Behavior of Steel Braced Frame Structures by using Pushover and Response Spectrum Analysis

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ABSTRACT

Bracing systems are one of the efficient methods used for buildings to resist lateral loads. Steel structures need to be strong and at the same time have adequate ductility against various loading conditions. The objective of this study was to investigate the behavior of the steel concentric and eccentric braced frames by using pushover and response spectrum analysis. Diagonal-shape, inverted chevron (Λ-shape) are the types concentric braced systems and diagonal and inverted chevron (Λ-shape) are the types of and eccentric bracing systems considered for the study, respectively. 4- and 12-story high buildings, H and square plan shape with 5x5 symmetric number of bays were used to design with relevant Eurocodes and carry out performance analysis. Pushover analysis results show that the collapsed plastic hinges mainly occurred in the buildings with eccentric diagonal bracing systems with low target displacements. Response spectrum analysis results show that buildings with diagonal concentric bracing systems achieved the lowest story displacement and furthermore it was the only braced system that met the displacement criteria. Comparing the results of story drift majority of the investigated cases with diagonal braced frame achieved the lowest drift value, except for 4-stroy H plan. The economical comparison between the selected braced frames has been done by comparing the weight of structure for all analyzed conditions. It was found that eccentric inverted chevron (Λ-shape) achieved the lowest structural weight. Comparing only the base shear for the pushover and response spectrum analysis it was found that the former had higher base shear than the latter in both x- and y-directions for all conditions, except for 4-story square plan, when response spectrum analysis achieved larger base shear than the pushover analysis.

Keywords: Eccentric Brace Frame, Concentric Brace Frame, Linear dynamic analysis, Nonlinear Static Analysis, Response Spectrum Analysis, Pushover Analysis

ÖZ

Çelik yapıların çeşitli yükleme koşullarına karşı güçlü ve sünümlü olması gerekir. Destek sistemleri, binaların yanal yüklere karşı direnmesini sağlayan etkili yöntemlerden biridir. Bu çalışmanın amacı, çelik diyagonal ve ters örgülü (Λ) şekilli eşmerkezli ve eksantrik parantez çerçevelerinin, itme ve tepki spektrumu analizini kullanarak davranışlarını araştırmaktır. H ve kare plan şeklinde, 5x5 simetrik sayıda koyları olan, 4 ve 12 kat yüksekliğindeki binalar Avrupa standardları kullanılarak tasarım ve performans analizleri gerçekleştirilmiştir. İtme analizi sonuçları, çökmüş plastik mafsalların ağırlıklı olarak, düşük hedef deplasmanlı eksantrik diyagonal destek sistemli binalarda meydana geldiğini göstermektedir. Tepki spektrumu analiz sonuçları, diyagonal konsantrik destek sistemlerine sahip binaların en düşük kat deplasmanlarını sağladığını ve yer değiştirme kriterlerini karşıladığını göstermektedir. İncelenen örneklerin kat ötelenme sonuçlarının çoğunlukla karşılaştırılması, 4-stroy H planı haricinde, en düşük kayma değerine ulaşmıştır. Analiz edilen yapıların çelik ağırlıkları karşılaştırıldığında, eksantrik tersine döneme ait şivron (Λ-şekli) en düşük yapısal ağırlığa sahipti. Kat ötelenme sonuçları karşılaştırıldığında, incelenen çapraz çerçeveli sistemlerin çoğunluğunun, 4 katlı H planı hariç, en düşük öteleme değerine ulaştığını görürüz. Analiz edilen yapıların çelik ağırlıkları karşılaştırıldığında, eksantrik ters V (Λ-şekli) desteği en düşük yapısal ağırlığa sahipti. İvme ve tepki spektrumu analizi için sadece temel kesme kuvveti ile karşılaştırıldığında, tepkime spektrumu analizi yapıldığında, hariç olmak üzere, tüm koşullar için x ve y yönlerinin her ikisinde de daha yüksek taban kayması olduğu bulunmuştur. Tepki spektrumu analizisonucu oluşan taban kesme kuvveti itme analizinden dolayı oluşan taban kuvvetinden büyüktür.

Anahtar Kelimeler: Dışmerkezli Karkas Çerçeve, Konsantrik Kütük Çerçeve, Doğrusal dinamik analiz, Doğrusal Olmayan Statik Analiz, Tepki Spektrum Analizi, İtme Analizi

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DEDICATION

To my Country

To My Family

To My Friends

To them, wherever they are, I dedicate this piece of work.

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Chapter 1

INTRODUCTION

Every year, many people lose their lives because of the earthquakes in different countries. This matter urges the engineers to find a ductile system for lateral stability that has been one of the main problems of steel framed structures in regions with high earthquake hazard. This issue has been studied, and the experts come up with concentric (such as X, Diagonal and chevron), eccentric and knee bracing lateral load resisting system for steel framed structure.

The performance especially due to inelastic behaviour is considered as one of the main factors that can affects the choice of bracing systems for a specific steel framed structure. The bracing system can achieve adequate plastic deformation before collapse as well as it can absorb more energy during the earthquake.

There are different types of bracing systems, and each one has different construction cost and performance, which should be considered by practicing engineers when designing structures.

1.1 Background

During the last few decades, response spectrum and pushover analysis of bracing systems has been studied and consequently parameters, such as, lateral displacement, amplification factor (Cd), over strength factor (W), and seismic behavior factor (R) were introduced to loading codes of practice like UBC (Uniform

Building Code) and IBC (International Building Code). These codes are widely used for design around the world in order to achieve adequate the inelastic behavior of the bracing systems.

In order to calculate the earthquake load on a structure, system ductility that can affect the impact of linear and nonlinear behaviour and performance of the steel bracing members should be obtained by illustrating seismic behaviour factor. In addition, changes in the magnitude of lateral load may have effects on the efficiency of the steel bracing members due to wind and particularly earthquake loads. The applied loads due to the earthquake on the structure can be obtained by using the following equation:

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

Where: A: site seismicity,

B: ground soil type

I: factor of the importance of structure.

1.2 Objectives of the Study

This study aims to do a comparison study for the ductility levels of different steel bracing systems (eccentric and concentric braced frame) by using pushover and response spectrum analysis, and to comparison process of the results from the economical point of view by comparing the weight of the selected frames for all structural conditions. By studying both weight and performance of the bracing systems simultaneously, the project states a realistic comparison between the selected braced frames.

1.3 Reasons of this Study

Steel framed structures was designed and constructed require bracing system. Performance and economical side are the two parameters that effecting the type of structural systems to be used, especially for steel structure that use bracing systems. By comparing these two parameters, this research can form the basis for new methods of evaluation for bracing systems. On the other hand, accurate information about behaviour of response spectrum and pushover analysis of different structural systems leads to higher quality in their design.

1.4 Guide to the Thesis

This study contains six chapters. Chapter two will be on the literature review, being divided into three sections. The first section will be about types of lateral and its effects on steel structures. The second section includes the types of lateral loads resisting systems. While the third section will be about the will describe the types of used analysis methods. Chapter three will talks about the methodology that have been used for this sturdy. While chapter four will shows the used sections for all conditions of structures as well as the results of the analysis. Chapter five will be devoted to talks about the description and discussion of the results. Finally, Chapter six will be the conclusion, over all conclusion and a recommendation of the future studies.

Chapter 2

LITERATURE REVIEW

2.1 Types of Lateral Loads and its Effect on Steel Structure

2.1.1 Background Information about Earthquake

Earthquake is wave motion generated by forces in constant turmoil beneath earth's surface moving through the earth's crust. Earthquake is considered as the most naturally uncertain load that applied on the buildings, which cause a ground shake. Earthquake caused by the plate tectonics moves and it happens under the earth crust. Earthquake occurs when stress in the earth at a given place is larger than the rock's strength and it sometimes caused by underground explosion. According to Landau, L.D.; Lifshitz, E. M. (1986), there are kinds of ground motion of earthquake depends on the way that earthquake move and where it acts. These kinds divided into two groups, which are:

1. Body wave: It can move through the inner layers of crust, and there are two kinds of this wave:

    - P-waves (Primary wave): It is the fastest kind of seismic wave, and it can move through the solid rocks and fluids. It pushes and pulls the rocks and move through it like a sound wave.

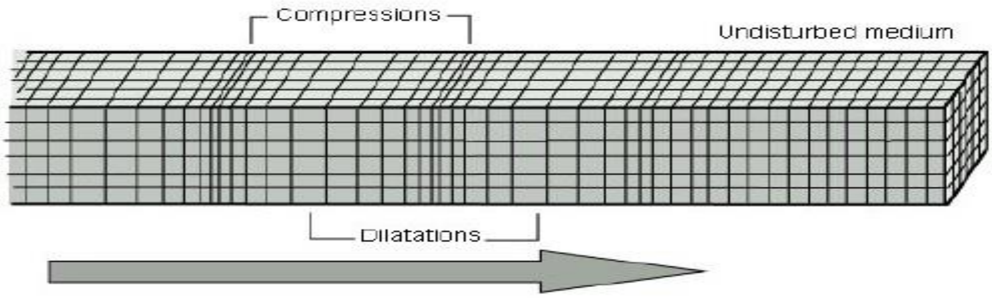

Figure 1: P-Wave Motion Direction Olivadoti, G. (2001).

- S-wave (Secondary wave): It is the second wave that we can feel during earthquake, and it is slower that P-wave. It moves up and down or side-to-side through the solid rock only.

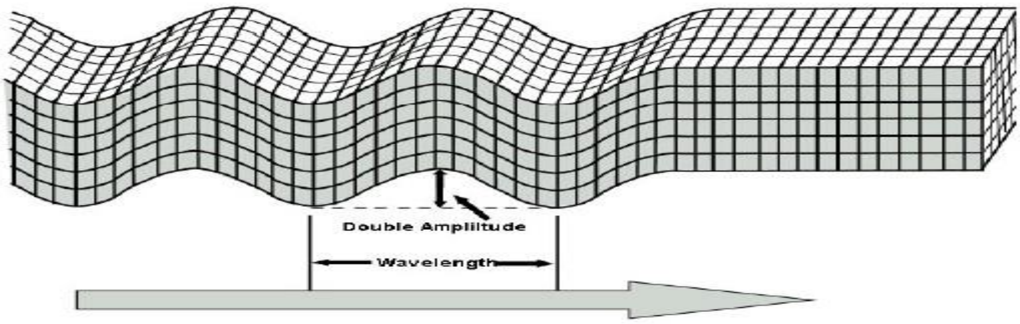

Figure 2: S-Wave Motion Direction Olivadoti, G. (2001).

2. Surface waves: It can move along the earth surface such as ripples on water. There are two kinds of surface waves:

- Love wave: It is also known as Q-wave (Quer wave) named according to Edward Hough Love, and it is a wave occurs due to interfaces of some S-waves through the elastic layer of the earth surface. It moves in horizontal line vertical on direction if propagation causes shifting for the surface layer during the earthquake. Love wave moves with low speed less than other waves except Rayleigh wave. The strength or the amplitude of Love wave can be found from the equation: $1/\sqrt{r}$, Where r is the distance that Love wave moves during the earthquake.

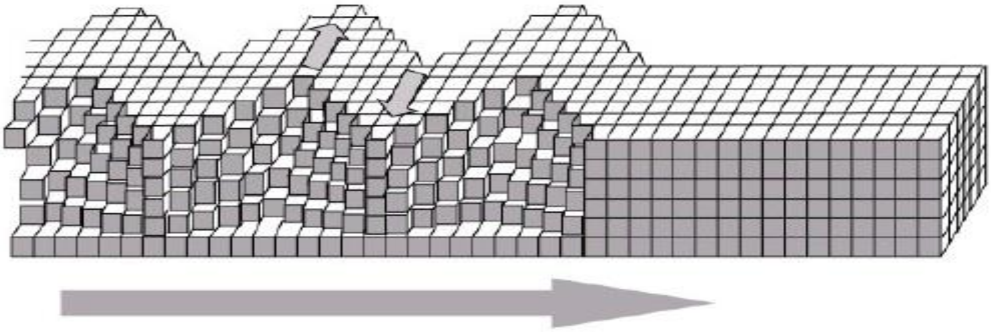

Figure 3: Love Wave Motion Direction Olivadoti, G. (2001).

- Rayleigh wave: Founded by Lord Rayleigh in 1885and it moves along near solid surfaces of the crust, and it includes a longitudinal and transverse motion which reduced the amplitude when the distance from the surface increasing. Produced in subsequences in some ways such as localized impact.

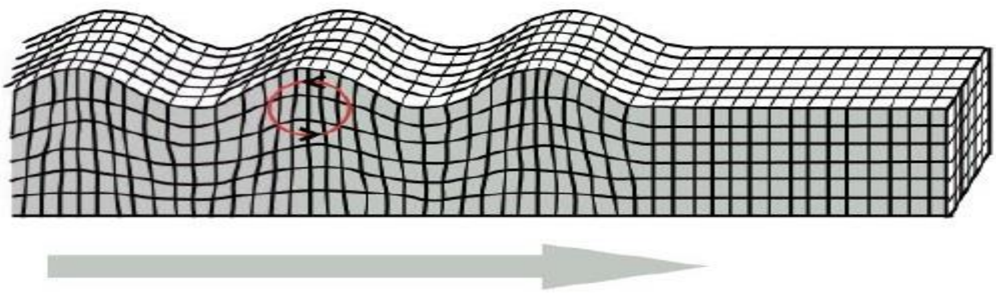

Figure 4: Rayleigh Wave Motion Direction Olivadoti, G. (2001).

Focus point is the first point that earthquake waves reach, and it is an underground point of origin of earthquake where the rocks break and move. The unexpected accurate of earthquake makes the applied loads the most dangerous loads, and that make it differs from the other loads, because the severity degree depends on important parameters such as frequency, continuity, intensity, ground acceleration and magnitude of the applied earthquake. The intensity is the visible effects experienced at specific location usually measured by Mercalli scale and it describes the effects of earthquake on steel structures, while magnitude (usually measured by Richter scale) is the measure of amount of the energy released. The ground acceleration or the ground displacement are recorded is the most straightforward data, its recorded as function time and used usually in time-history analysis. Ground acceleration depends on two sub-products:

- The maximum value of peak ground acceleration or acceleration at the bedrock level, this parameter is used to define earthquake in a given area. Earthquake zones presented as peak round acceleration (Figure.5) and its range usually from 0.05 g in the low earthquake zones and 0.5 g for the high earthquake zones.
- The standard representation of an earthquake is the acceleration response spectrum and it is considered in buildings design.

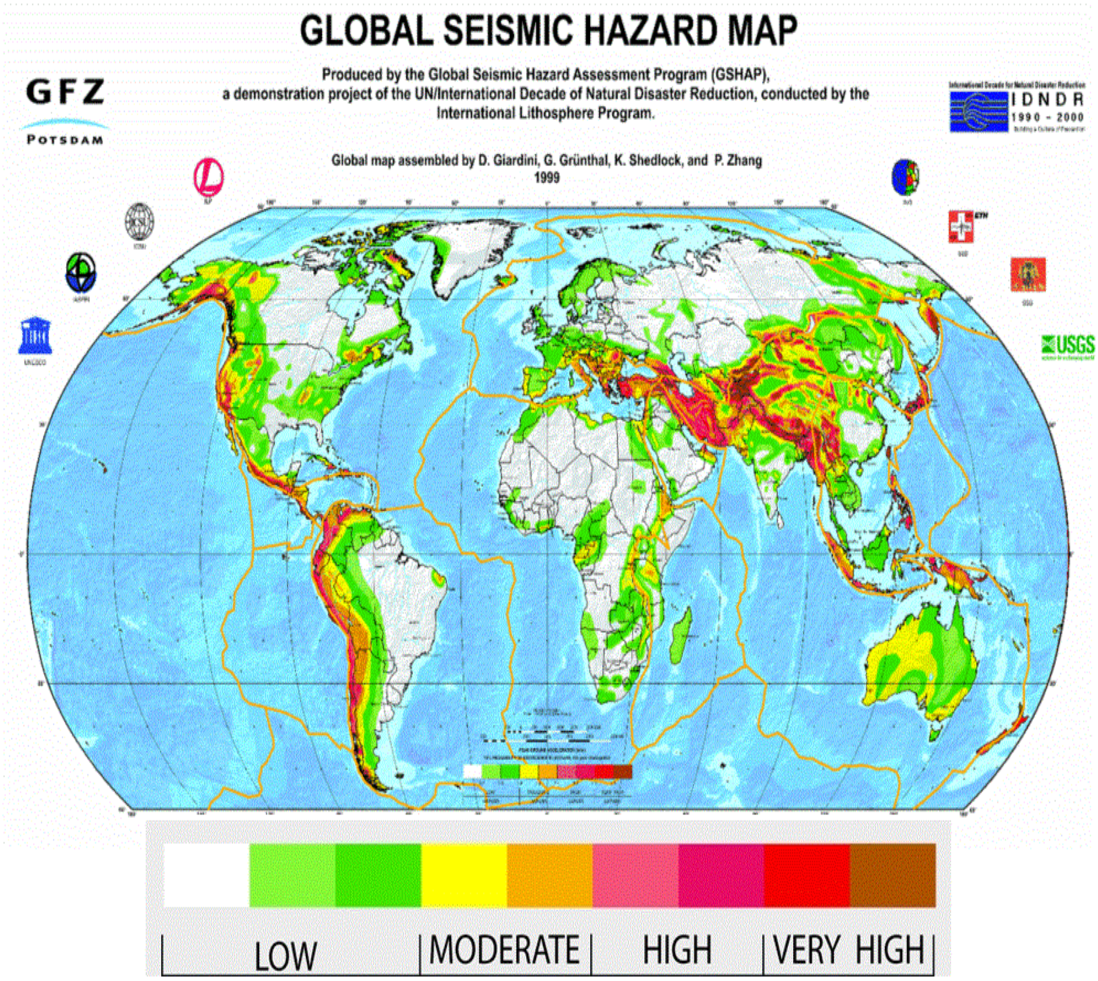

Figure 5 World Peak Ground Acceleration Map

There are other problems of inaccuracy in structural response like type of used material, soil properties, location of the building, in which earthquake zone this building is located, center of earthquake and the depth of the earthquake. Lateral loads caused by earthquake differs than other loads and make design of structure more difficult, because usually structures are designed according to withstand gravity loads that are acting vertically with factor of safety, so lateral loads increasing due to ground motion and it can cause severe damage. The cyclic and reversal of stresses of earthquake motion may makes the axially loaded members resist tension and compression and makes the beams resist positive and negative moments. Also the dynamic loading and degree of response of the earthquake requires consideration of elastic forces and moment of inertia Agrawal, P., & Shrikhande, M. (2006).

2.1.2 Background about Wind Action

The wind action is represented by a simplified set of pressures or forces whose effects are equivalent to the extreme effects of the turbulent wind. Wind actions are usually subjected to change according to the time and act it directly on the external surface of structure as pressures, and also it acts on the internal surfaces but in indirectly way, and in direct way in open structure. When these pressures are applied on the surface and result a lateral forces acting on the surface of the structure or of individual cladding. The characteristics of the pressures created by wind load can be effected by the approaching of wind and the shape of the structure. The effect of the wind upon the structure depends on size, shape and dynamic properties of the structure. The importance of wind turbulence is that superimposes peaks and troughs on the mean wind speed, and consequently increases the peak pressures to be designed against. Usually, wind actions are accelerated and deflected when high ground is encountered. The area of the structure effect the wind load and the design consideration that should be taken for designing structure against wind, which is known as orography factor. Sometimes wind actions lead to damage to buildings when strong winds occurs, Holmes, J. D., Kwok, K. C. S., Ginger, J. D., & Walker, G. R. (2012).

Wind load is considered as a lateral load that can acts on the buildings. Wind loads can be destructive because winds can generate compression acts on the structure. Winds effective load depends on the size and shape of the building, and the height of each floor because each floor will have different value of applied wind load, Khanduri, A. C., Stathopoulos, T., & Bédard, C. (1998).

2.1.3 Behaviour of Steel Structure during Seismic Action

Steel is considered as a ductile material, and it features strong compression and tension capacity, produced with high quality control, as well as being a good material for building structures to resist the lateral loads. Higher elastic limits of steel structure can be provided by high strength steels, but have less ductility because of increasing in some of the chemical component. High strength steel require less cross sectional area than mild steels and therefore it becomes more prone to instability effects, Duggal, S. K. (2013).

