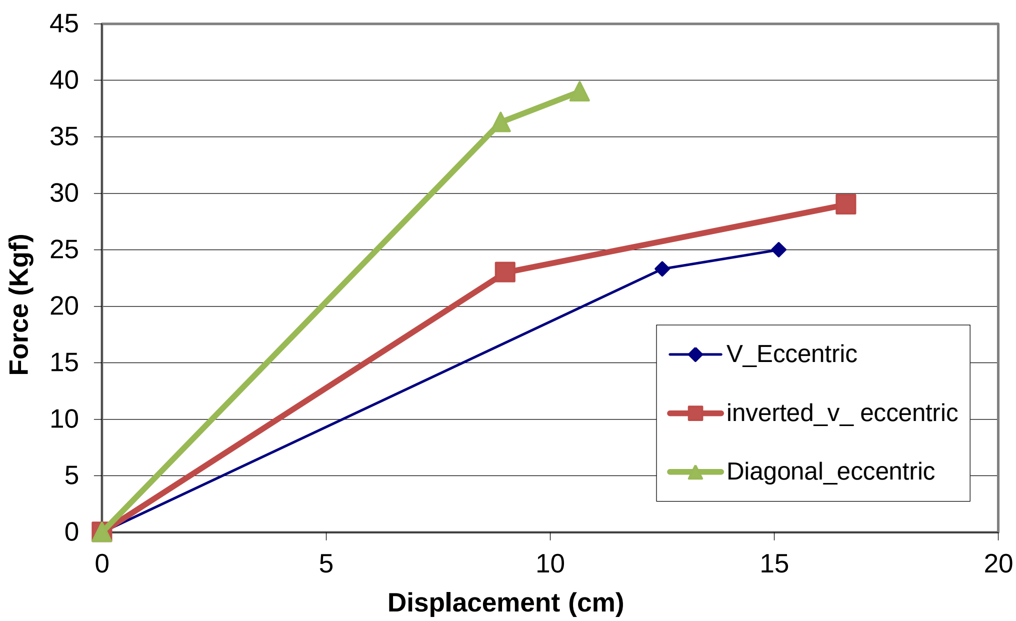

Figure 4.21: Comparison amongst the 3 three different kinds of bracing system of the eight story structure

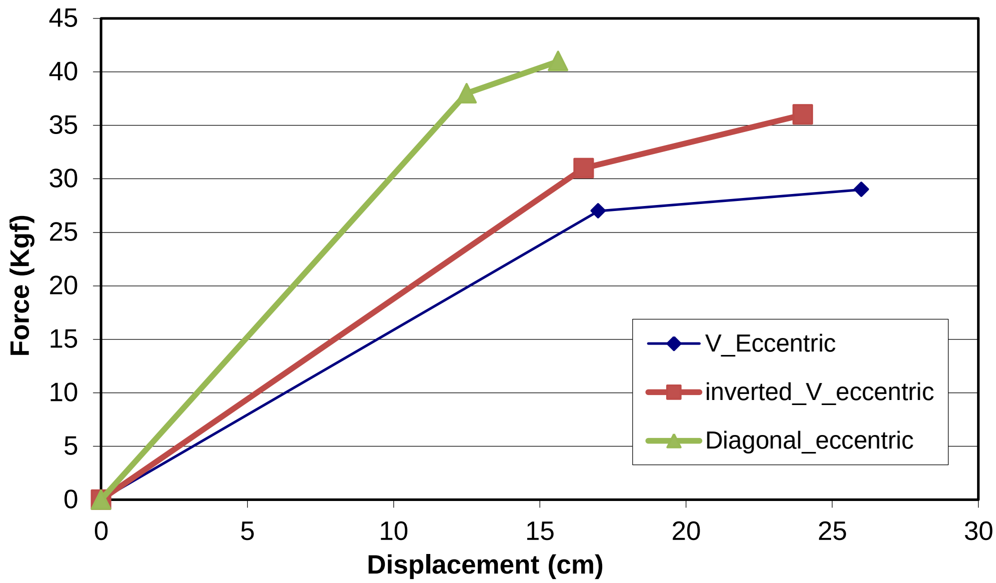

Figure 4.22: Comparison amongst the 3 three different kinds of bracing system of the twelve story structure