Plastic hinges are important for capacity design and accuracy of the actual yield stress. Because it will lead to formation of plastic hinges, when the actual strength of members is more than design strength. To avoid the formation of plastic hinges in this case, ratio of expected yield strength factor should be specified for minimum yield strength of steel members. In addition, the ratio of expected yield strength factor is used to ensure that connections or members of steel frames should resist the plastic hinges in other members that have enough strength. Plastic hinges are normally expected to be formed in beams and columns at the critical sections, so these beams and columns must be plastic cross sections, which is not very efficient. Therefore, compact and semi-compact sections can be used to achieving enough ductility or rotation capacity, ultimate moment capacity and hysteretic energy dissipation capacity. Local buckling factor is considered as the most reliable way to control the evaluation these amount in nature by using one of methods for evaluating these amounts. Important buildings such as hospitals, fire stations and government sections, should be designed for a high level of earthquake resistance, at the same time it must remain usable immediately after the earthquake, and the structures must sustain very little damage.

Chitte, C. J., & Sonawane, N. Y. (2016) state that the design philosophy of steel structure under an earthquake can be described as follow (See Figure 6):

- Under minor, frequent shaking: the structural members of the building should not be damaged, while the damage in other members that do not carry loads can have repairable damage. Therefore, after this shaking, the repair costs will be small, and the building will be fully operational within a short time.

- Under moderate, occasional shaking: the main members can have repairable damage, but the other parts of the building may be damaged and should be replaced with new steel members. The building will be operational once the repair and strengthening of the damaged members is completed.

- Under strong, rare shaking: the building will not collapse, but the main members will have irreparable damage. After strong earthquake shaking, the building will not be usable anymore.

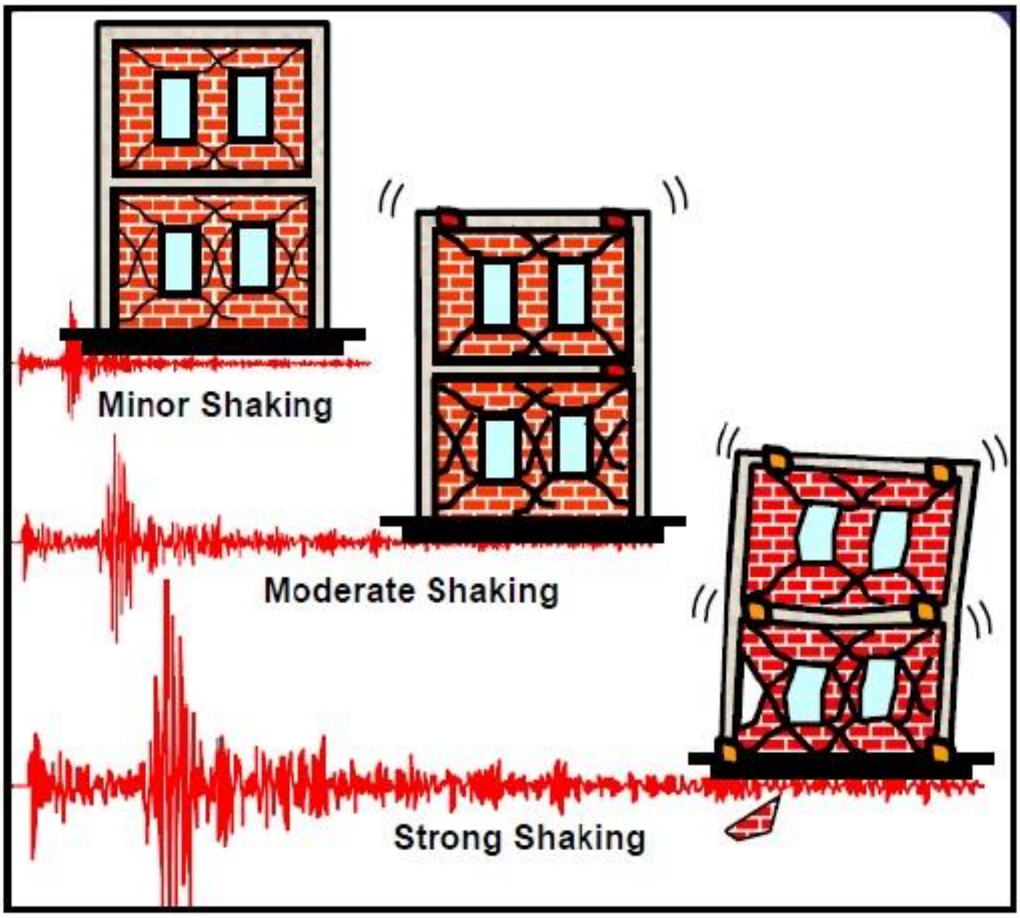

Figure 6: Diagram of Earthquake Resistant design Philosophy Duggal, S. K. (2013).

2.1.3.1 Seismic Behaviour of I-sections

In I sections of steel structure failure steps start with cracks in the web-flange junction in welded sections, and welded elements in built up sections, and fails in the end by local buckling in the flanges. On the other hand, I sections can be considered as a good sections with good ductility and energy dissipation capacities. Many researchers have done tests for I sections such as Ballio and Castiglioni (1994), Krawinkler and Zhorei (1984), and they come up with the hysteretic curve (Figure 7) for I sections for constant amplitude cycling.

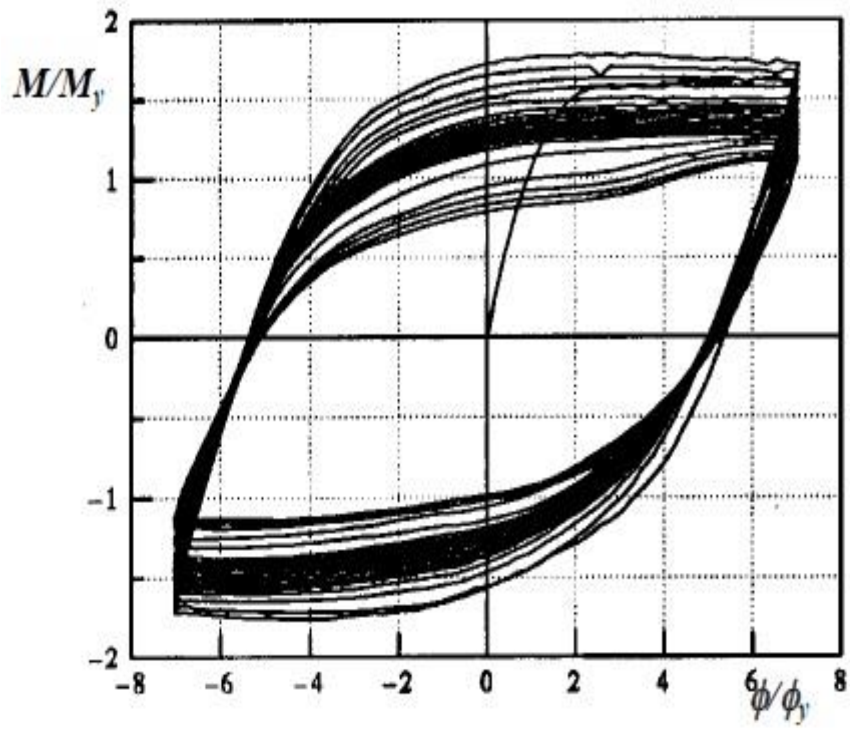

Figure 7: Hysteretic behaviour of an I-section Duggal, S. K. (2013).

Subsequent pinching of the hysteretic curve after the cracks start, the degradation due to local buckling and the gradual stabilization are the ranges of response can be observed. When I sections become more compact, the middle range will be smaller and the tendency of cracking will increase. Therefore, required rotations will not be provided by highly compact sections and the rigid connections. The damage in I sections can be modeled by using a specific approach which is the low cycle fatigue.

2.1.3.2 Seismic Behaviour of Rectangular Hollow Sections (RHS)

Rectangular hollow sections, either hot-rolled or fabricated by welding four plates are used in buildings and bridge piers. The sections rectangular hollow sections can made by welding four plates or hot rolled, and it is used in buildings that have small width to thickness ratio of component plates Agrawal, P., & Shrikhande, M. (2006). The ultimate strengths for rectangular hollow section are high as well as the post-local buckling performance. Some tests are done for RHS by Ballio and Calado

(1994) and Kumar and Usami (1996), and they illustrated the hysteretic curve for RHS under incremental amplitude cycling (Figure 8).

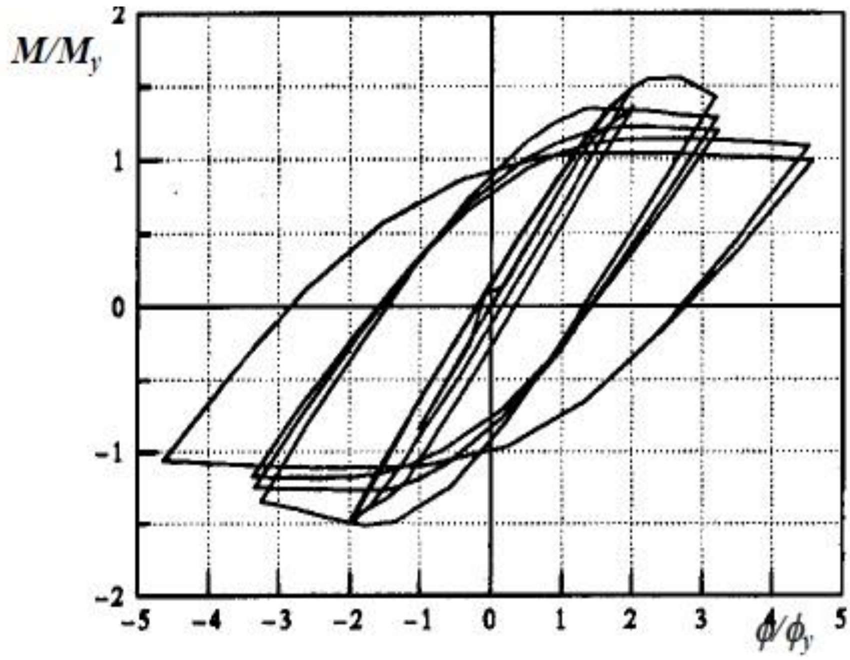

Figure 8: Hysteretic behaviour of rectangular hollow section Duggal, S. K. (2013).

The hysteretic loop is similar to the hysteretic loop in I section curve in figure 7. The degradation in strength with cycling is considerable in the case of high width to thickness ratios under increasing in the amplitudes. For this, the calculations for damage accumulation are needed to consider the deformation damage and the low-cycle fatigue damage.

2.2 Types of Lateral Load Resisting Systems in Steel Structure

Many lateral resisting systems such as braced frames, moment resisting frames and steel plate shear walls are used in by steel structures to resist earthquake motion as much as possible and prevent the brittle collapse. For every lateral load resisting system, there are some factors and specifications that should be existed in the structure.

2.2.1 Steel Bracing System

Bracing systems can be defined as members that can resist lateral loads through axial forces in the components. So bracing members usually carry axial loads due to seismic actions, which can be compression or tension. It is used to save the steel structure from seismic actions. Braced frames perform like vertical trusses where beams and bracing system represent the web members and the columns represent the chords. Bracing system can be exist in more than one form, such as, steel with masonry encasement or concrete, steel bare or steel with nonstructural coating for fire. There are three types of bracing system, which are buckling-restrained braced frame, Concentric Braced Frame (CBF) and Eccentric Braced Frame (EBF). CBF and EBF will be further described in the following section Hong.J, (2005).

2.2.1.1 Concentric Braced Frame

Concentric braced frame system is a steel structure with diagonal members that can resist lateral loads by transferring the lateral loads into vertical loads acting on the column. Bracing system is known to have high elastic stiffness and an efficient system that can resist earthquake or wind loads. The meaning of concentric braced frame is where the components are intersecting at a single point. They either intersect in the main joints or at the center of beam/column, thus decrease the residual moments in the structure. This system can reach high stiffness by using internal axial loads, which is lower than the flexural actions. When CBFs system subjected to less seismic response, it may tend to have high acceleration due to seismic load and low drift capacity Farzam, A. (2009). The ductility of concentric braced frame is limited but it can provide stiffness and strength at low cost. Concentric braced frame divided into two types: Ordinary Concentric Braced Frame (OCBF) and Special Concentric Braced Frame (SCBF), which is a special class of CBF used to maximize the inelastic drift capacity, and it is used for steel structure and composite structure. In general CBF members are connected with members by gusset plate which can be welded or bolted. Designing approach of CBF concentrate on energy dissipation in bracing system that identify with the design, and on the connections to be sure that it will stay in elastic stage during load administration. Connections of CBF's members should be designed to be stronger than the members it-self to reach maximization of energy dissipation and makes bracing members yield and buckle Sabelli, R., Roeder, C. W., & Hajjar, J. F. (2013). Also in designing stage, CBF design should be focused and checked especially for tall buildings on strength drift control which is low compared to the strength. In building less than 14 stories, drift constraints are not the main parameter for any kind of CBF. There are different shapes of CBF such as:

1. X Bracing: X-bracing (Figure 9) considered as the most common type of bracing system. X-bracing members can be categorized as tension and compression when lateral force applied similar to truss members. X-bracing members can develop ductility when its size to yield before the beams and columns. The connections of X-bracing CBF are placed at the joint of beam to column that is gusset plate. According to Eurocode 8, it is designed by assuming that the compression bracing members do not contribute strength or stiffness. The slenderness of diagonal braces in X-braced systems has upper and lower limits, and usually the lower limit around 110 to prevent overload the column, while the upper is around 180 depends the on yield strength to prevent the strength and stiffness degradation.

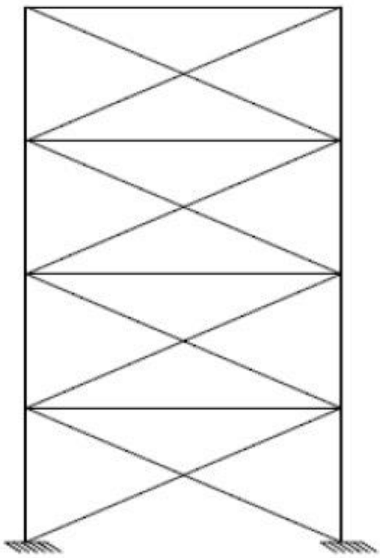

Figure 9: Cross Steel Bracing System

2. Diagonal: Direction of loading of diagonal brace can decide the braces response. In addition, the diagonal bracing systems are located in two at the corners of one bay. There are two types of diagonal bracing:

- Parallel diagonal bracing as shown in (Figure 10) that causes compression in the bracing members, therefore it considered as flexible bracing in the same direction of the applied lateral load.

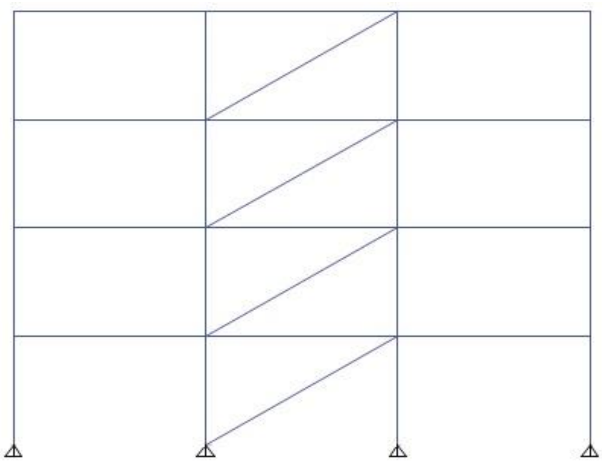

Figure 10: Parallel Concentric Diagonal Steel Bracing

- Sequential diagonal bracing as shown in (Figure 11), is compression bracing and it is more flexible to be used in elevation since it can effectively resist lateral loads in respect of which direction they apply.

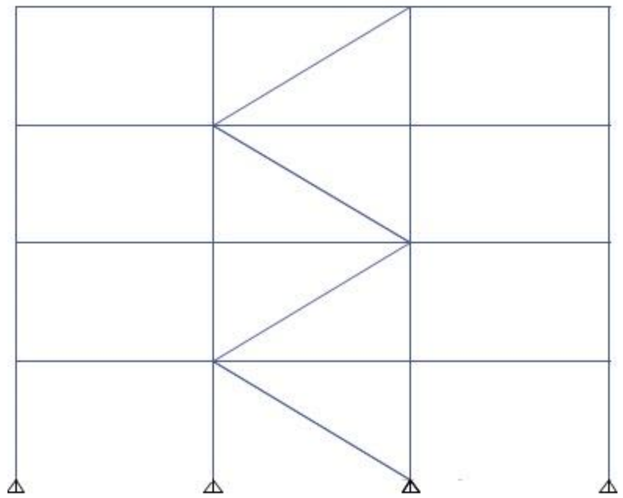

Figure 11: Sequential Concentric Diagonal Steel Bracing

3. V Bracing: The V-bracing (Figure 12) and inverted V-bracing (Figure 13) both suffer from the buckling capacity of the compression members, which may be less than the tension yield capacity of the tension members. Therefore, when the brace members reach their capacity, there should be an out-of balance load on the beams.

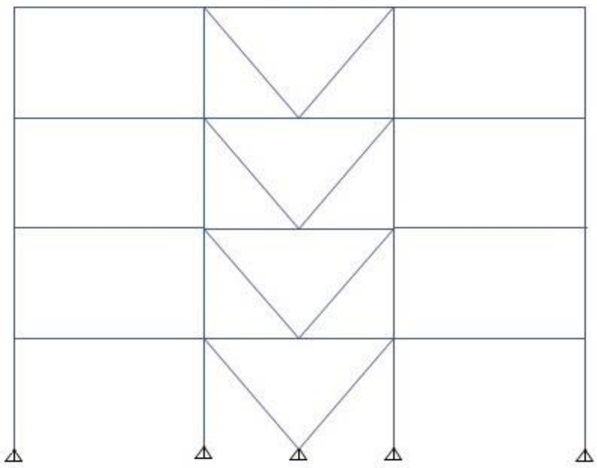

Figure 12: V-Chevron Steel Bracing

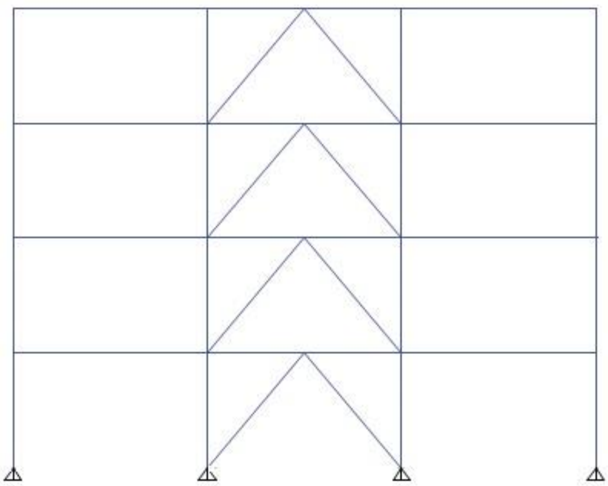

Figure 13: Inverted V-Chevron Steel Bracing

2.2.1.2 Eccentric Braced Frame

Eccentric braced frame (Figure 14) is defined as a combination of moment resisting frames and bracing (concentric) frame. Eccentric braced frame is a system used for resisting lateral force especially for resisting seismic events in a predictable manner,. It is associated with the needs of make the structure not collapse during seismic load because it has enough stiffness, ability of adopt during a large seismic force and dissipation of energy Charles W.Roeder & P.Popov (1978). Eccentric steel braced system can may arranged, so that the ends of these members will meet eccentrically not concentrically either in the columns or beam. In eccentric braced frame system, the horizontal lateral forces due to siesmic hazards are resisted by the links by cyclic bending or cyclic shear Landolfo, R. (2014). The aim of using eccentric bracing system is to provide high elastic stiffness for the braced frame system, an inelastic response that is consider as stable under lateral forces due to wind and earthquake, and it can lead to good energy dissipation capacity and ductility for the structure. Eccentric braced frame system usually using the flexural behaviour of the beam section and axial loading that act through the bracing system to resist the lateral forces. This is the reason behind the high energy dissipation capabilities when bracing system subjected to huge lateral forces. This type of bracing helps on controlling the drift by increasing the stiffness in the lateral direction.

Eccentric steel braced frames are usually designed by taking into account that the eccentric bracing members are to be pin-ended, while the connection between beam-column is moment resisting. Also it should be designed to behave in a ductile way through flexural yielding or shear of a link element.