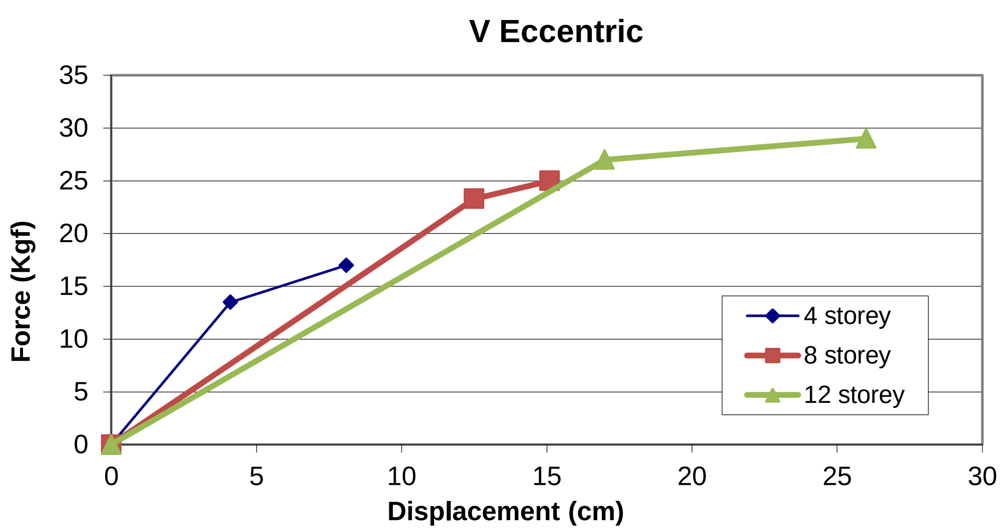

Figure 4.23: Comparison of Eccentric-V-bracing system amongst the 3 different heights of the buildings (4, 8, and 12 story)

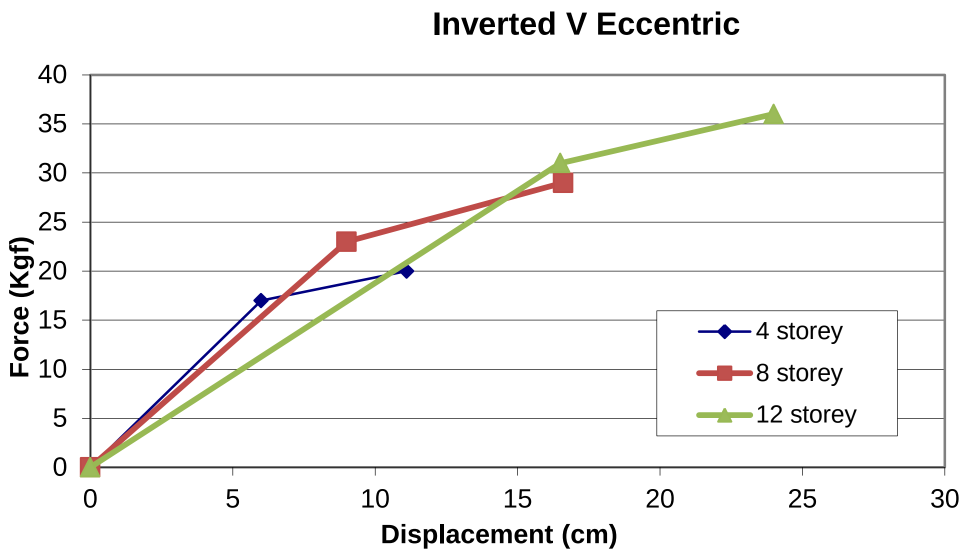

Figure 4.24: Comparison of Eccentric inverted –V-bracing system amongst the 3 different heights of the buildings (4, 8, 12 story)