In general, the configurations eccentric braced frames behave in a similar way as the traditional bracing systems except that the end of every eccentric brace member has to be connected to the frame in eccentrical way. Bending moment and shear force of beam in the adjacent area of bracing system are introduced by eccentric connection. The performance of an eccentric braced frame depends on the first place on the links. The classification of the links should be modified taking into consideration the plastic hinges that may occur in these links Kasai, K., & Popov, E. P. (1984, July).

The lateral stiffness in eccentric braced steel frames are related to the length of the link, which is compared to the connected beam length Egor p.popov,Kazuhiko Kasai& Michael D, p. 44 (1987). The part of the frame that connect the eccentric braced members to either beam or column is called “link”, and it is consider as the important characteristic in eccentric braced steel frame. The nonlinear activity and behaviour should be limited to the links, because these links are usually designed as a weak part, but it is ductile and yields before other members in the structure. The link is created through eccentric braced member with either the column centerlines or the beam midpoint. Links can be considered as structural fuses that work on transferring less forces of the lateral loads to the bracing member, beam and column that connected with it. The ductile yielding member remain elastic and stiff due to an normal seismic motion as well as provide ductility protection from buckling in high seismic hazards, but this member produces good energy dissipation, wide, balanced hysteresis loops, which is required for high seismic events Khan, Z., Narayana, B. R., & Raza, S. A. (2015).

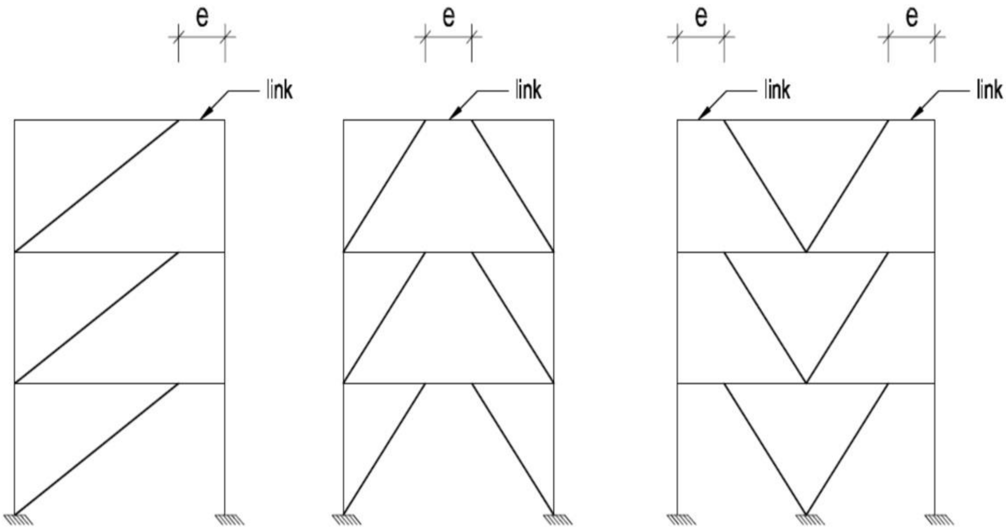

Figure 14: Eccentric Steel Braced Frame

There are some important factors that should be considered during designing the eccentric braced frame. These factors are:

1. Bracing configuration: Selection of an eccentric bracing members has some configuration that is related to various factors, and these factors includes the size and position of required open areas in the structure. Nourbakhs, S. M. (2011).

2. Eccentric member angle: The angel of the inverted V eccentric bracing system (Figure15) should be 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 Michael D.Engelhardt,and Egor p.popov.(1989).

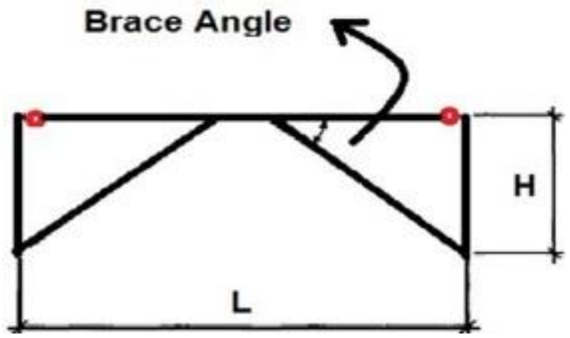

Figure 15: Eccentric Steel Braced Frame Angle

3. The link length: In eccentric braced frame, the length of link can affect the inelastic performance of this link. When this link becomes shorter, the structure will become stiffer and approximately like the concentric braced. But when the link is longer, the frame will become more flexible and close to the stiffness of a moment frame, as well as the long links will yields essentially in bending. When the eccentric braced member subjected to equal shear load in ends of the link of the braced member, then this link will behave as a short beam. The bending moment will be produced at the both ends on the link due to the type of loading, and this moment is usually equal to half of the shear multiplied by the length of the link. Deformation shape of the link will be like S shape at the middle length of the span at flexural counter point. Link lengths usually will be $1.3 \times Ms/Vs$ and it will perform well Egor p.popov, Kazuhiko Kasai and Michael D. 8, p. 46. (1987). When the restraints have not been considered then the initial link length estimates of 0.15L for chevron configurations are reasonable. The behaviour of lengths of the link are as follows:

If $E < 1.3 \times M_s/V_s$ (Eq. 2.1)

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

If $E < 1.6 \times M_s/V_s$ (Eq. 2.2)

Link post - elastic deformation is controlled by shear yielding.

If $E = 2M_s/V_s$ (Eq. 2.3)

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

If $E < 2M_s/V_s$ (Eq. 2.4)

Link behavior considered to be controlled by shear.

If $E > 3\ M_s/V_s$ (Eq. 2.5)

By flexural yielding, Link post-elastic deformation is controlled.

There are some factors that affecting the selection of eccentric bracing:

- The frames usually combine stiffness with behaviour factor which is higher in structure that contain concentric bracing q = 6 instead of q = 4.

- The connections in eccentric bracing system are usually between three elements, but in concentric bracings, it will be four connections. The decreases cost and will result in less complicated connection details.

- The links are considered as a part of the structure, and they will increase the stiffness, as well as it will supporting the gravity loads.

2.2.2 Moment Resisting Frame

Moment resisting frames (Figure 16) can be defined as the connection or joints between column and beam, in which beams and columns should be spliced rigidly. The rigidity of frame members and this connection is the reason that makes this connection resists the lateral forces, and that cause the frame to resist moment. Therefore, it is impossible for moment frame to move horizontally without deformation of the connected beam and column because of the rigidity of the connection. Moment resisting frames are very good energy dissipating systems, but this kind of design require larger beam and column sections since moment resisting frames requires energy dissipating system to achieve the drift requirements Li and Chen (2005). The strength of the frames and bending rigidity will generate the strength and lateral stiffness of the structure. However, the use of larger members means less economical design

Compared to steel braced frames, moment frames required larger member section in order to keep the lateral deflection within limits. The high ductility achieved by moment resisting frames can be challenged by the brittle failures that occur at the connection between beam and column Michel Bruneau et al. (1998). Drift-induced nonstructural damage under earthquake can be introduced by the inherent flexibility of the structure. Beam, column and the panel zone could be part of source of the total plastic deformation at the joint depending accordingly to the yield strength and the yield thresholds. Structural components that dissipate the hysteretic energy during seismic action should be accordingly to make sure that it will allow large plastic hinges rotation and provide the required of plastic energy dissipation, because it have to allow the occurrence of large plastic rotation. This plastic rotation demand of moment frames can be obtained by inelastic history analysis. The plastic rotation capacity that should be achieved in moment resisting frame increased to 0.03 radian in the new constructed buildings SAC (1995). Moment resisting frames are generally not adequate for the stiffening tall buildings due to its high cost as mentioned by Kameshki and Saka (2001).

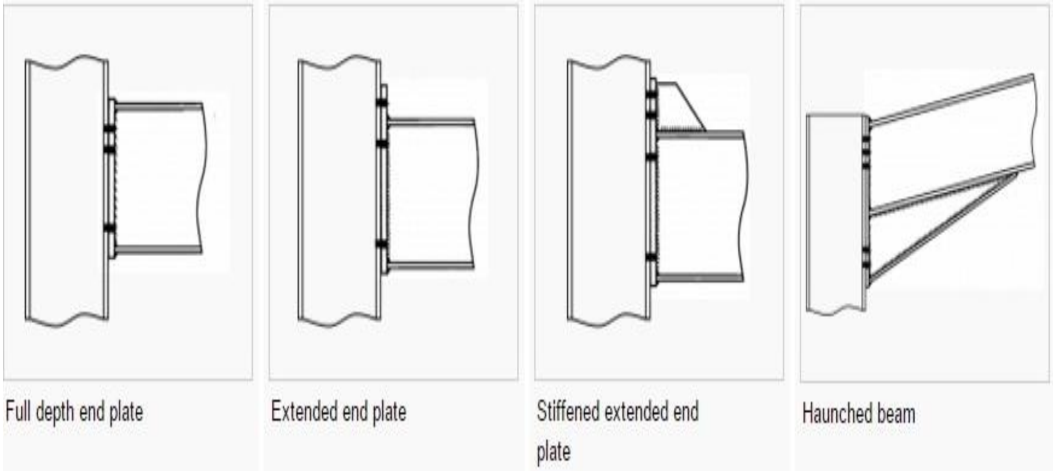

Figure 16: Moment Resisting Frame

2.3 Types of Analysis

Structures response curve should be evaluated in order to get the structure analysis and the structural response. There are some methods for obtaining the response curves of structure. The following four methods are widely used for structural analysis are four methods: Linear Static, Non-Linear Static, Linear Dynamic and Non-Linear Dynamic analysis.

In static loads, the acceleration is much less than the natural frequency of the structure. While the dynamic loads are changes quickly if it is compared with the natural frequency of structure. However, the field of structural engineering will never be automated. The idea that an expert system computer program, with artificial intelligence, will replace a creative human is an insult to all structural engineers.

There are several differences between linear and non-linear analysis.

1. Linear analysis

   - Structure can return to its original form after analysis done.

   - Material properties will not change.

- Small deformations and strains can occur in the shape and stiffness of structure.

- Loading direction or magnitude that applied on the structure will not change.

2. Nonlinear analysis

- Deformation in steel may not return to its original shape.

- Changes in the geometry of structure due to changes in stiffness.

- Support in the nonlinear curves of loads.

- Nonlinear analysis may support the changes in load constraint locations and the load direction.

2.3.1 Linear Static Analysis

Linear static analysis is a method to obtain reactions forces, strains, displacements, and stresses forces under the effect of applied loads. It determines the deflections that are close to the predicted deflections of the structure according to size of structure. Usually, the deflection between two supports must present a small percentage of the full distance between these two supports if the deflection may cause differential stiffness effect on the structure. Furthermore, the rotations in linear static analysis are very small, and tangent of any angle must be approximately equal to the angle measured in radians Di Julio Jr, R. M. (2001). Therefore, Linear static method is a simplified technique to substitute the effect of dynamic loading of an expected earthquake by a static force that can be distributed as lateral forces on the structure of building for design aims Bourahla, N. (2013).

Linear Static Analysis consider as the simplest method of structural analysis. The building should be modeled near the yield level or with linearly elastic level considering damping values as well as the stiffness. The actual internal forces during the yielding stage of the building might be different than the forces that can be calculated using linear static method during the inelastic response of structure. In linear static analysis procedure, the inertial forces are specified as static forces by using empirical formulas. The building responds due to linear static method assumed that its in fundamental mode. Therefore, the building should not be twisted when ground motion occur, and should be low-rise. The response can be read by given the frequency of the building that can be defined from the code of designed building or calculated. The equivalent static lateral force method is a simplified technique to substitute the effect of dynamic loading of an expected earthquake by a static force distributed laterally on a structure for design purposes.

Mohammadi and EL NAGGAR (2004) state that for a performance of structural design using linear static analysis, maximum inter story drift and maximum roof displacement should be estimated. Reasons are:

- Checking P-delta effects.
- Support to estimate maximum damage.
- Checking deformation capacity of critical structural members.
- Detailing connections for nonstructural components.
- Estimating minimum building separation to avoid pounding.

Table 1: Buildings for which Equivalent Seismic Load Method is Applicable

Seismic Zone	Type of Building	Total Height Limit
1,2	Buildings without type A1 torsional irregularity, or those satisfying the condition h_{bi} £ 2.0 at every storey	$H_N \leq 25$ m
1,2	Buildings without type A1 torsional irregularity, or those satisfying the condition h_{bi} £ 2.0 at every storey and at the same time without type B2 irregularity	$H_N \leq 60$ m
3,4	All buildings	$H_N \leq 75$ m

Linear static analysis depends of some assumptions:

- The structure of the building is assumed rigid.
- The fixity between structure and foundation assumed perfect.
- Every point of the structure is assumed to have the same accelerations due to the earthquake.
- The magnitude of the horizontal forces of earthquake assumed that has dominant effect on the structure, and this forces vary at each floor (varying over the height of floors).

2.3.2 Nonlinear Static (Pushover) Analysis

Nonlinear static analysis it is a method to assess the actual strength of the structure. It is method for designing based on the performance. Nonlinear static analysis of a structure have been used between 1960s to1970s for a reason of investigating the stability of steel structure by using a specified force pattern from zero load to a prescribed ultimate displacement.

The failure modes that obtained from the structure under pushover analysis can be obtained at the same time in which the amounts of applied pushover loads are increasing Nourbakhs, S. M. (2011).

Hinges can be introduced and formed in nonlinear static analysis, and to explain the behaviour of these hinges, base shear-displacement curve will be illustrated. There are five points: A, B, C, D and E (Figure 17), will be designated and that points will define the force deflection performance and behaviour of the hinges. In addition, there are three points which are IO (immediate occupancy), LS (life safety) and CP (collapse prevention), in which the value of these points vary according to the basis of considered parameters and the type of member, as well as these points will identify the acceptance criteria of hinges.

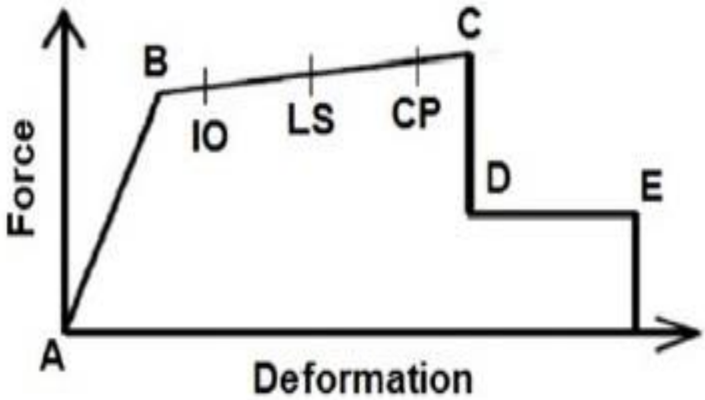

Figure 17: Force _Deformation of Hinges for Pushover Analysis

Nonlinear static analysis is related to the assumption that structures oscillate usually on either lower mode of vibrations or the first mode due to earthquake action Themelis, S. (2008). ETABS software or SAP2000 can be used for modeling three-dimensional structural and nonlinear static analysis.

2.3.2.1 Previous Researches on Nonlinear Static Analysis

The nonlinear static analysis method introduced for the first time by Freeman et al. (1975) as the capacity spectrum method. The main aim of NLA method was to use a simple and fast method of analysis in order to assess the earthquake effects on 80 buildings located in USA. In that study, site response spectra was combined with another analytical method in order to obtain peak ductility demands, residual capacities, equivalent period of vibration, the peak values of structural response and the equivalent percentage of critical damping. In the end, this study concluded that this method might perform in a reasonable time and cost a worthwhile elevation of the structure.

Saiidi M., Sozen M.A. (1981) found low cost analytical or the Q-Model has been used for calculating the multi-story displacement for reinforced concrete building that subjected to earthquake action. Gulkan et al. (1974) found that the Q-model, and it was involved two facilitations, the first facility was about the reducing the multi degree of freedom to single degree of freedom, while the second facility was about the properties differences of stiffness in structure by using a single spring in order to consider the relationships of nonlinear displacement due to applied force that characterize its properties. The experiments performed on eight small scale structure, and the obtained results of displacement compared with the results from Q-Model analysis that depends on nonlinear static analysis of structure. The results of performance of Q-Model analysis were satisfactory for most of the test structures for high and low amplitude responses. Saidi and Sozen (1981) state that the model may need to be further validated by more experimental and theoretical analyses.

Fajfar and Fischinger (1988) presented the N2 method, which is a variation of pushover analysis. The study was on a seven-storey reinforced concrete structure using uniform and inverted triangular load distributions in Tsukuba, Japan as part of the joint U.S, and aimed to perform nonlinear analysis of the structure. The curves that result from the nonlinear analysis were compared with the nonlinear dynamic experimental and analytical in order to show the differences in the used shapes. In addition, authors observer that the nonlinear dynamic analysis of single degree of freedom structure yielded in general non-conservative shear forces. The displacement at the ultimate limit state and the rotations of the floors were approximated satisfactorily compared with the experimental and theoretical results.
Chopra and Goel (2001) developed the modal of nonlinear static analysis procedures. The authors tried to estimate the seismic story-drift demands that should be sufficient for most structural design were accurate to a degree that as well as retrofit applications. The height-wise distribution of seismic storey drift demands determined by nonlinear static analysis was exactly same with the results from nonlinear RHA.

The procedure of nonlinear static analysis was accurate more than the can be obtained using the force distribution.

Penelis and Kappos (2002) performed a 3D nonlinear static analysis that aim to include the torsional effects. The study done by achieving the mass load vectors at the center of two single-storey structural building. In the end, nonlinear static analysis procedures of structure can be hinted as a separation of the capacity in the structure and demand on an applied earthquake. The distribution of internal force after the elastic stage of response for a structure shows that this separation is not justifiable compared with the internal force.

2.3.3 Response Spectrum Analysis

Response of a structure due to response spectrum method is combination of many special shapes or modes that in a vibrating string correspond to the harmonics. Response spectrum analysis is a method can obtain the contribution of vibration in each mode to identify the maximum response of structure due to earthquake. Response spectrum analysis can be considered as a linear dynamic statistical analysis method that can provides insight into dynamic behavior by measuring the displacement, velocity or pseudo-spectral acceleration for a specific damping ratio as a function of structural period Chopra, A. K. (1995). The response spectrum analysis is useful method for designing the structure, because it relates structural type-selection to dynamic performance. Structures will give a greater acceleration in shorter period, and the longer period will lead to greater displacement. The performance of structural members should be considered during using response spectrum analysis and design stages.

Response spectrum analysis can be defined as curves that plotted between maximum response of single degree of freedom system that subjected to specified seismic motion and its time period or frequency. Response spectrum can be interpreted at the area of maximum response of a single degree of freedom system for given damping ratio. Therefore, response spectrum analysis can help to obtain the peak response of structure under linear range and use it to obtain the lateral loads on the structure during seismic actions to make the design of structure that can resist lateral loads easier. The responses of single degree of freedom can be estimated by domain analysis, and for a given time period of system until the maximum response of structure will be picked. This analysis will continue for all possible ranges time periods for single degree of freedom system, then the plot a response spectrum curve for a specific damping ration and other parameters of earthquake motion, in which time period will represent the x-axis and response quantity on y-axis (Figure 18), then this process will continue for different damping ratios to obtain overall response spectra Chopra, A. K. (2007). The response spectrum analysis is the most widely used method in seismic analysis, because it can account for irregularities as well as higher mode contributions and gives more accurate results.

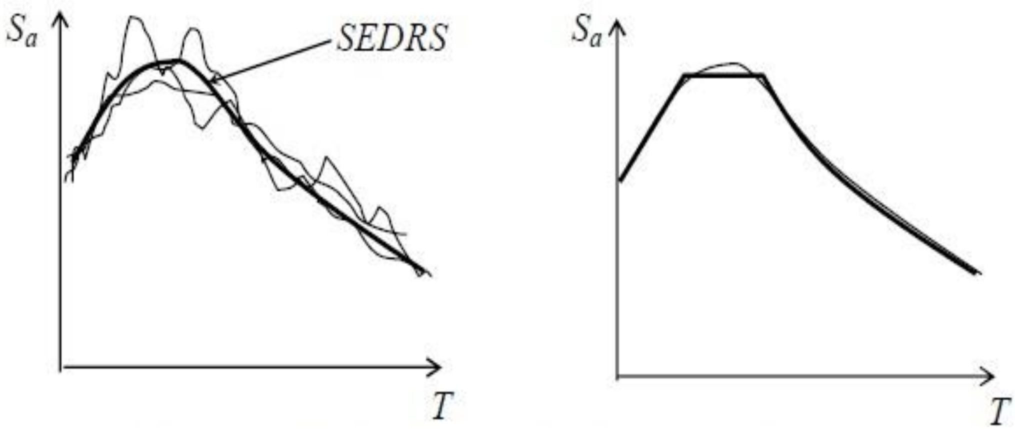

Figure 18: Developing the Design Response Spectrum Duggal, S. K. (2013).