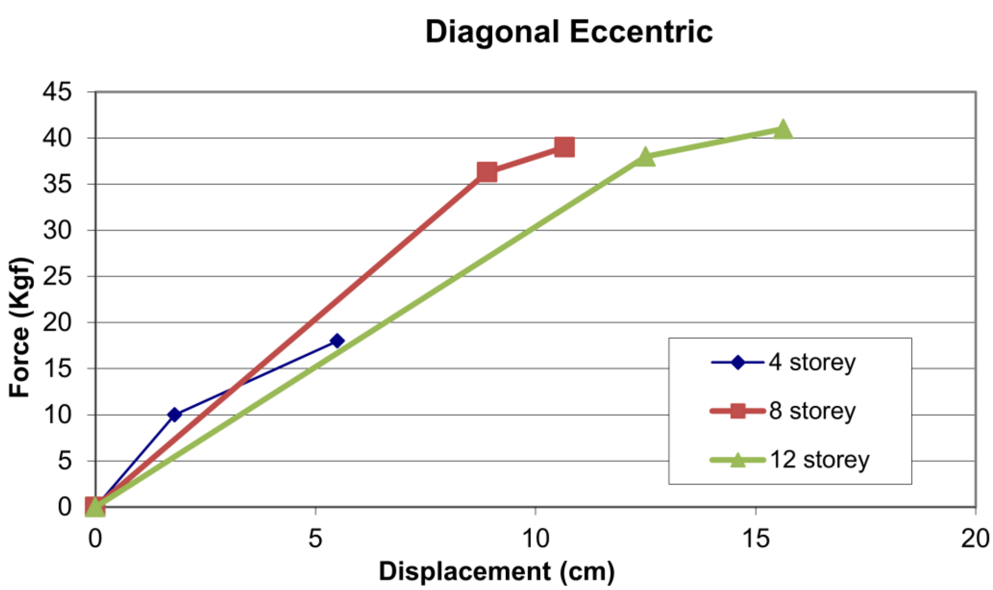

Figure 4.25: Comparison of Eccentric-Diagonal-bracing system amongst the 3 different heights of the buildings (4, 8, and 12 story)

4.4.3 Steel Weight for Different Frames:

In this section the weights of steel materials used in the frames are calculated separately and put in order in accordance with the heights of structures and also the types of eccentric bracing systems used in the frames.

Table 4.3: Total Weight Calculation of Each Frame

Number of frames	Explanation	Total weight (ton)
1	4 story, V-braced	12.70
2	4 story, I-V-braced	15.43
3	4 story, Diagonal-braced	17.36
4	8 story, V-braced	58.55
5	8 story, I-V-braced	78.37
6	8 story, Diagonal-braced	78.75
7	12 story, V-braced	173.64
8	12 story, I-V-braced	179.58
9	12 story, Diagonal-braced	261.54

Chapter 5

CONCLUSIONS

5 .1 Conclusion

After drawing the idealization pushover curves for the studied models and also conducting comparisons amongst them through the figures 4.20 to 4.25, in accordance with their disparity in height and the type of eccentric bracing system the following are the conclusions:

I) comparing the idealization curvature of frames with the same height but braced with different type of eccentric bracing system:

In the 4 story models which are braced with the three different types of eccentric bracing systems (Inverted-V, V, diagonal), on the basis of their initial slopes (in the idealization curvature) it can be observed that the eccentric Diagonal bracing system has more stiffness in comparison with eccentric Inverted-V bracing system and meanwhile the stiffness of eccentric Inverted-V bracing system is more than eccentric V bracing system.

Furthermore, considering the deduction which can be derived from the second part of the idealization curvature we can understand that the stiffness of the model structures still increases even after the occurrence of some plastic hinges, then again it can be noticed that the slope of the second part in idealization curvature relevant to the frame which is braced with eccentric Diagonal bracing system is more than that of eccentric Inverted-V bracing system and the slope of eccentric Inverted-V bracing system is more than that of eccentric V bracing system.

According to the surface measurement of the lower part of Force-Displacement diagram that bodes and illustrates the energy absorption and dissipation, it can be inferred that the eccentric Diagonal bracing system has more capacity than Inverted-V bracing system and the also the eccentric Inverted-V bracing system has more capacity than eccentric V-bracing system. Exactly the same conclusion can be obtained about 8-storey frames and 12_storey frames with different eccentric bracing systems.

II) Comparing amongst the performance of the frames with identical bracing system but at different heights (4-story, 8-story, and 12-story), through figures 4.23 to 4.25:

With regard to the fact that by increasing the height of the structure then the degree of indetermination becomes more, and also as a result of this more plastic hinges will occur therefore it can be concluded that the model structure with 12-storey has more capacity than that of 8-storey and finally the model structure of 8-storey has more capacity than 4-storey.

III) Comparison amongst braces with the same height but with different type of eccentric bracing system from the economical point of view:

From the table it can be inferred that amongst 4-storey frames, The V-braced structure is lightest and diagonal braced frame is the heaviest while the inverted V-braced is in between. So the V-braced frame is better from economical point of view. For 8_storey frames there is a slight difference between Inverted-V and Diagonal braced frames that are considered as heavy frames in this case, but the V-braced frame is about 20 (ton) lighter than the others which could be more economical in this case. Finally for the 12_storey there is a slight difference between V-braced and Inverted-V-braced frames in their weight and both of them are much lighter than the Diagonal braced frame which is in this case as the heaviest frame at last for the 12-storey frames the V-braced structure cab be named as the most economical one.