The structural response of a building during the earthquake action using linear dynamic analysis can be calculated in the time domain, therefore, all phase information will be maintained. Linear dynamic method will use modal decomposition as a means of reducing the degrees of freedom in the analysis. In the structure, the single degree of freedom has mass m, stiffness k and structural damping ξ. Thus mass and stiffness have the same natural period of $2\pi\ m\ k$ (Figure19) Murty and Goswami and Vijayanarayanan and Mehta (2012).

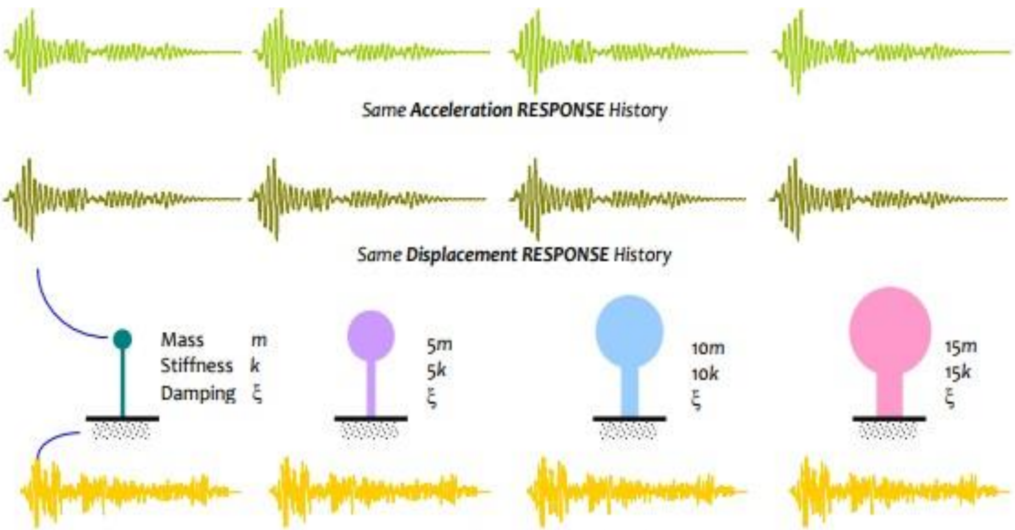

Figure 19: Acceleration and Displacement of different Masses with same Natural period and with same Damping Duggal, S. K. (2013).

Clough and Penzien (1993) state that, when the natural periods are sufficiently apart, the square root of the sum of the squares (SRSS) method will be the most common way of combining the maximum responses of structure.

When the structure is significance to a community in disaster response, irregular or tall, the linear dynamic response spectrum analysis in this case is not appropriate, and it will require more complex analysis, such as nonlinear static analysis or nonlinear dynamic analysis.

2.3.3.1 Previous Researches on Response Spectrum Analysis

Ruiz-García and González. (2014), studied the strategy for seismic coefficient on delicate soft soil for an existing structure. The study was on six reinforced concrete frames and four steel casings under 20 distinctive seismic stacking condition and horizontal uprooting is analyzed. The results concluded that seismic coefficient was used to estimate the maximum displacement of the roof for any structure with inelastic behaviour, as well as the results of the building described that the effect of seismic response depends on ratio of period of vibration with respect to the ground motion. The results were more accurate than the results that obtained from statistics method.

Moschen and Adam and Vamvatsikos (2016), represents a method of response spectrum for peak floor response for any type structure, and explained the concept of stochastic base excitation for various high rise building. The all present day technique and mix of strategy are considered. The modal was prepared with quadratic combination. This paper aimed to compares modern quadratic rule with modal displacement for calculation. The tests of multi-story structures at various planes have done with consideration of particular ground motion technique. The tests have embraced both flexible and inelastic structures all the while.

Chen and Racic (2016), explained the impact and reaction of floor acceleration due to earthquake loads with considering a jumping occurs with n the floor. The experiment was on 506 recorded jumping forces. At each record, single degree of freedom with calculated damping ration according to the response spectrum analysis, varying frequency, design spectra (0.5 Hz - 1.5 Hz) was considered. Response spectrum curve was plotted as per the results obtained by statistical method. The experiment and analysis done under various floor models. The results of this study concluded that response spectrum varies according to the existing floor design to any individual or crowd. Holmes and Tamura and Krishna (2009), described a comparison study of wind load calculations on three buildings. The analysis and experiments used fifteen different wind-loading codes. The building was low rise typical warehouse steel industrial framed buildings assumed to be located in a rural area. The design of the structure was considered to be designed according to the wind speed at the top of each building with other wind parameters such as turbulence intensity were prescribed. The results of this study showed that building with low rise have a large coefficient of variation comparing to the complexities in the Buildings, while building with high rise results with a significant amount of resonant dynamic response to wind that makes acceleration, bending moments and the elevation of base shear complicated at the top of the building.

2.4 Significance

Lateral stability for steel structure has been one of the important problems that can affect the structure behaviour and performance due to earthquake and especially in the high seismic hazard regions. This issue pushed the engineer to find a lateral resisting system in order to avoid the immediate collapse of structure. Steel structure should resist the lateral loads by using of one the mentioned system either steel braced frame of moment resisting frames.

Economy and performance are the two factors can effects on the chosen structural members section as well as the type of the used braced frame system. The performance of chosen braced frame will effect on the structure. In addition, the method used for analyze the structure will also affect the used section of steel structure and the type of bracing system that has more plastic deformation capacity ability to collapse and absorb energy due to earthquake action. By making a comparison between different braced frames and using different types of analysis methods, it will make choosing of bracing system easier.

Number of stories, number of bays, types of bracing system, plan shape and analysis method will be tested in this study in order to obtain the effective braced frame for a different specification of structures, which depends on the performance, and total weight of the structural members.

Chapter 3

DESIGN OF MODEL STRUCTURES

This chapter will contain two sections, the first section will describe the methodology and basis of the design, and the second section will list the design of sections. The units to be used in this study are Kg, kN and meter for weights, forces and distances respectively.

In this study the frames geometry will be designed and analyzed in section 3.1. The economical comparison between different bracing systems will be given in section 3.2. The chosen of 2-D and 3-D models will be explained in section 3.3. Design criteria are in section 3.4. The design materials will be given in section 3.5. The loading consideration will be described in section 3.6. The section 3.7 will describe the method of analysis used for this study, includes linear static, pushover and response spectrum analysis. While the discussion of selected software are given in section 3.8.

3.1 Frames Geometry

Bracing system should be designed and analyzed in order to make comparison between different types of bracing system. Therefore, bracing models, shapes and sizes should be adequate for purpose of design during linear behaviour outcomes of the frames (Tremblay, 2002, Kim & Choi, 2005, D. Ozhendekci & N. Ozhendekci, 2008).

The information of models geometry is:

- The numbers of stories in the building are assumed to be 4 and 12-story which considered to be low and medium rise.

- There are two different plan section assumed to be used for this study. The first plan (Figure 20) chosen to be symmetric regular plan with 6 frame bays and 5 meter length for each bay. The second plan (Figure 21) assumed to be symmetric irregular plan with 6 frame bays and 5 meter length for each bay.

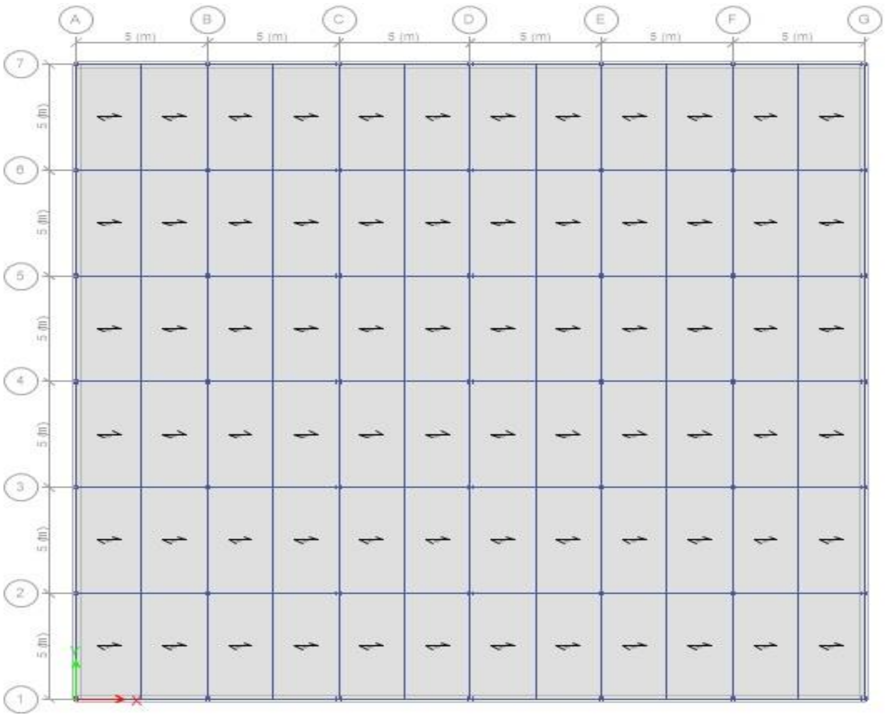

Figure 20: Regular Symmetric Plan

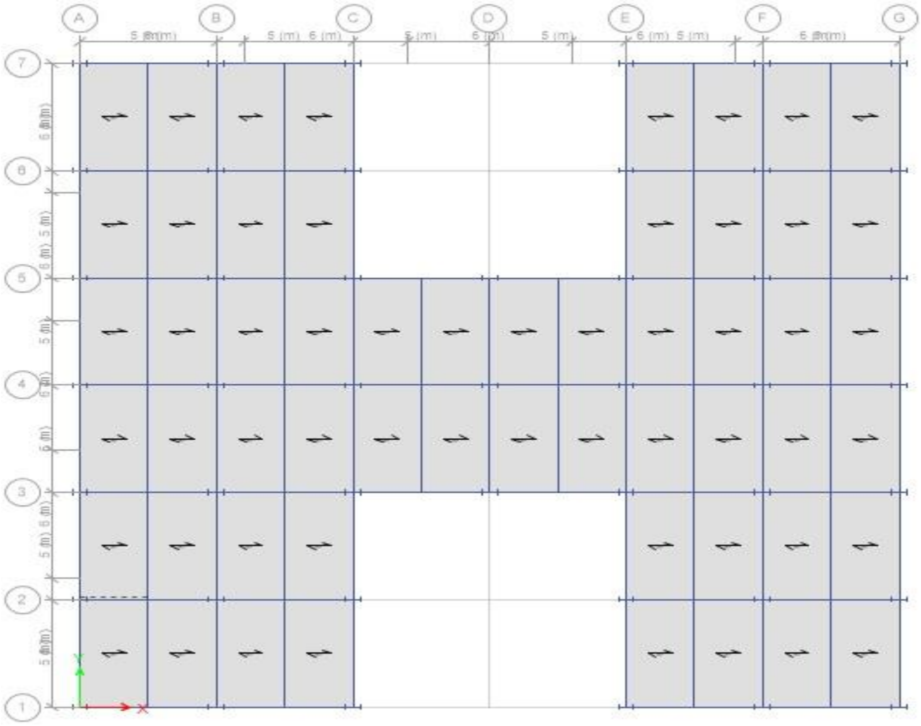

Figure 21: Irregular Symmetric Plan

• The two chosen eccentric braced frames are inverted V-bracing and diagonal bracing as concentric bracing system (Figure 22), and inverted V-bracing and diagonal bracing eccentric bracing system (Figure 23).

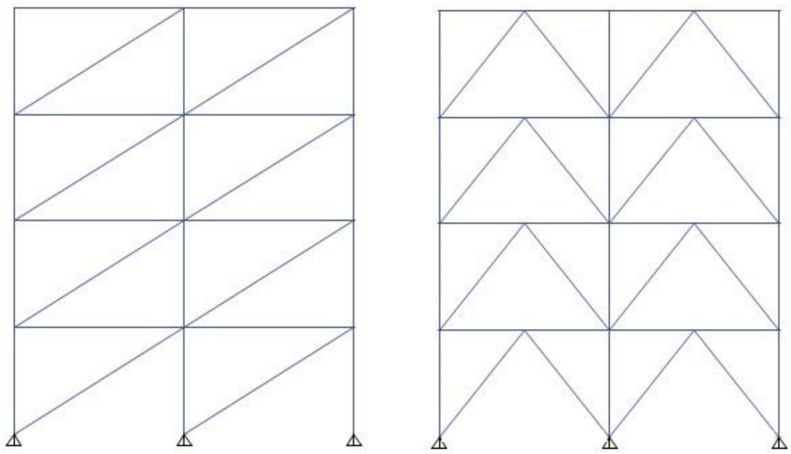

Figure 22: Chosen Concentric Diagonal and Inverted V Bracing

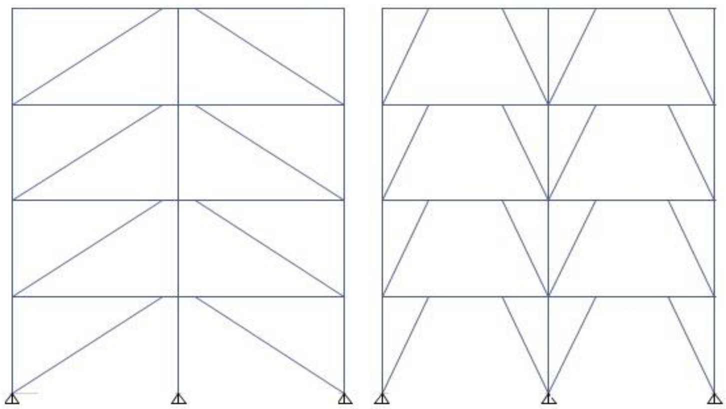

Figure 23: Chosen Eccentric Diagonal and Inverted V Bracing

- The assumed height of all stories is 3 meters.
- All frames will be braced against lateral loading with bracing system.

3.2 Economical Comparison (Calculation of the Frame Weight).

Tremblay (2002), Richards (2009) and (Kim & Choi, 2005) state that effect the seismic column demand (and consequently weight) and seismic beam demand. Therefore, the economical comparison between the bracing systems cannot be done only by checking the weight of the bracing members, but it should be checked for the whole building because lateral force effect also the weight and section of the beams and columns Saka (2001) and D. Ozhendekci and N. Ozhendekci (2008).

3.3 2D versus 3D Models

2D or 3D modeling is another factor that should be taken into consideration during design and analysis because its effects on the computer analysis time. Choosing between 2D and 3D modeling depends on the degree of regularity. Therefore, decision was difficult because 2D modeling is better for showing the behaviour of structure, while 3D modeling requires more time for analysis as well as its decrease the analysis speed. On the other hand, 3D modeling consider as more realistic and it can by this model rotational drift can be obtained for the building. In this study 3D modeling (Figure 24 and 25) considered to be used for analyzing.

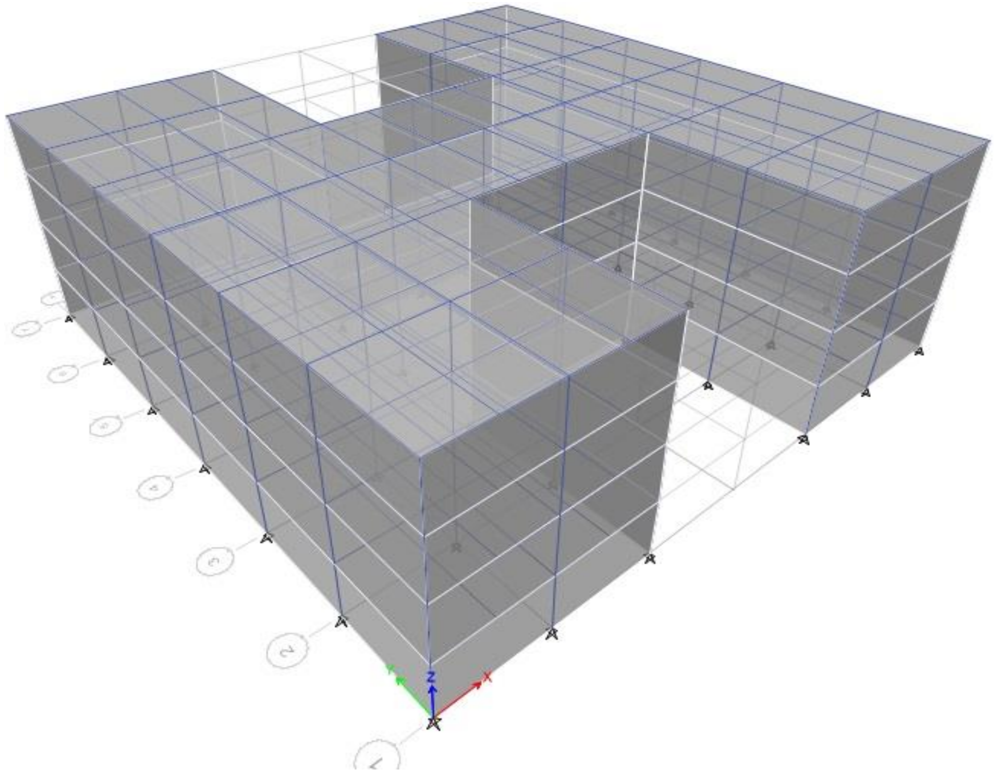

Figure 24: 3D View of Irregular plan Symmetric

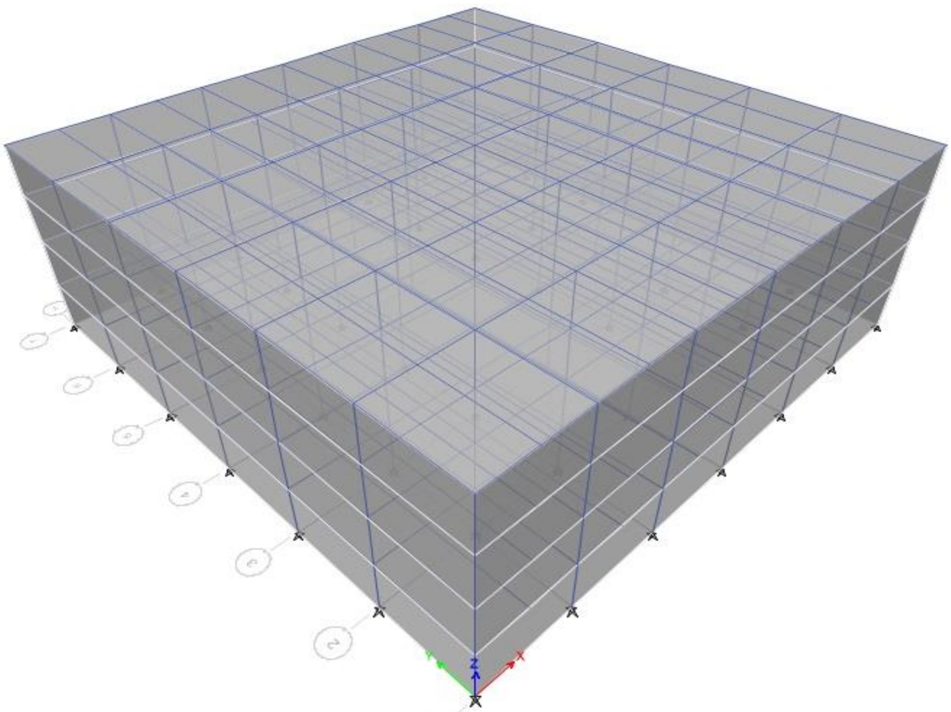

Figure 25 3D View of Regular plan Symmetric

3.4 Design Criteria

Steel design will be taken according Eurocode (1991). In addition, Eurocode BS EN (1991) will be used as loading code. The manual of steel Construction will be taken from seventh Edition of Steel Designers' Manual (2011) and for availability of materials and sections. Richard Lees Steel Decking manual will be used for estimating the slab properties including its depths in order to calculate the dead load of frames.