5.1.1 Overall Conclusion

On the basis of the above mentioned reasons and discussions about the different frames in this study it can be concluded that amongst 4 story frames in accordance with performance and also from the economical point of view the Eccentric Diagonal bracing system is better because of its good performance. Although the Eccentric Diagonal braced frame in 4 story is the heaviest but the weight differences can be ignored for low stories (4 stories), as their weights are to some extent in the same range.

In 8 story frames the Eccentric V-braced frame has the lightest frame and the other two eccentric bracing systems (Inverted-V & Diagonal Bracing systems) are relatively heavier and have to some extent the identical weights, although the weight of V-bracing system is low but because of its bad performance it is not selected as the best option here. Therefore in accordance with their performance, the Eccentric Inverted-V is the best one in a total view.

Finally amongst the frames of 12 story models, the one which is framed with Eccentric Diagonal is so heavy that should not be taken as a good option for practical projects that include high-rise structures .And between the other two types (V & Inverted-V) , the Eccentric Inverted-V is better in total and is considered as the best one amongst 12 stories ,while its weight difference in comparison with V- Bracing system is trivial but its better behavior in comparison with V-braced frame makes it the best one among the others.

Part of the aim of this research was to fulfill the further work recommended by Arash Farzam (2009). He did his research on the concentrically braced steel frames while in this research the eccentrically braced steel frames were considered. Despite the fact that the frames used in both research are not geometrically identical and the steel sections used are also not the same, the following are some points that can be noted when the results of the two studies are compared.

I) Comparing the four story models, the slope of the first part of idealized curvature of the concentric bracing system is more than that of for the eccentric bracing systems which bodes more stiffness for this kind of bracing system. On the other hand the slope of the second part of idealized curvature for the eccentric bracing system is more than that of concentric bracing system which indicates more ductility for concentric bracing systems.

II) Comparing the eight story models, it can be seen that for the concentric bracing systems the disparity between the slopes of the two linear parts of idealized curvature is trivial and this to some extend confirms the argument that stiffness increases with the increase in the height. But for the models with eccentrically braced frames the disparity between the slopes of the idealized curvature is tangible which bodes higher ductility for the eight story models with eccentric bracing system in comparison with concentric bracing system.

III) Comparing the twelve story frames it can be inferred that the first and second part of idealized curvature for the concentrically brace frames are to some extent continuum and continuous with approximately the same slope. On the other hand, considering the models with eccentric bracing systems, after the linear behavior and having developed a number of plastic hinges the structure would lose its initial stiffness and enters into a perfect plastic behavior.

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APPENDIX

Appendix A:

Table 5-4 Steel Moment Frame Connection Types

Connection	Description[1, 2]	Type
Welded Unreinforced Flange (WUF)	Full-penetration welds between beam and columns, flanges, bolted or welded web, designed prior to code changes following the Northridge earthquake	FR
Bottom Haunch in WUF w/Slab	Welded bottom haunch added to existing WUF connection with composite slab[3]	FR
Bottom Haunch in WUF w/o Slab	Welded bottom haunch added to existing WUF connection without composite slab[3]	FR
Welded Cover Plate in WUF	Welded cover plates added to existing WUF connection[3]	FR
Improved WUF-Bolted Web	Full-penetration welds between beam and column flanges, bolted web[4]	FR
Improved WUF-Welded Web	Full-penetration welds between beam and column flanges, welded web[4]	FR
Free Flange	Web is coped at ends of beam to separate flanges, welded web tab resists shear and bending moment due to eccentricity due to coped web[4]	FR
Welded Flange Plates	Flange plate with full-penetration weld at column and fillet welded to beam flange[4]	FR
Reduced Beam Section	Connection in which net area of beam flange is reduced to force plastic hinging away from column face[4]	FR
Welded Bottom Haunch	Haunched connection at bottom flange only[4]	FR
Welded Top and Bottom Haunches	Haunched connection at top and bottom flanges[4]	FR
Welded Cover-Plated Flanges	Beam flange and cover-plate are welded to column flange[4]	FR
Top and Bottom Clip Angles	Clip angle bolted or riveted to beam flange and column flange	PR
Double Split Tee	Split tees bolted or riveted to beam flange and column flange	PR
Composite Top and Clip Angle Bottom	Clip angle bolted or riveted to column flange and beam bottom flange with composite slab	PR
Bolted Flange Plates	Flange plate with full-penetration weld at column and bolted to beam flange[4]	PR[5]
Bolted End Plate	Stiffened or unstiffened end plate welded to beam and bolted to column flange	PR[5]
Shear Connection w/ Slab	Simple connection with shear tab, composite slab	PR
Shear Connection w/o Slab	Simple connection with shear tab, no composite slab	PR

1. Where not indicated otherwise, definition applies to connections with bolted or welded web.
2. Where not indicated otherwise, definition applies to connections with or without composite slab.
3. Full-penetration welds between haunch or cover plate to column flange conform to the requirements of the AISC (1997) *Seismic Provisions*.
4. Full-penetration welds conform to the requirements of the AISC (1997) *Seismic Provisions*.
5. For purposes of modeling, the connection may be considered FR if it meets the strength and stiffness requirements of Section 5.5.2.1.

Appendix B:

Table A.2.a: Part one of Table 5-6 of FEMA 356 (Courtesy of Federal Emergency Management Agency).

Table 5-6 Modeling Parameters and Acceptance Criteria for Nonlinear Procedures—Structural Steel Components

	Modeling Parameters			Acceptance Criteria				
	Plastic Rotation Angle, Radians		Residual Strength Ratio	Plastic Rotation Angle, Radians				
					Primary		Secondary	
Component/Action	**a**	**b**	**c**	**IO**	**LS**	**CP**	**LS**	**CP**
Beams—flexure								
a. $\frac{b_f}{2t_f} \leq \frac{52}{\sqrt{F_{ye}}}$ and $\frac{h}{t_w} \leq \frac{418}{\sqrt{F_{ye}}}$	$9\theta_y$	$11\theta_y$	0.6	$1\theta_y$	$6\theta_y$	$8\theta_y$	$9\theta_y$	$11\theta_y$
b. $\frac{b_f}{2t_f} \geq \frac{65}{\sqrt{F_{ye}}}$ or $\frac{h}{t_w} \geq \frac{640}{\sqrt{F_{ye}}}$	$4\theta_y$	$6\theta_y$	0.2	$0.25\theta_y$	$2\theta_y$	$3\theta_y$	$3\theta_y$	$4\theta_y$
c. Other	Linear interpolation between the values on lines a and b for both flange slenderness (first term) and web slenderness (second term) shall be performed, and the lowest resulting value shall be used							
Columns—flexure [2, 7]								

Appendix C:

Table A.2.b: Part two of Table 5-6 of FEMA 356 (Courtesy of Federal Emergency Management Agency).

Table 5-6 Modeling Parameters and Acceptance Criteria for Nonlinear Procedures—Structural Steel Components (continued)