3.5 Design Materials

Materials property of steel that will be used in this study will be according to BS EN (1991) manual. The chosen steel properties for this study are given below:

- $Minimum\ Yield\ Stress$: $275\ N/mm^2$
- $Mass\ per\ Unit\ Volume$: $7850\ Kg/m^3$
- $Poisson's\ Ratio$: $\nu =\ 0.3$
- $Weight\ per\ Unit\ Volume$: $76.98\ KN/m^3$

3.6 Loading Consideration

Loading consideration will be according to BS EN (1991-1-1), which is consider close to the Turkish code as well as it is one of the most used loading codes worldwide.

3.6.1 Dead Load

According to BS EN (1991-1-1), dead load represents the total self-weight of structural and non-structural members and attachments or accessories, and it should be taken into account in combinations of actions as a single action. Weight of structural and non-structural members will be determined after the analysis will be done, but the weight of steel deck and concrete will be specified in this section.

According to Richard Lees Steel Decking manual, the chosen steel deck is listed below:

- Ribdeck which made by Aluminum will be chosen as deck for this study. The Ribdeck dimensions are provided in figure below (Figure 26).

Figure 26: Chosen Deck Dimensions (Richard Lees Steel Deck Manual)

- The Ribdeck properties chosen to be according to RLSD (Table 2).

Table 2: Deck Properties (Richard Lees Steel Deck Manual)

Ribdeck AL Section Properties (per metre width)					
Gauge mm	Self Weight		Area mm²	Inertia cm⁴	Y NA mm
	kg/m²	kN/m²			
0.9	9.5	0.093	1,171	67.4	28.0
1.0	10.5	0.103	1,301	75.2	28.0
1.2	12.6	0.124	1,570	90.9	28.0

3.6.2 Additional Dead Load

The additional dead load will include two loads:

- Screed load that will be distributed on the slabs. The unit weight of screed is $18 kN/m^3$. Therefore, Self weight of screed will be: 18 x 0.8 = 1.5 KN/m^2

- The walls assumed to be distributed on the whole beams with the its finishing. The chosen brick will be Tugla Fabrikasi Tgd-20 bricks (100x200x300) mm produced by Tugla (Figure 27). Therefore, the weight of wall with finishing will be 6 kN/m

Figure 27: Brick Properties (Richard Lees Steel Deck Manual)

3.6.3 Imposed Load

Live load or imposed load on buildings is those arising from occupancy BS EN (1991-1-1). According to BS EN (1991), imposed load depends on type of the building or it is category. The building is assumed to be in category B (Table 3) which is offices building. According to BS EN (1991-1-1), the imposed load for the will between $2.0 - 3.0\ KN/m^2$, but it will be taken as $3.0\ KN/m^2$ (Table 4).

Table 3: Categories of Building Uses Eurocode 8, BS EN 1998-1 2004..

Category	Specific Use	Example
A	Areas for domestic and residential activities	Rooms in residential buildings and houses; bedrooms and wards in hospitals; bedrooms in hotels and hostels kitchens and toilets.
B	Office areas	
C	Areas where people may congregate (with the exception of areas defined under category A, B, and D[1)])	**C1**: Areas with tables, etc. e.g. areas in schools, cafés, restaurants, dining halls, reading rooms, receptions. **C2**: Areas with fixed seats, e.g. areas in churches, theatres or cinemas, conference rooms, lecture halls, assembly halls, waiting rooms, railway waiting rooms. **C3:** Areas without obstacles for moving people, e.g. areas in museums, exhibition rooms, etc. and access areas in public and administration buildings, hotels, hospitals, railway station forecourts. **C4**: Areas with possible physical activities, e.g. dance halls, gymnastic rooms, stages. **C5**: Areas susceptible to large crowds, e.g. in buildings for public events like concert halls, sports halls including stands, terraces and access areas and railway platforms.
D	Shopping areas	**D1:** Areas in general retail shops **D2**: Areas in department stores

Table 4: Imposed Load on floor Eurocode 8, BS EN 1-1-1991.

Categories of loaded areas	q_k [kN/m²]	Q_k [kN]
Category A		
- Floors	1,5 to2,0	2,0 to 3,0
- Stairs	2,0 to4,0	2,0 to 4,0
- Balconies	2,5 to 4,0	2,0 to 3,0
Category B	2,0 to 3,0	1,5 to 4,5
Category C		
- C1	2,0 to 3,0	3,0 to 4,0
- C2	3,0 to 4,0	2,5 to 7,0 (4,0)
- C3	3,0 to 5,0	4,0 to 7,0
- C4	4,5 to 5,0	3,5 to 7,0
- C5	5,0 to 7,5	3,5 to 4,5
category D		
- D1	4,0 to 5,0	3,5 to 7,0 (4,0)
- D2	4,0 to 5,0	3,5 to 7,0

3.6.4 Wind Load

The wind action should be determined for each design situation identified BS EN (1990-1-4). In this study, Famagusta city in Northern Cyprus republic is taken as a location for calculation of wind load. The fundamental wind velocity in Famagusta city is $35\ m/s$. According to BS EN (1991-1-4), wind load can be calculated by the following:

1. Basic Wind Velocity

$V_b = C_{dir} * C_{season} * C_{prob} * V_{bo}$;   Where: (Eq. 3.1)

V_{b0} is the fundamental value of Basic Wind Velocity.

V_b is the Basic Wind Velocity.

C_{dir} is the directional factor, and its vary form direction to another, and it will equal to 1.0 because the wind will acts on both directions.

C_{season} is the season factor which is equal to 1.0 if the bulding will be used in the whole year.

C_{prob} *is the probability of wind velocity*

$$C_{prob} = \left(\frac{1-K*\ln(-\ln(1-p))}{1-K*\ln(-\ln(0.98))}\right)^{n} \qquad \text{(Eq. 3.2)}$$

K is the shape parameter depending on the coefficient of variation of the extreme value.

2. Mean Wind

$$V_m(z) = C_r(z) * C_o(z) * V_b; \quad \text{Where:} \qquad \text{(Eq. 3.3)}$$

$C_o(z)$ is the orography factor

$C_r(z)$ Is the roughness factor and it can be calculated according to the height Z of each floor.

$$C_r(z) = K_r * \ln\left(\frac{z}{z_o}\right) \quad \text{for} \quad z_{min} \leq z \leq z_{mzx}, \quad \text{for } z_{max} = 200 \text{ m.} \qquad \text{(Eq. 3.4)}$$

$$K_r = 0.19 * \left(\frac{Z_0}{Z_{o,II}}\right)^{0.07} \qquad \text{(Eq. 3.5)}$$

Z_{min} and Z_0 can be estimated from BS EN (1991-1-4) as shown in the table below in table 5.

Table 5: Terrain Categories and terrain Parameters for $Z_{min} and Z_0$. Eurocode 8, BS EN 1998-1 2004.

Terrain category		z_0 m	z_{min} m
0	Sea or coastal area exposed to the open sea	0,003	1
I	Lakes or flat and horizontal area with negligible vegetation and without obstacles	0,01	1
II	Area with low vegetation such as grass and isolated obstacles (trees, buildings) with separations of at least 20 obstacle heights	0,05	2
III	Area with regular cover of vegetation or buildings or with isolated obstacles with separations of maximum 20 obstacle heights (such as villages, suburban terrain, permanent forest)	0,3	5
IV	Area in which at least 15 % of the surface is covered with buildings and their average height exceeds 15 m	1,0	10

$Z_{o,II}$ it is the terrain categories, and according to BS EN (1991-1-4) it is equal to 0.05 (Figure 28).

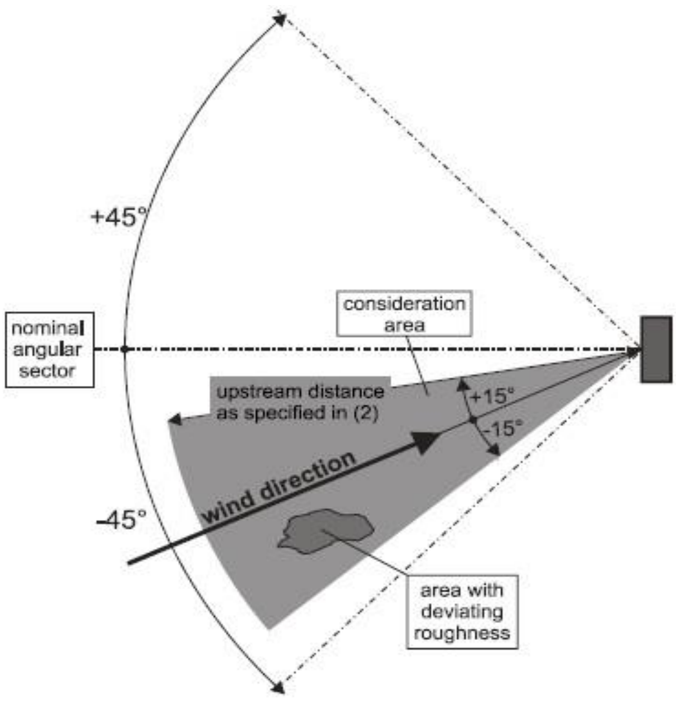

Figure 28: Terrain Categories for $Z_{o,II}$. Eurocode 8, BS EN 1998-1 2004.

3.7 Methods of Analysis

The structure will be designed first with linear static analysis in order to obtain the design sections for structure for all plans and number of stories.

3.7.1 Linear Static Analysis

Linear static method can be obtained by apply some factors to obtain low levels of twisting, and higher buildings for modes of the building. In addition, this method can obtain the yielding of structure by applying modification factors that reduce the design factor such as reduction factor Hughes, T. J. (2012). In general, linear static can obtain the lateral force that acting on the structure of the building during earthquake as equivalent static force. Eurocode 8 described that this type of analysis may be applied to buildings whose response is not significantly affected by contributions from modes of vibration higher than the fundamental mode in each principal direction. Linear static analysis applied for buildings when response of structure is not significantly affected by contributions from modes of vibration higher than the fundamental mode in each principal direction. It assumes that the building responds in its fundamental lateral mode Bourahla, N (2013). There are some factors effects the calculation of the base shear force V_B.

3.7.1.1 Zone factor

The seismic zone can assess the maximum dangerous of shaking the ground due to earthquake that is unpredicted. Therefore, zone factor (Z) (Table 6) is used to obtain the design spectrum depends on the location of structure in which seismic hazards perceived in that location. The zone factor can affect the peak ground acceleration according to the used code.

Table 6: Effective Ground Acceleration Coefficient (Z). Eurocode 8, BS EN 1998-1 2004.

Seismic zone	II	III	IV	V
Seismic intensity	Low	Moderate	Severe	Very Severe
Z	0.10	0.16	0.24	0.36

3.7.1.2 Importance factor

It is factor used to obtain seismic design force depending on the type of the building or what this building will be used for. Table 7 will express the importance class and importance factor.

Table 7: Importance Class and Importance Factor I for different types of Building Eurocode 8, BS EN 1998-1 2004.

Importance class	Buildings	Importance Factor I
I	Buildings of minor importance for public safety, e.g. agricultural buildings, etc.	1.0
II	Ordinary buildings, not belonging in the other categories.	1.2
III	Buildings whose seismic resistance is of importance in view of the consequences associated with a collapse, e.g. schools, assembly halls, cultural institutions etc.	1.4
IV	Buildings whose integrity during earthquakes is of vital importance for civil protection, e.g. hospitals, fire stations, power plants, etc.	1.5

3.7.1.3 Response reduction factor

The aim of designing a strong structure is to resist against ground motion and should not collapse. Structure should be designed to resist lateral forces to save the structure from damage in case of severe ground motion. Response reduction factor is used to reduce the base shear force and calculate the design lateral force. Response reduction factor values for different types lateral resisting systems are defined in table 8.

Table 8: Structural Behaviour Factor R Eurocode 8, BS EN 1998-1 2004.

BUILDING STRUCTURAL SYSTEM	*Systems of Nominal Ductility Level*	*Systems of High Ductility Level*
(1) CAST-IN-SITU REINFORCED CONCRETE BUILDINGS		
(1.1) Buildings in which seismic loads are fully resisted by frames..........	4	8
(1.2) Buildings in which seismic loads are fully resisted by coupled structural walls..........	4	7
(1.3) Buildings in which seismic loads are fully resisted by solid structural walls..........	4	6
(1.4) Buildings in which seismic loads are jointly resisted by frames and solid and/or coupled structural walls..........	4	7
(2) PREFABRICATED REINFORCED CONCRETE BUILDINGS		
(2.1) Buildings in which seismic loads are fully resisted by frames with connections capable of cyclic moment transfer	3	6
(2.2) Buildings in which seismic loads are fully resisted by single-storey hinged frames with fixed-in bases..........	—	5
(2.3) Buildings in which seismic loads are fully resisted by prefabricated solid structural walls..........	—	4
(2.4) Buildings in which seismic loads are jointly resisted by frames with connections capable of cyclic moment transfer and cast-in-situ solid and/or coupled structural walls	3	5
(3) STRUCTURAL STEEL BUILDINGS		
(3.1) Buildings in which seismic loads are fully resisted by frames..........	5	8
(3.2) Buildings in which seismic loads are fully resisted by single-storey hinged frames with fixed-in bases..........	4	6
(3.3) Buildings in which seismic loads are fully resisted by braced frames or cast-in-situ reinforced concrete structural walls		
(a) *Concentrically braced frames*..........	3	—
(b) *Eccentrically braced frames*..........	—	7
(c) *Reinforced concrete structural walls*..........	4	6
(3.4) Buildings in which seismic loads are jointly resisted by frames and braced frames or cast-in-situ reinforced concrete structural walls		
(a) *Concentrically braced frames*..........	4	—
(b) *Eccentrically braced frames*..........	—	8
(c) *Reinforced concrete structural walls*..........	4	7

3.7.1.4 Fundamental natural period

It is the first modal time or period in the structure due to vibration, that generated by earthquake. Building period cannot be calculated unless finishing design stage, but the design loading is depends on building period. Therefore, the period T_a will be calculate from the following equations:

- For building in which moment resisting frames used without using brick infill panels

$$\mathrm{T_a} = 0.075 \times \mathrm{h}^{0.75} \quad \text{For RC building} \qquad \text{(Eq. 3.6)}$$

$$\mathrm{T_a} = 0.085 \times \mathrm{h}^{0.75} \quad \text{For steel building} \qquad \text{(Eq. 3.7)}$$

- For all other building with brick infill panels with moment resisting frames

$$\mathrm{T_a} = \frac{0.09 \times \mathrm{h}}{\sqrt{d}} \qquad \text{(Eq. 3.8)}$$

In which h is the height of building.

3.7.1.5 Design response spectrum

It is the specifying of level of the lateral resisting system in the building to be designed. Base shear coefficients are ordinates of the acceleration of spectrum divided by acceleration due to gravity Duggal, S. K. (2013). In figure 29, the relationship between period per second and response acceleration coefficient is illustrated.

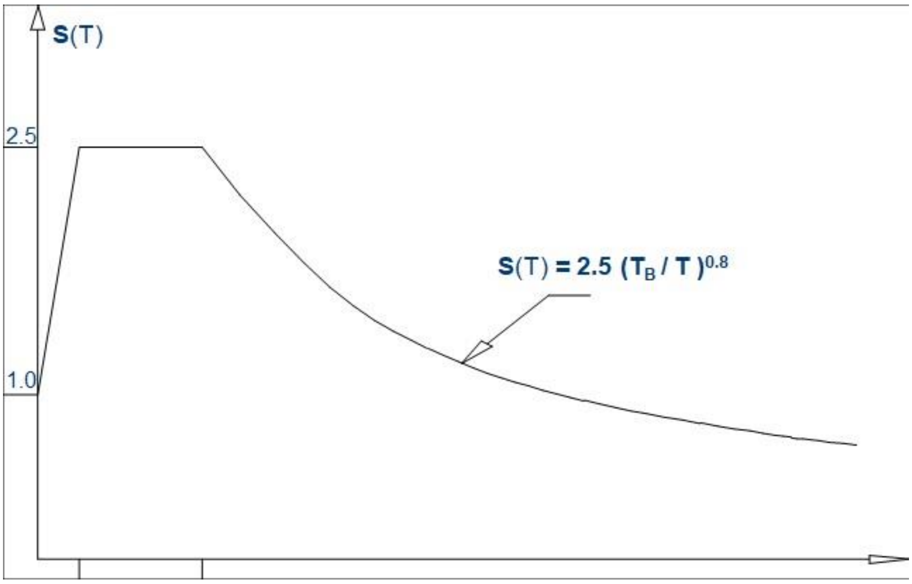

Figure 29: Relationship between Period and Response Acceleration Coefficient Eurocode 8, BS EN 1998-1 2004.

The lateral forces analysis using linear static method can be obtained by calculating the base shear force V_B using the following equations:

1. $V_B = A_h \times W$; Where W is the total weight of the structure. (Eq. 3.9)

2. $A_h = \frac{Z \times I \times S_a}{2 \times R \times g}$ (Eq. 3.10)

Where Z is the zone factor, I is importance factor, R is response reduction facto, g is the gravity and S_a/g is the response acceleration coefficient and it is depends on the damping factor and period factor as shown in table 9.

Table 9: Damping Factors for different percentages Eurocode 8, BS EN 1998-1 2004.

Damping (Percent)	0	2	5	7	10	15	20	25	30
Factors	3.20	1.40	1.00	0.90	0.80	0.70	0.60	0.55	0.50

3. The distribution of the design force or base shear force on each floor can be obtained from the equation:

$$Q_i = V_B \times \frac{W_i \times h_i^2}{\sum_{j=1}^{n} W_j \times h_j^2} \qquad \text{(Eq. 3.11)}$$

Where W_i, h_i is the weight and height of the floor and W_j, h_j is the total weight and height of the building.

3.7.2 Pushover Analysis

3.7.2.1 Assessment of Nonlinear Behavior of Pushover Analysis

The response of structure curve is the most important factor as mentioned in section.2.3.2. This curve can estimates the parameters nonlinear analysis of a structure such as displacement amplification factor, modification of response factor as well as the factor of over-strength that can be taken from the demand-capacity curve of the structure that obtained from pushover analysis.

3.7.2.2 Pushover Load Pattern

Inverted triangular load pattern will be used in this study, which depends of the lateral load distribution proportional gives the most accurate results for the first mode of the structure. The global drift will be used in order to check the failure of structure, as well first mode of load vector will be used to obtain SRSS load vector for interstory drift estimation.

3.7.2.3 Displacement During Pushover Analysis

Force and displacement methods that introduced by displacement modification curve (Figure 30), have higher precision and can be used for ductile frames, because computer software cannot do the displacement-based shear for pushover analysis. Therefore, displacement-force method should be done even if it has lower precision. The computer software will adjust displacement increments, which represent only 2% of displacement procedure, in order to decrease the variation of the pushover curve in the structure response using pushover analysis. In order to get the nonlinear behaviour of the structure, virtual pushover curve should be defined which know as idealization curve Kim and Choi (2005). This curve contains the displacement on X-axis due to the base shear that will be on Y-axis.

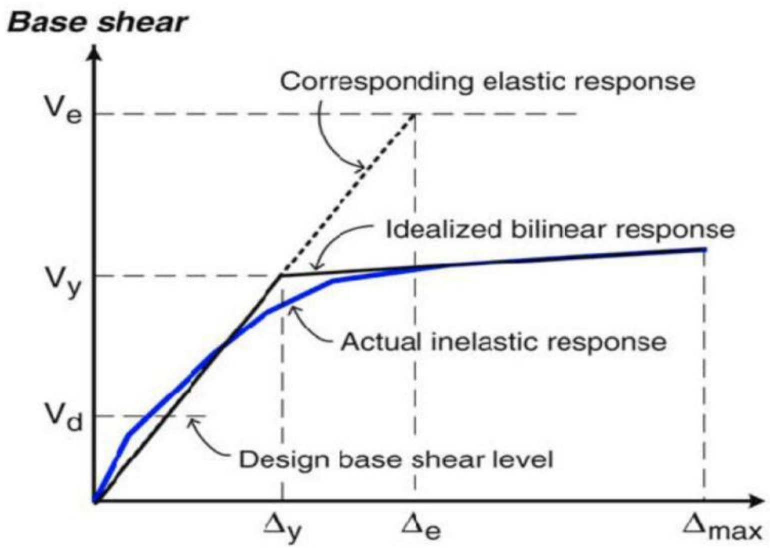

Figure 30: Displacement Modification Curve. Eurocode 8, BS EN 1998-1 2004.

3.7.2.4 Plastic Hinges

The plastic hinge in a column is made on interaction of weak points (M3), Strong moment (M2) and on the interaction of axial force (P) in direction of the section. Plastic hinges for column was applied on the ending and beginning points taking into account P-M2-M3 interaction. One or two plastic hinges will occur in V or inverted V bracing frame in the shear link, which the brace members are connected. The hinges are factors depending on the eccentricity ratio of the bracing system. In this study, plastic hinges of shear load in beams and bracing members are modeled with in 0.5 at the place of connection between beam and bracing members. Usually in concentric braced frames, beam to column connections are not pinned and resist a little amount of moment. Therefore, for this study, the beam to column connection assumed to be pinned.

3.7.3 Response Spectrum Analysis

3.7.3.1 Assessment and Procedure of Response Spectrum

Dynamic procedure is required for non-orthogonal structure or torsional irregularities for the tall buildings. The building should be modeled first with linearly elastic stiffness matrix and an equivalent viscous damping matrix, at or near yield level as a multi degree of freedom. Response Spectrum analysis method of structure will consider the response of multiple modes in the frequency domain.

3.7.3.2 Computer Analysis Method of Response Spectrum

Computer analysis for response spectrum analysis will determine the modes for a structure. The response of structure can be read from the design spectrum that depends on the modal frequency and the modal mass. To check the effects of lateral loads on a structure, the forces in all directions X, Y & Z should be obtained. The methods of analysis response spectrum analysis are:

- Complete quadratic combination (CQC) - a method that is an improvement on SRSS for closely spaced modes.
- Absolute - peak values re-added together.
- Square root of the summation squares (SRSS).

In the study CQC method will be used as modal combination method, while SRSS method will be used for directional combination type during the software analysis of response spectrum.

3.7.3.3 Accidental Torsion in Response Spectrum Analysis

Accidental torsion in response spectrum analysis will be considered without rigid diaphragms. The torsional moment will be calculated by first resolving the total lateral force at joint locations within a flexible diaphragm in a given direction at each story level multiplied by the eccentricity to generate torsional moment (T), and it can be done summing the auto-seismic loads in the given storey.

3.7.3.4 Modal case Ritz vs. Eigen vectors

Eigen vector method it is a suitable method that determines frequencies of the system, the undamped free vibration mode shapes and the response of structure from the horizontal ground acceleration. This method is a good method for checking the behaviour of the structure as well as it locates the problems within the model.

Ritz-vector method is involving nonlinear FNA method, ground acceleration and localized machine vibration. It is an efficient method and has been used for the dynamic analysis from long time taken into consideration horizontal ground motion. Ritz-vector method used to find the predicted particular loading modes. It can provide better participation factor that can make the analysis faster with the same level of accuracy. In this study, Eigen-vector method will be used in this study in order to have better results for torsional accidental.

3.8 Design Software

In this study, ETABS 13.2.2 will be used for analysis since it has widely used especially for steel structure analysis amongst civil engineers. ETABS consider as powerful software for the linear dynamic analysis as well as it is easy to use and to get an understandable results.

Chapter 4

RESULTS AND DISCUSSION

After modeling the structure for H-plan and square plan, 4 and 12 stories, the structures will be designed first with considering linear static analysis with considering other loads (Figure31). After that, all loads will be un-run and the pushover (Figure32) and response spectrum (Figure33) analysis loads will be applied individually on both direction X and Y, with considering dead and live load only. The results of both analysis pushover and response spectrum will be given in order to discuss the results and make the comparison between different the bracing systems. This discussion will aim to compare the performances of structure using four types of bracing frames in order to identify the best bracing system for different number of floors and different types of analysis.

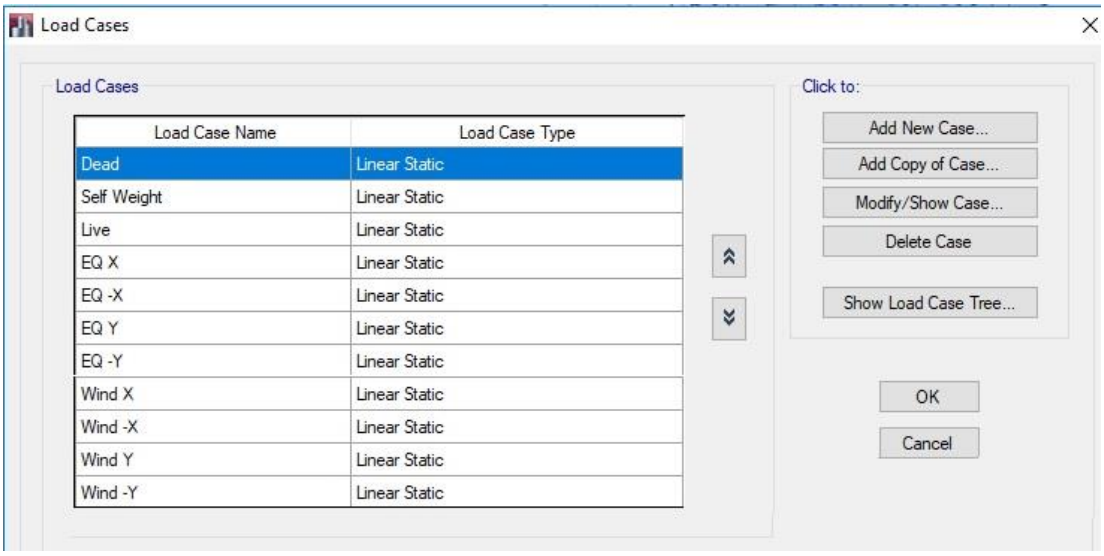

Figure 31: Load Cases During Linear Static Analysis

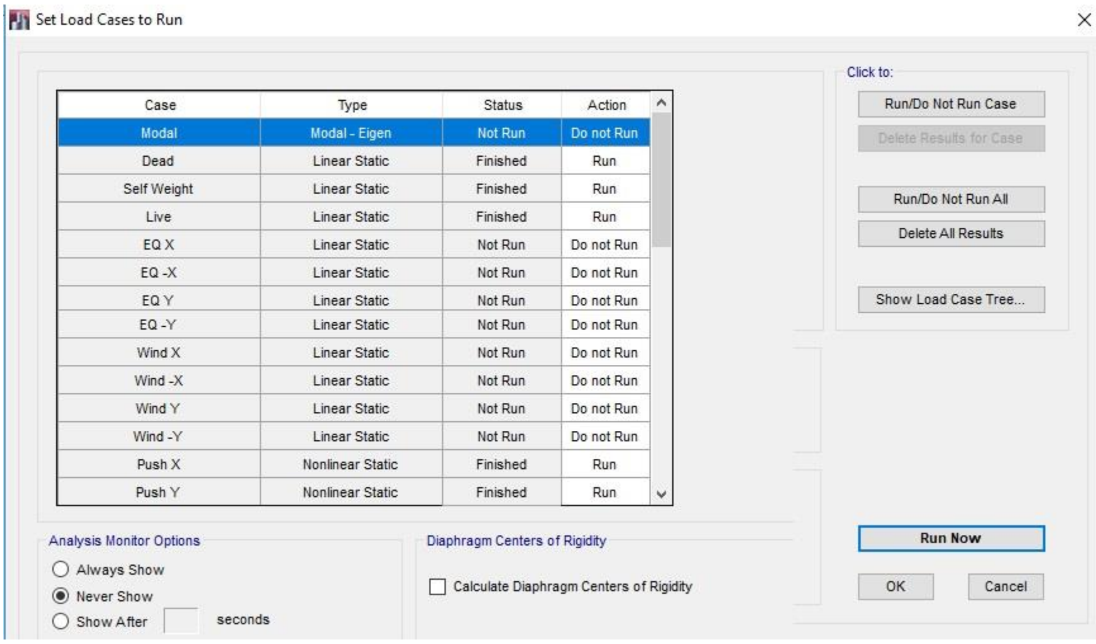

Figure 32: Load Cases During Pushover Analysis

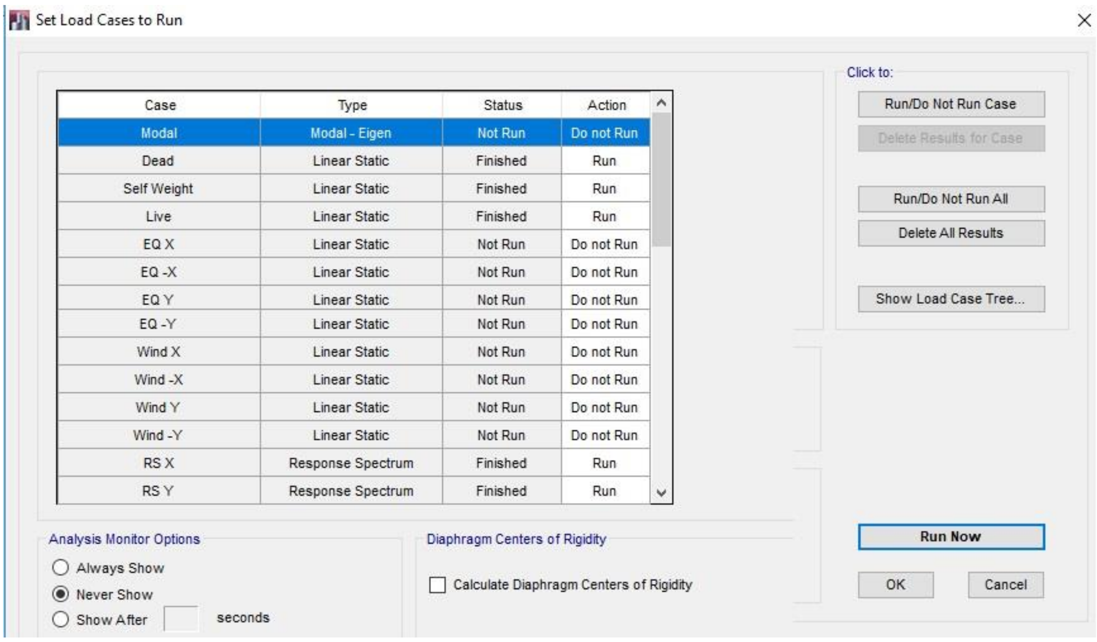

Figure 33: Load Cases During Response Spectrum Analysis

The discussion of results using pushover analysis will be presented in section 4.1. Then, results of response spectrum analysis will be discussed in section 4.2. Finally, a comparison between both analyses in order to compare the effects of these analyses on the structure will be presented in section 4.3.

4.1 Discussion of Pushover Analysis

Comparison between the selected types of bracing systems using pushover analysis will be through two methods. These method will depends on the performance of braced frames and the structures in order to identify the best bracing system for 4- and 12 story structure, and H and square plan.

The sections that have been used for pushover and response spectrum analysis will be same for different bracing systems, different plan shape and different number of stories. In order to obtain these sections, the structure will be modeled first with liner static analysis, and after checking all sections are adequate, response spectrum and pushover analysis will be applied. For H-plan, four sections on x-axis and four for y-axis will be listed, while two sections for square plan will be listed on x-axis and y-axis, because the plans are symmetric. All the sections that used in this study will be listed in the appendix A.

During the analysis, the effect of applied pushover on a direction will lead to displacement and occurrence of plastic hinges on the same direction with a small displacement for the other direction and to occurrence of immediate occupancy hinges in some cases.

4.2.1 Plastic Hinges

The occurrence of plastic hinges in the structure using pushover analysis will varies according to the number of stories, type of bracing system and used plan shape. As mentioned in section 2.3.2, the plastic hinges will reach different cases according to the change that happened in the bracing members because of the failure moment under earthquake motion.

In this study, plastic hinges will be under three properties, IO (immediate occupancy), LS (life safety), CP (collapse prevention), A, B, C, D and E according to the condition of these hinges (Figure34). In this section, the comparison between the same number of stories and plan shape will be discussed through pushover on analysis on X axis then Y axis.

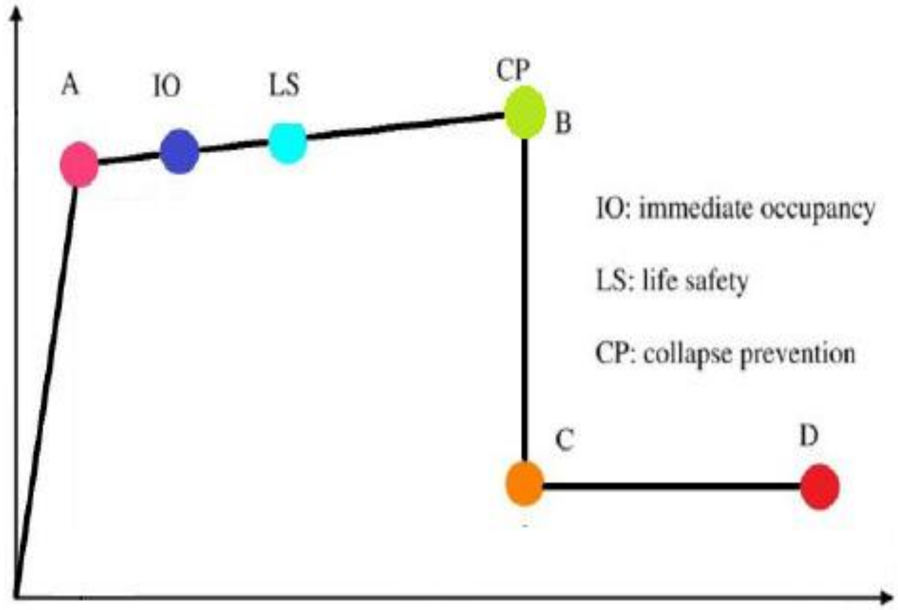

Figure 34: Expected Types Plastic hinges that occur using ETABS. Eurocode 8, BS EN 1998-1 2004.

The plastic hinge occurred in X and Y axis during pushover on X axis will be different than the plastic hinges that occurred on X and Y axis during pushover on Y axis because the distribution of bracing members is not same for both axis, as well as there are differences in the number of modes for the same axis between each of the bracing systems and even for the same bracing system for each axis. After analyzing the 4-story H plan with different types of concentric braced frames, plastic hinges occurs in various members with different occurrence of plastic hinges.

The plastic hinges that occurred in the structure due to pushover analysis in X axis then in Y axis. Pushover applied either on X or Y axis in some cases effects on the other axis with occurrence of plastic hinges (Figure 129 and 130) with story displacement. Following figures will be an example on some plastic hinges that formed in the modeled structure.

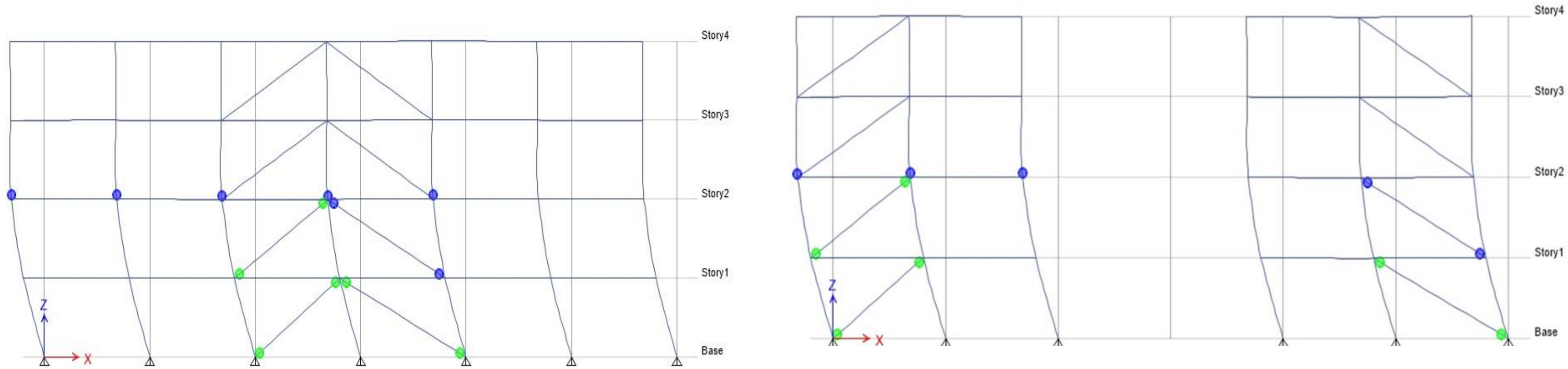

Figure 35: Plastic Hinges of 4-Story H Plan Diagonal Concentric Braced Frame for Pushover on X-axis

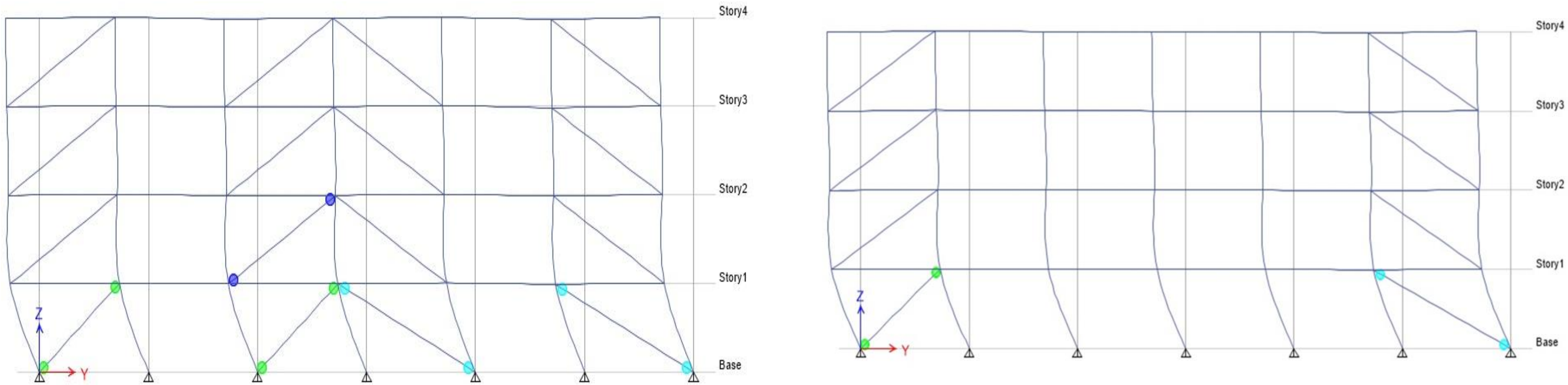

Figure 36: Plastic Hinges of 4-Story H Plan Diagonal Concentric Braced Frame for Pushover on Y-axis

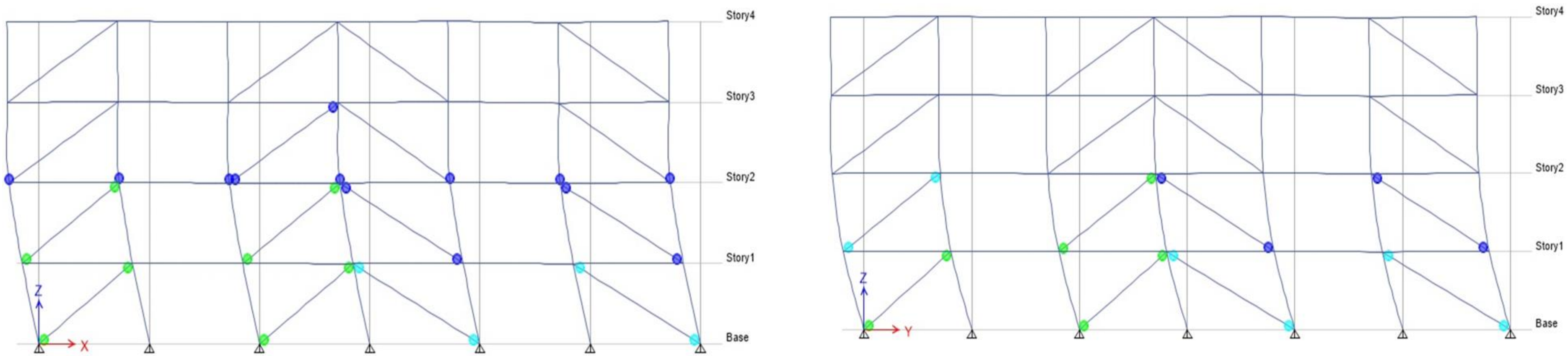

Figure 37: Plastic Hinges of 4-Story Square Plan Diagonal Concentric Braced Frame for Pushover on X and Y axis