	Modeling Parameters			Acceptance Criteria				
	Plastic Rotation Angle, Radians		Residual Strength Ratio	Plastic Rotation Angle, Radians				
					Primary		Secondary	
Component/Action	**a**	**b**	**c**	**IO**	**LS**	**CP**	**LS**	**CP**
For $0.2 < P/P_{CL} < 0.50$								
a. $\frac{b_f}{2t_f} \le \frac{52}{\sqrt{F_{ye}}}$ and $\frac{h}{t_w} \le \frac{260}{\sqrt{F_{ye}}}$	—[3]	—[4]	0.2	$0.25\theta_y$	—[5]	—[3]	—[6]	—[4]
b. $\frac{b_f}{2t_f} \ge \frac{65}{\sqrt{F_{ye}}}$ or $\frac{h}{t_w} \ge \frac{400}{\sqrt{F_{ye}}}$	$1\theta_y$	$1.5\theta_y$	0.2	$0.25\theta_y$	$0.5\theta_y$	$0.8\theta_y$	$1.2\theta_y$	$1.2\theta_y$
c. Other	Linear interpolation between the values on lines a and b for both flange slenderness (first term) and web slenderness (second term) shall be performed, and the lowest resulting value shall be used							
Column Panel Zones	$12\theta_y$	$12\theta_y$	1.0	$1\theta_y$	$8\theta_y$	$11\theta_y$	$12\theta_y$	$12\theta_y$
Fully Restrained Moment Connections[13]								
WUF[12]	0.051-0.0013*d*	0.043-0.0006*d*	0.2	0.0128-0.0003*d*	0.0337-0.0009*d*	0.0284-0.0004*d*	0.0323-0.0005*d*	0.043-0.0006*d*
Bottom haunch in WUF with slab	0.026	0.036	0.2	0.0065	0.0172	0.0238	0.0270	0.036
Bottom haunch in WUF without slab	0.018	0.023	0.2	0.0045	0.0119	0.0152	0.0180	0.023
Welded cover plate in WUF[12]	0.056-0.0011*d*	0.056-0.0011*d*	0.2	0.0140-0.0003*d*	0.0319-0.0006*d*	0.0426-0.0008*d*	0.0420-0.0008*d*	0.056-0.0011*d*
Improved WUF-bolted web[12]	0.021-0.0003*d*	0.050-0.0006*d*	0.2	0.0050-0.0001*d*	0.0130-0.0002*d*	0.0210-0.0003*d*	0.0375-0.0005*d*	0.050-0.0006*d*
Improved WUF-welded web	0.041	0.054	0.2	0.0103	0.0312	0.0410	0.0410	0.054
Free flange[12]	0.067-0.0012*d*	0.094-0.0016*d*	0.2	0.0168-0.0003*d*	0.0509-0.0009*d*	0.0670-0.0012*d*	0.0705-0.0012*d*	0.094-0.0016*d*
Reduced beam section[12]	0.050 0.0003*d*	0.070 0.0003*d*	0.2	0.0125 0.0001*d*	0.0380 0.0002*d*	0.0500 0.0003*d*	0.0525 0.0002*d*	0.07 0.0003*d*
Welded flange plates								
a. Flange plate net section	0.03	0.06	0.2	0.0075	0.0228	0.0300	0.0450	0.06
b. Other limit states	force-controlled							
Welded bottom haunch	0.027	0.047	0.2	0.0068	0.0205	0.0270	0.0353	0.047
Welded top and bottom haunches	0.028	0.048	0.2	0.0070	0.0213	0.0280	0.0360	0.048
Welded cover-plated flanges	0.031	0.031	0.2	0.0078	0.0177	0.0236	0.0233	0.031

Appendix D:

Table A.2.c: Part three of Table 5-6 of FEMA 356 (Courtesy of Federal Emergency Management Agency).

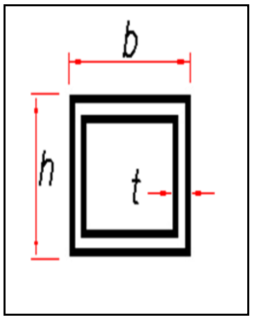
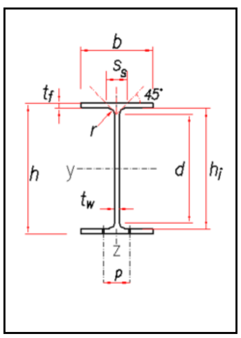
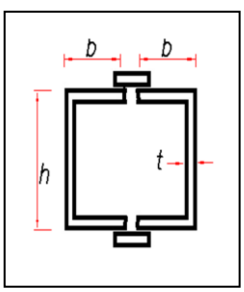
Table 5-6 ***Modeling Parameters and Acceptance Criteria for Nonlinear Procedures—Structural Steel Components (continued)***