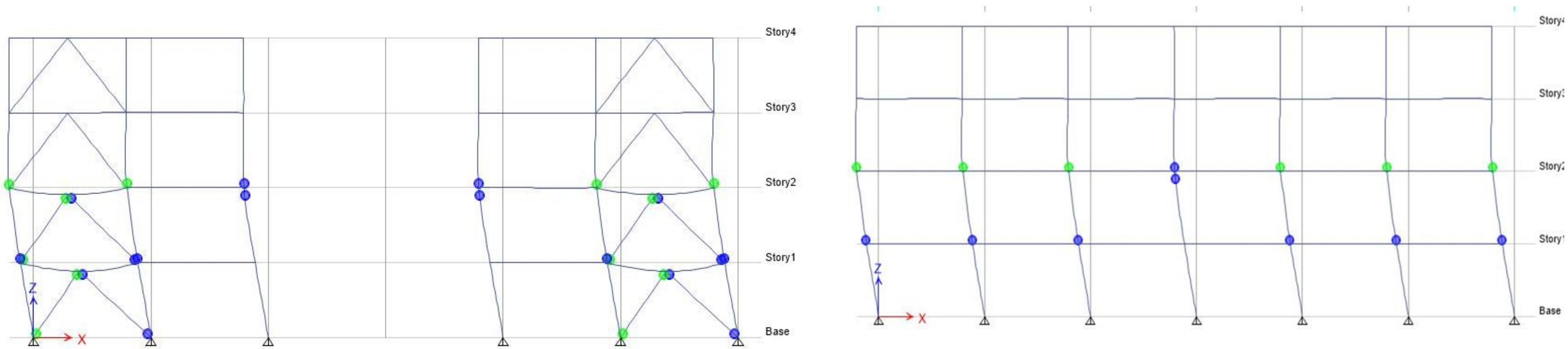

Figure 38: Plastic Hinges of 4-Story H Plan Concentric Inverted V Braced Frame for Pushover on X-axis

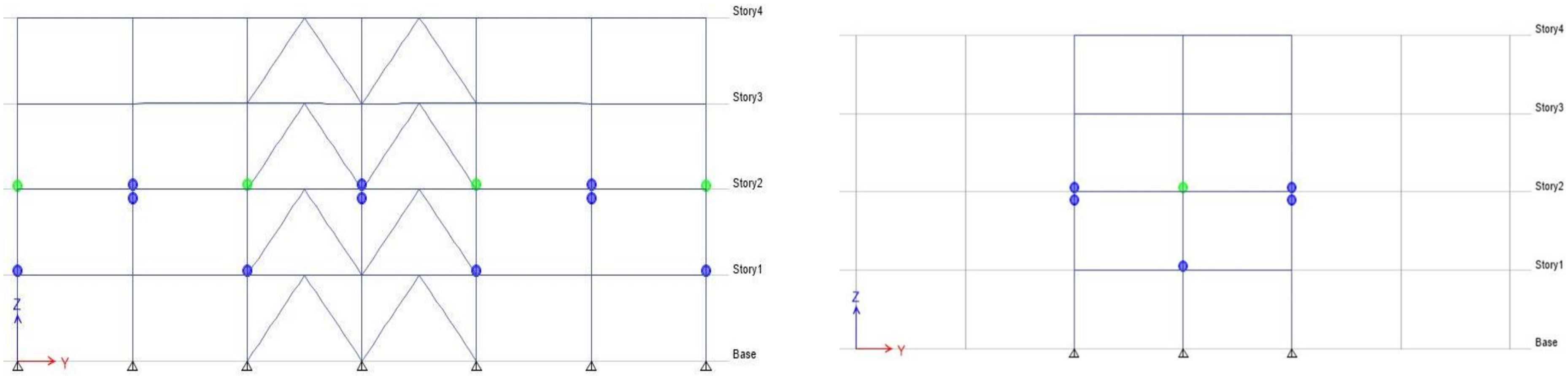

Figure 39: Plastic Hinges on of 4-Story H Plan Concentric Inverted V Braced Frame for on Y-axis During Pushover on X-axis

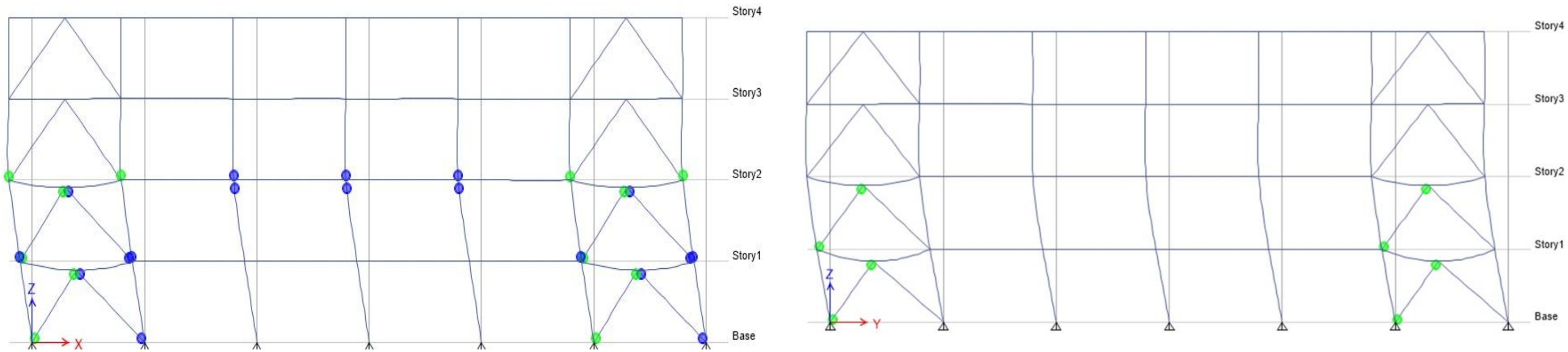

Figure 40: Plastic Hinges of 4-Story Square Plan Concentric Inverted V Braced Frame for Pushover on X and Y axis

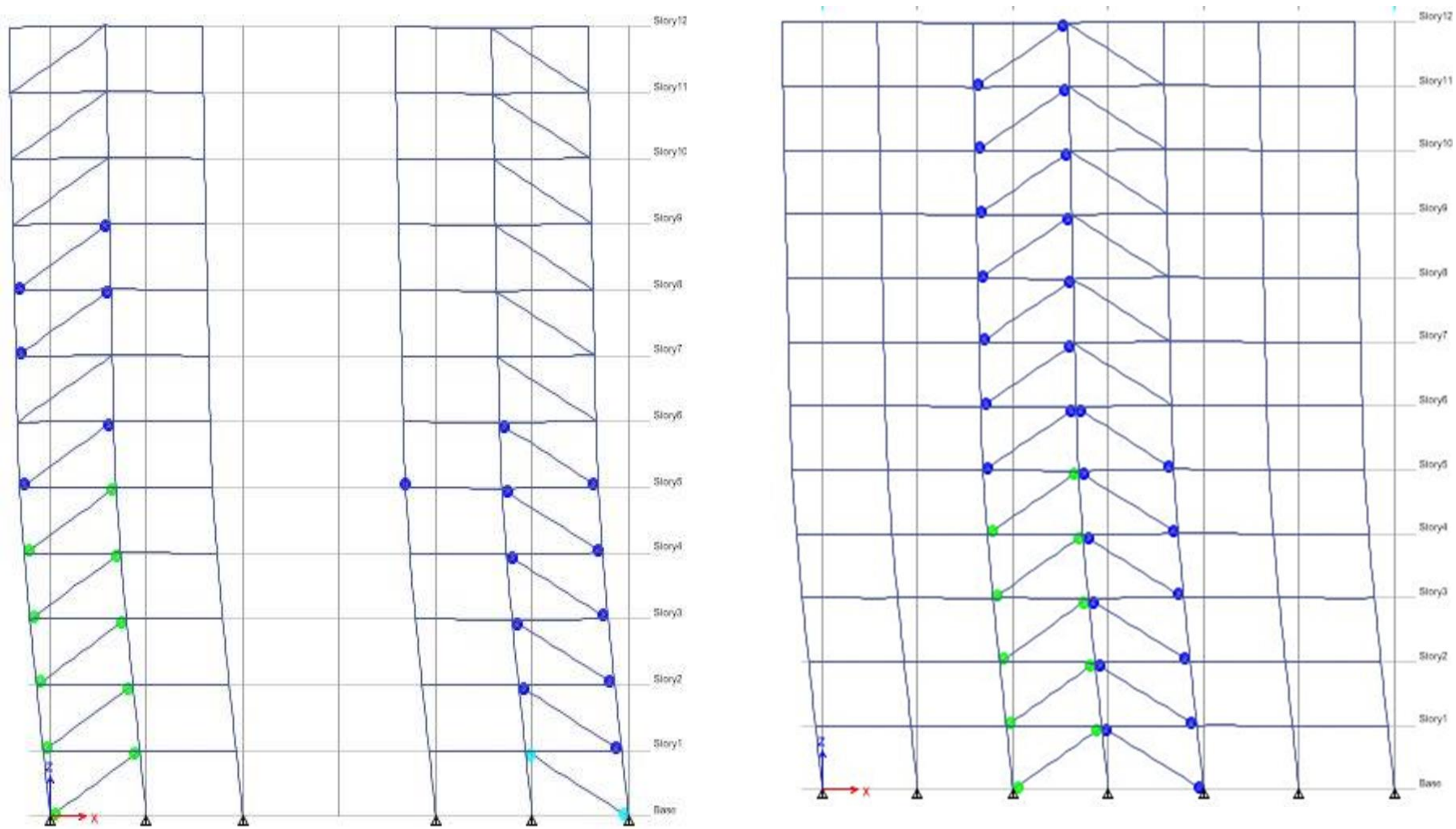

Figure 41: Plastic Hinges of 12-Story H Plan Diagonal Concentric Braced Frame for Pushover on X-axis

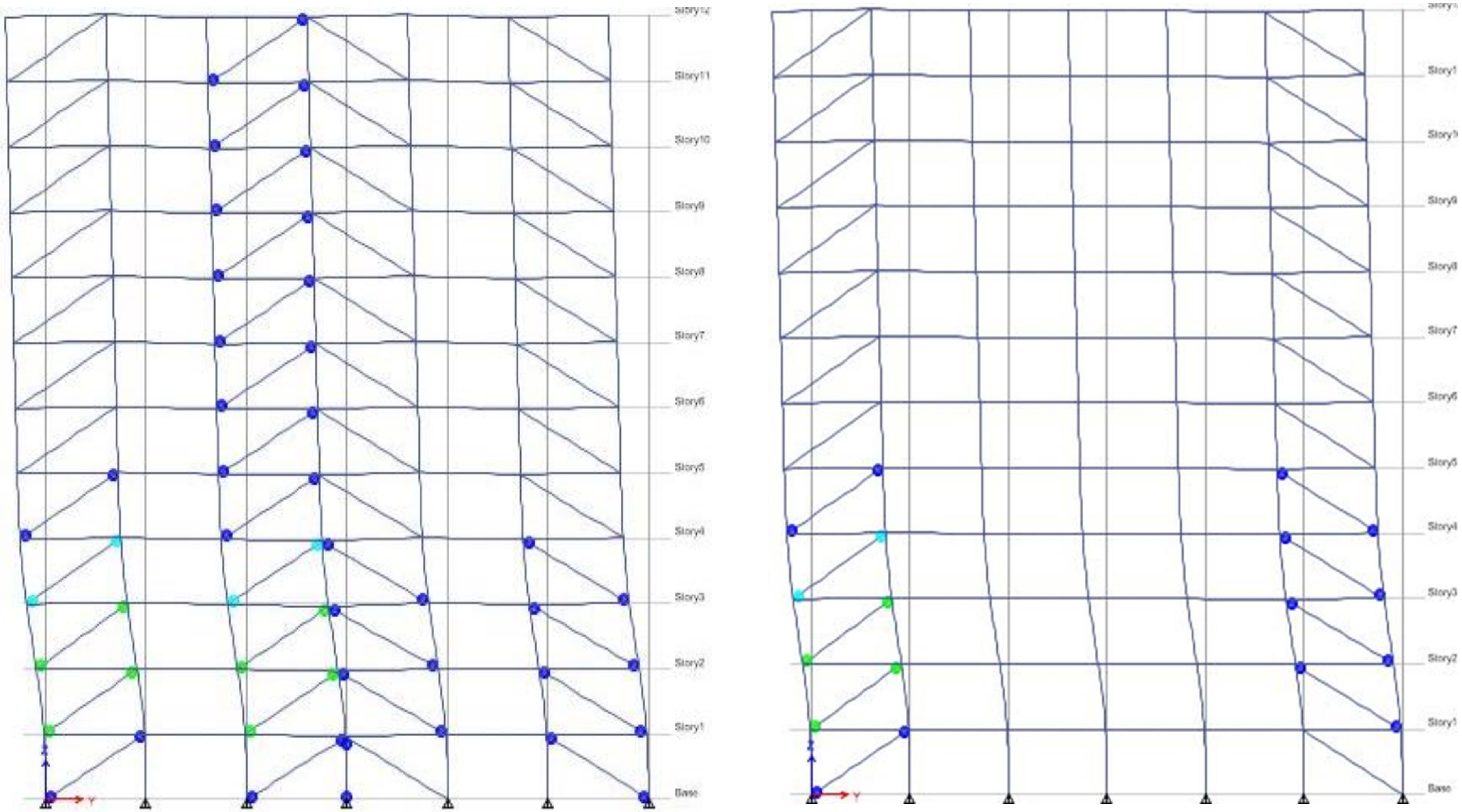

Figure 42: Plastic Hinges of 12-Story H Plan Diagonal Concentric Braced Frame for Pushover on Y-axis

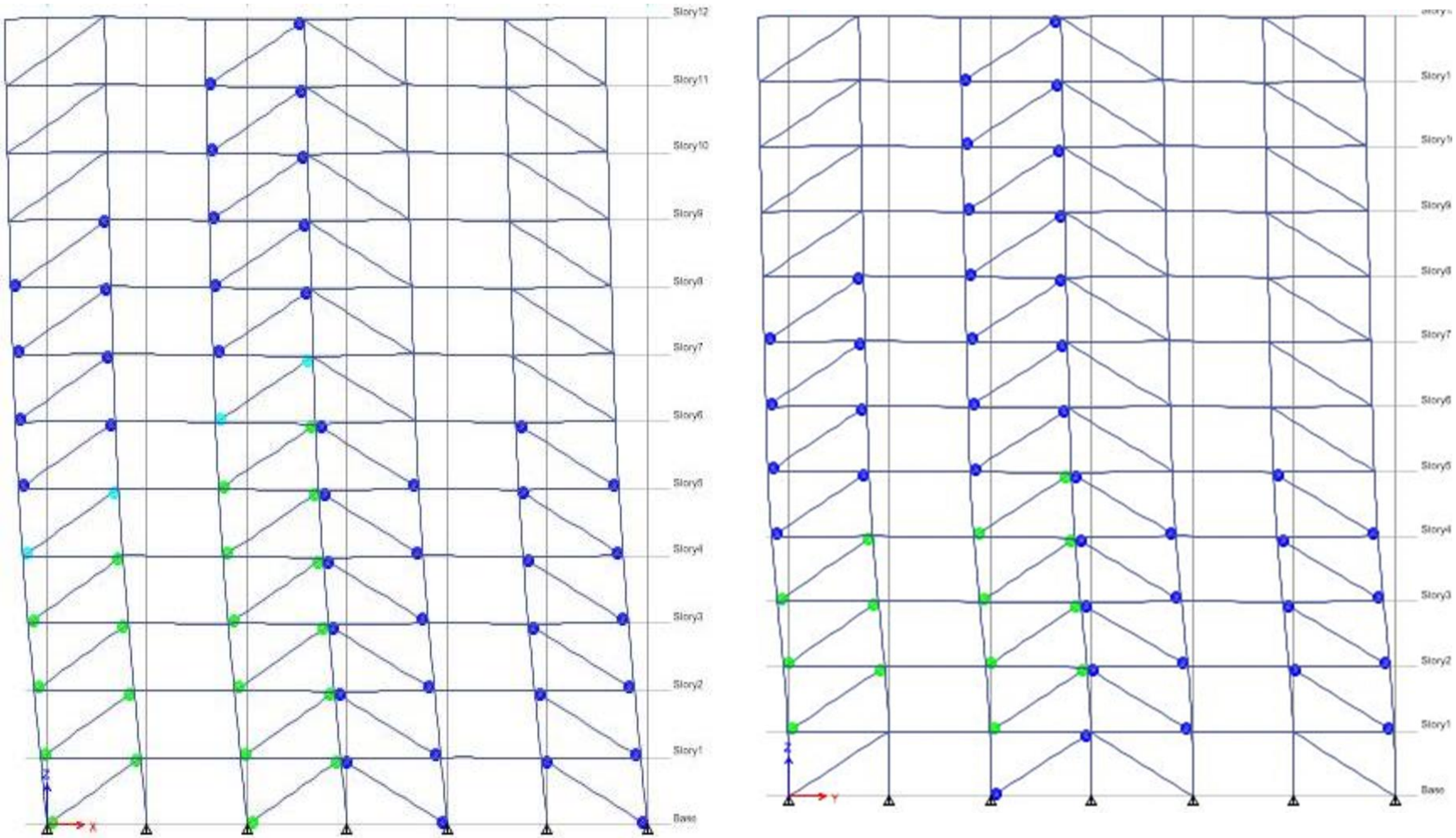

Figure 43: Plastic Hinges of 12-Story Square Plan Diagonal Concentric Braced Frame for Pushover on X-axis

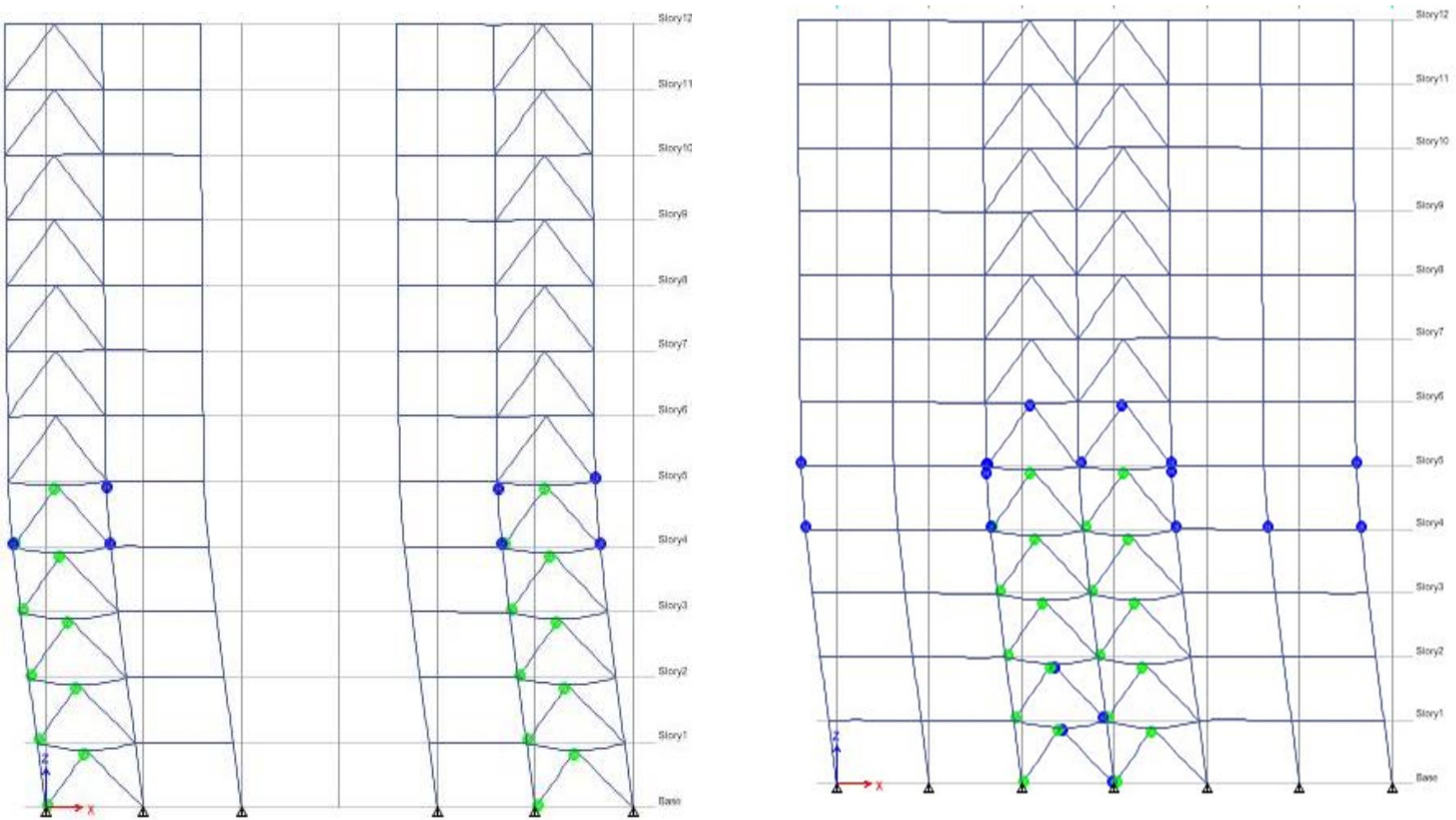

Figure 44: Plastic Hinges of 12-Story H Plan Concentric Inverted V Braced Frame for Pushover on X-axis