	Modeling Parameters			Acceptance Criteria				
	Plastic Rotation Angle, Radians		Residual Strength Ratio	Plastic Rotation Angle, Radians				
					Primary		Secondary	
Component/Action	**a**	**b**	**c**	**IO**	**LS**	**CP**	**LS**	**CP**
Partially Restrained Moment Connections								
Top and bottom clip angle[9]								
a. Shear failure of rivet or bolt (Limit State 1)[8]	0.036	0.048	0.200	0.008	0.020	0.030	0.030	0.040
b. Tension failure of horizontal leg of angle (Limit State 2)	0.012	0.018	0.800	0.003	0.008	0.010	0.010	0.015
c. Tension failure of rivet or bolt (Limit State 3)[8]	0.016	0.025	1.000	0.005	0.008	0.013	0.020	0.020
d. Flexural failure of angle (Limit State 4)	0.042	0.084	0.200	0.010	0.025	0.035	0.035	0.070
Double split tee[9]								
a. Shear failure of rivet or bolt (Limit State 1)[8]	0.036	0.048	0.200	0.008	0.020	0.030	0.030	0.040
b. Tension failure of rivet or bolt (Limit State 2)[8]	0.016	0.024	0.800	0.005	0.008	0.013	0.020	0.020
c. Tension failure of split tee stem (Limit State 3)	0.012	0.018	0.800	0.003	0.008	0.010	0.010	0.015
d. Flexural failure of split tee (Limit State 4)	0.042	0.084	0.200	0.010	0.025	0.035	0.035	0.070
Bolted flange plate[9]								
a. Failure in net section of flange plate or shear failure of bolts or rivets[8]	0.030	0.030	0.800	0.008	0.020	0.025	0.020	0.025
b. Weld failure or tension failure on gross section of plate	0.012	0.018	0.800	0.003	0.008	0.010	0.010	0.015
Bolted end plate								
a. Yield of end plate	0.042	0.042	0.800	0.010	0.028	0.035	0.035	0.035
b. Yield of bolts	0.018	0.024	0.800	0.008	0.010	0.015	0.020	0.020
c. Failure of weld	0.012	0.018	0.800	0.003	0.008	0.010	0.015	0.015
Composite top clip angle bottom[9]								
a. Failure of deck reinforcement	0.018	0.035	0.800	0.005	0.010	0.015	0.020	0.030
b. Local flange yielding and web crippling of column	0.036	0.042	0.400	0.008	0.020	0.030	0.025	0.035

Appendix E:

European standard beams IPE					
Designation	Unit Weight	Depth of Section	Width of Section	Thickness of	
				Web	Flange
	kg/m	mm	mm	mm	mm
IPE 80	6.00	80	46	3.80	5.20
IPE 100	8.10	100	55	4.10	5.70
IPE 120	10.40	120	64	4.40	6.30
IPE 140	12.90	140	73	4.70	6.90
IPE 160	15.80	160	82	5.00	7.40
IPE 180	18.80	180	91	5.30	8.00
IPE 200	22.40	200	100	5.60	8.50
IPE 220	26.20	220	110	5.90	9.20
IPE 240	30.70	240	120	6.20	9.80
IPE 270	36.10	270	135	6.60	10.20
IPE 300	42.20	300	150	7.10	10.70
IPE 330	49.10	330	160	7.50	11.50
IPE 360	57.10	360	170	8.00	12.70
IPE 400	66.30	400	180	8.60	13.50
IPE 450	77.60	450	190	9.40	14.60
IPE 500	90.70	500	200	10.20	16.00
IPE 550	106.00	550	210	11.10	17.20
IPE 600	122.00	600	220	12.00	19.00
IPE 750 x 137	137.00	753	263	11.50	17.00
IPE 750 x 147	147.00	753	265	13.20	17.00
IPE 750 x 173	173.00	762	267	14.40	21.60
IPE 750 x 196	196.00	770	268	15.60	25.40
Engineer Diary (www.strleng.blogspot.com)					

Appendix E:

Column (Box section) 	4 story	Box 200x200x10 / Box 200x200x20 / Box 300x300x10 / Box 300x300x20 / Box 300x300x30
	8 story	Box 200x200x10 / Box 200x200x20 / Box 300x300x20 / Box 300x300x30 / Box 400x400x20 / Box 400x400x30 / Box 600x600x10/ Box 600x600x30 / Box 700x700x30/ Box 800x800x40 Box 900x900x30 / Box 900x900x60 / Box 1000x1000x40
	12 story	Box 300x300x10 / Box 300x300x20 / Box 400x400x20 / Box 500x500x20 / Box 500x500x30 / Box 600x600x20/ Box 700x700x10 / Box 700x700x20 / Box 700x700x30/ Box 900x900x50 / Box 1100x1100x60 / Box 1100x1100x100
Beam (IPE section) 	4 story	IPE 270 / IPE 300 / IPE 330 / IPE 400 /2 IPE 220 /2 IPE 240
	8 story	2 IPE 200 /2 IPE 220 /2 IPE 240 /2 IPE 270 /2 IPE 300
	12 story	2 IPE 200 /2 IPE 240 /2 IPE 270 /2 IPE 300
Brace (Channel section) 	4 story	2 CH 120x60x12 / 2 CH 140x70x14
	8 story	2 CH 120x60x12 / 2 CH 140x70x14 /2 CH 160x80x16
	12 story	2 CH 140x70x14 / 2 CH 150x75x15 / 2 CH 200x100x15