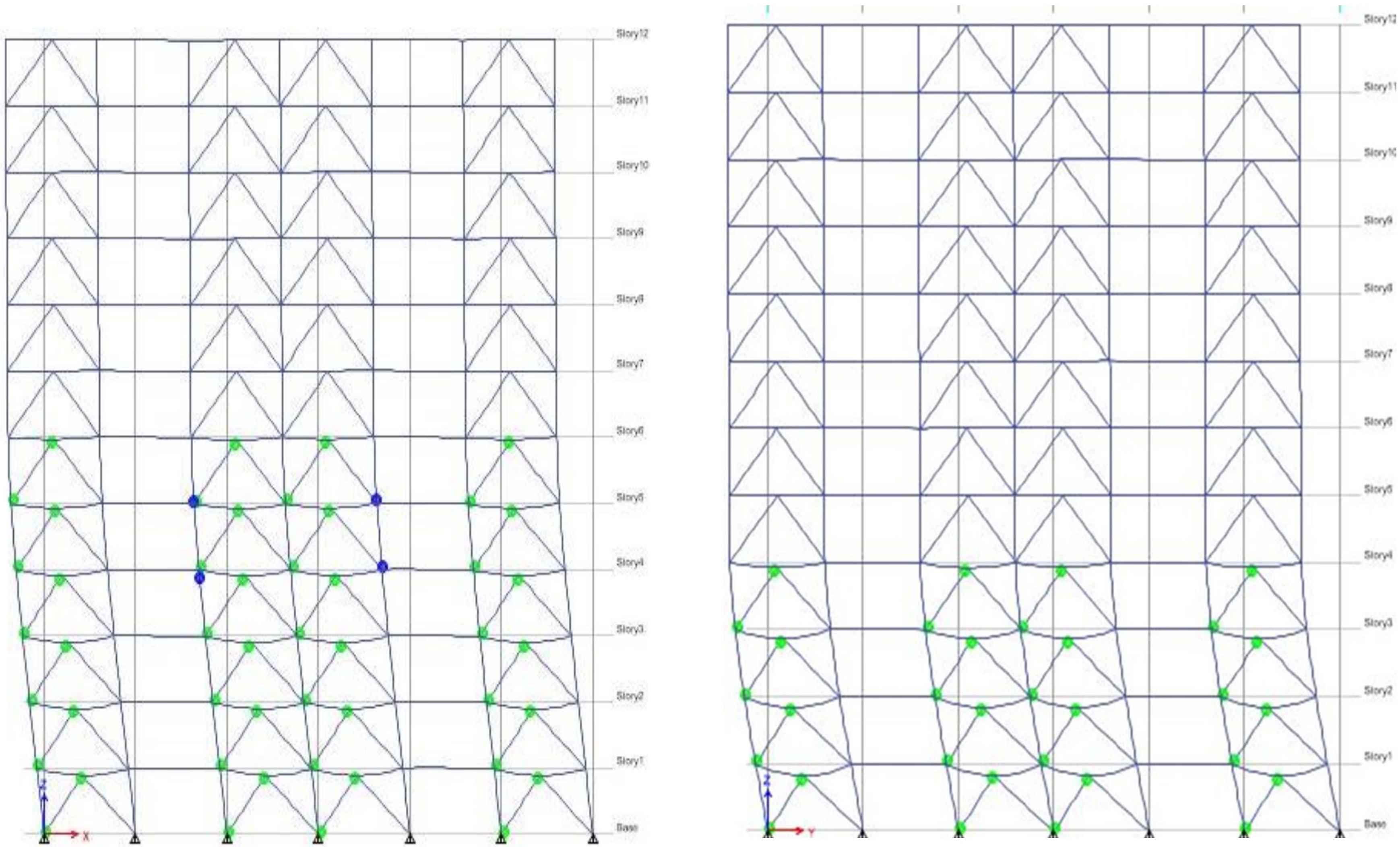

Figure 45: Plastic Hinges of 12-Story Square Plan Concentric Inverted V Braced Frame for Pushover on X and Y axis

4.1.1.1 Discussion of Results of 4-Story H Plan for All Braced Frames

The analysis of concentric diagonal braced frames for 4-story H plan shows that, the plastic hinges starts in the bracing members and it were in immediate occupancy condition with no effects on the columns or beams. But on the step number 12 or X direction and step number 7 on Y direction, which are the last steps, the hinges collapsed (>CP) and immediate occupancy hinges formed in the external columns as shown in figures mentioned in chapter 4 for X direction and figures 123 and 124 for Y direction.

For inverted chevron V concentric braced frames, the plastic hinges for Y direction effect on the same direction with occurrence of plastic hinges and story displacement, and in last step number 19, 45 hinges collapsed out of 1288 hinges in the structure. While on X direction, plastic hinges started to be form on step number 5 on the bracing members. In the pushover on X direction, plastic hinges started to occur also on Y direction, and on step number 17 these plastic hinges on Y direction due to pushover on X direction collapsed. The total number of collapsed hinges due to pushover on X direction were 45 hinges.

During the analysis of diagonal eccentric braced frame, the hinges within the structure was without any effects in the first two steps for X and Y direction out of 5 and 6 steps respectively. A very small number of hinges collapsed out of the total hinges during the applied pushover on both directions with formed of immediate occupancy hinges.

Analysis of structure using eccentric inverted chevron V was good with applied pushover on Y direction, because 16 hinges collapsed out of 1288, 1264 hinges were between A-IO and 8 hinges between LS-CP . But in applied pushover on X axis, 61 hinges collapsed, 17 hinges in region between IO-LS and 55 hinges between LS-CP, as well as the applied pushover on X direction effect the other direction and formed plastic hinges in columns and bracing members . Tables 10 and 11 below will describe the effects of pushover analysis on 4-story H plan for all selected braced frames.

Table 10: Plastic Hinges for 4-Story H plan on During Pushover on X-axis

Type of Bracing system	Step	A-B	B-C	C-D	D-E	>E	A-IO	IO-LS	LS-CP	>CP	Total Hinges
Concentric Diagonal	12	1210	86	16	0	8	1210	49	37	24	1320
Concentric Inverted Chevron V	17	1133	110	29	0	16	1133	33	77	45	1288
Eccentric Diagonal	5	1200	10	14	0	0	1200	2	8	14	1224
Eccentric Inverted Chevron V	17	1155	76	41	0	16	1155	17	55	61	1288

Table 11: Plastic Hinges for 4-Story H plan on During Pushover on Y-axis

Type of Bracing system	Step	A-B	B-C	C-D	D-E	>E	A-IO	IO-LS	LS-CP	>CP	Total Hinges
Concentric Diagonal	7	1280	20	8	0	12	1280	12	0	28	1320
Concentric Inverted Chevron V	19	1268	0	0	0	20	1268	0	0	20	1288
Eccentric Diagonal	6	1212	2	6	0	4	1212	0	2	10	1224
Eccentric Inverted Chevron V	17	1264	8	0	0	16	1264	0	8	16	1288

For 4-story H-plan as shown in tables 10 and 11, the diagonal eccentric braced frame was the system with the lowest number of collapsed hinges for both axis X-axis (14 collapsed hinges) and Y-axis (10 collapsed hinges), as well as for other conditions of hinges wherever it were between immediate occupancy and life safety or life safety and collapse prevention.

4.1.1.2 Discussion of Results of 4-Story Square Plan for All Braced Frames

For diagonal concentric braced frames, collapsed hinges formed in columns and bracing members at the base story with 28 collapsed hinges on X direction, and 24 collapsed hinges on Y direction.

After 18 steps, some plastic hinges for concentric inverted V braced frame for pushover on X direction, reached collapse prevention region in the bracing members and columns and remain in immediate occupancy condition for the other columns in the external section, while all hinges in the third story columns for the internal sections reached the collapse prevention region, and corner columns of the second story remain in the immediate occupancy region. For pushover in Y direction, only the first two story bracing members reached the collapse prevention.

Analysis of eccentric diagonal braced frame due to pushover on X and Y shows that the hinges in bracing members only has been reached the collapse level in X direction and immediate occupancy level in Y direction.

In inverted V braced frame, hinges of Y direction has been affected by applied pushover on X direction and reached IO and LS levels. Tables 12 and 13 show the steps and hinges conditions for all braced frames.

Table 12: Plastic Hinges for 4-Story Square plan on During Pushover on X-axis

Type of Bracing system	Step	A-B	B-C	C-D	D-E	>E	A-IO	IO-LS	LS-CP	>CP	Total Hinges
Concentric Diagonal	13	1387	65	12	4	12	1387	8	57	28	1480
Concentric Inverted Chevron V	18	1355	72	37	0	16	1355	20	52	53	1480
Eccentric Diagonal	12	1363	31	14	0	8	1363	25	6	22	1416
Eccentric Inverted Chevron V	15	1336	116	12	0	16	1336	12	104	28	1480

Table 13: Plastic Hinges for 4-Story Square plan on During Pushover on Y-axis

Type of Bracing system	Step	A-B	B-C	C-D	D-E	>E	A-IO	IO-LS	LS-CP	>CP	Total Hinges
Concentric Diagonal	7	1444	12	16	0	8	1444	4	8	24	1480
Concentric Inverted Chevron V	17	1464	0	0	0	16	1464	0	0	16	1480
Eccentric Diagonal	2	1410	0	6	0	0	1410	0	0	6	1416
Eccentric Inverted Chevron V	16	1456	8	0	0	16	1456	0	8	16	1480

For 4-story square plan as shown in tables 12 and 13, the diagonal eccentric braced frame was the system with the best system between all braced frame that hinges in this system behave better due to collapse with lowest number of collapsed hinges for both axis X-axis (28 collapsed hinges) and Y-axis (6 collapsed hinges).

4.1.1.3 Discussion of Results of 12-Story H and Square Plan for All Braced Frames

Tables 14, 15, 16 and 17 describe the conditions of hinges that applied for 12-story H and square plans for all bracing systems in order to make a comparison between these braced frames for applied pushover on both X and Y directions. The occurred plastic hinges during the analysis with its different conditions and locations.

Table 14: Plastic Hinges for 12-Story H plan on During Pushover on X-axis

Type of Bracing system	Step	A-B	B-C	C-D	D-E	>E	A-IO	IO-LS	LS-CP	>CP	Total Hinges
Concentric Diagonal	29	3801	81	50	0	28	3801	31	46	76	3960
Concentric Inverted Chevron V	70	4214	138	8	0	80	4214	82	56	88	4440
Eccentric Diagonal	21	3846	48	46	0	20	3846	4	44	66	3960
Eccentric Inverted Chevron V	63	4303	41	16	0	80	4303	33	8	96	4440

Table 15: Plastic Hinges for 12-Story H plan on During Pushover on Y-axis

Type of Bracing system	Step	A-B	B-C	C-D	D-E	>E	A-IO	IO-LS	LS-CP	>CP	Total Hinges
Concentric Diagonal	18	3812	64	72	0	12	3812	24	40	84	3960
Concentric Inverted Chevron V	23	4332	42	18	0	48	4332	42	0	66	4440
Eccentric Diagonal	4	3888	39	33	0	0	3888	28	11	33	3960
Eccentric Inverted Chevron V	5	4406	12	22	0	0	4406	12	0	22	4440

Table 16: Plastic Hinges for 12-Story Square plan on During Pushover on X-axis

Type of Bracing system	Step	A-B	B-C	C-D	D-E	>E	A-IO	IO-LS	LS-CP	>CP	Total Hinges
Concentric Diagonal	25	4300	56	60	0	24	4300	8	48	84	4440
Concentric Inverted Chevron V	60	4667	61	12	0	84	4667	46	15	96	4824
Eccentric Diagonal	13	4308	64	64	4	0	4308	10	54	68	4440
Eccentric Inverted Chevron V	67	4660	68	0	0	96	4660	54	14	96	4824

Table 17: Plastic Hinges for 12-Story Square plan on During Pushover on Y-axis

Type of Bracing system	Step	A-B	B-C	C-D	D-E	>E	A-IO	IO-LS	LS-CP	>CP	Total Hinges
Concentric Diagonal	16	4312	48	68	0	12	4312	16	32	80	4440
Concentric Inverted Chevron V	33	4760	0	0	0	64	4760	0	0	64	4824
Eccentric Diagonal	3	4414	20	6	0	0	4414	6	14	6	4440
Eccentric Inverted Chevron V	26	4760	0	0	0	64	4760	0	0	64	4824

For both H and square plan, eccentric diagonal braced frames are the best tested and analyzed system with the lowest number of collapse plastic hinges for X and Y axis. For H plan, 66 hinges on X axis and 33 on Y axis out of 3960 reached the collapse

(>CP) region, while for square plan 68 hinges on X axis and 6 on Y axis out of 4440 reached the collapse (>CP).

4.2.2 Modification and Capacity-Demand Curves

Capacity-demand curve is represented by the spectral acceleration and spectral displacement. This curve will identify the target displacement performance point. At this point hinges were in collapse prevention level and the overall performance of the structure will be in the collapse level. While as mentioned in Section 3.7.2.3, the modification curves shows the displacement due to the base shear. The values of these displacements used in order to set at which step the performance point will occur according to the value of target displacement.

Equivalent linearization or idealization curve as mentioned in section 3.7.2.3will be listed below as well as modification curve for various models. These two curves equivalent linearization curve and modification curve will be obtained from ETABS according to ASCE and FEMA 440 respectively for both axis X and Y.

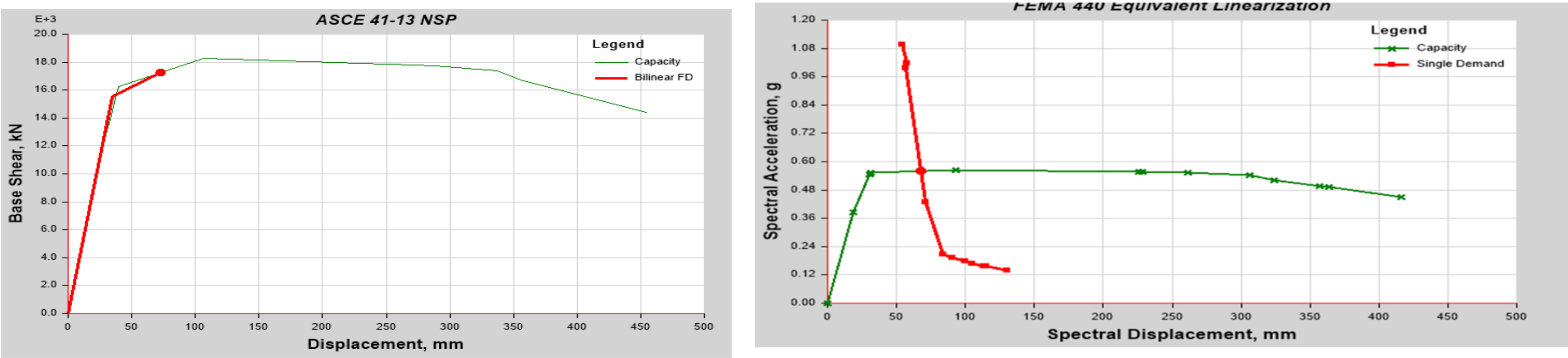

Figure 46: Modification and Demand-Capacity Curve for 4-Story Square Plan Concentric Diagonal Braced Frame for on X-axis

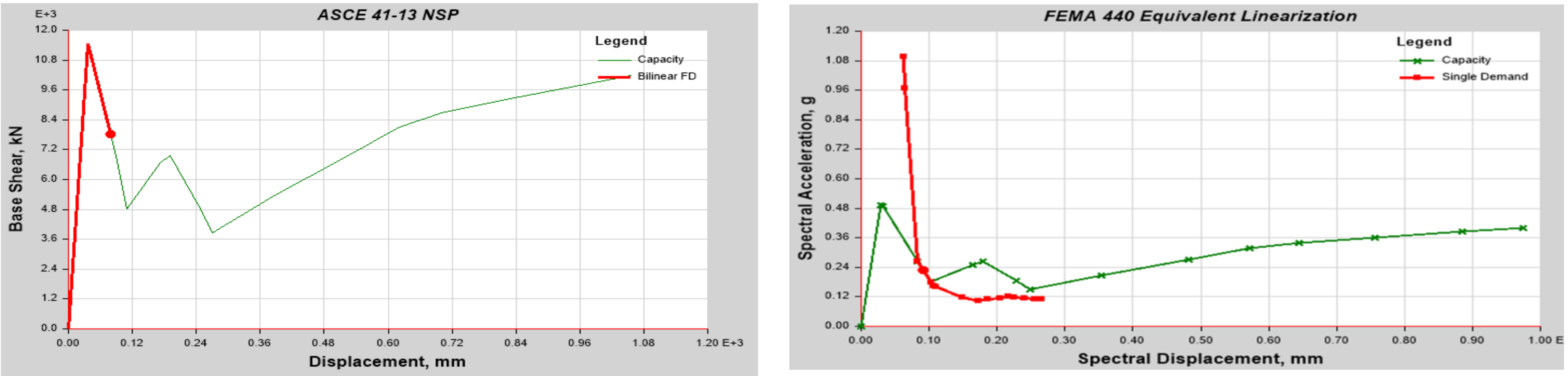

Figure 47: Modification Curve and Capacity-Demand for 4-Story H Plan Concentric Inverted V Braced Frame for on X-axis

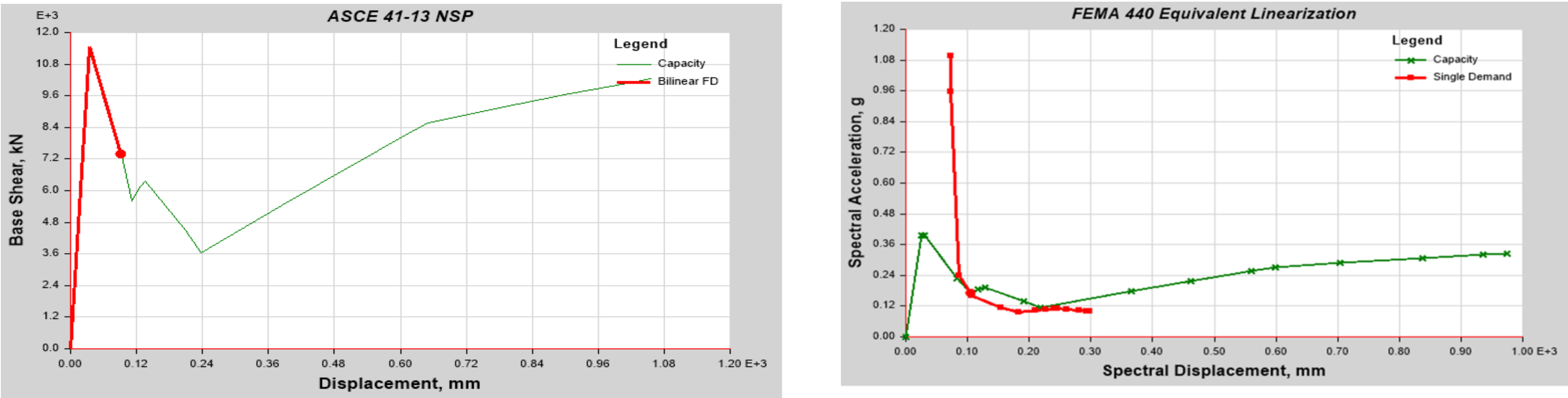

Figure 48: Modification Curve and Capacity-Demand for 4-Story Square Plan Concentric Inverted V Braced Frame for on X-axis

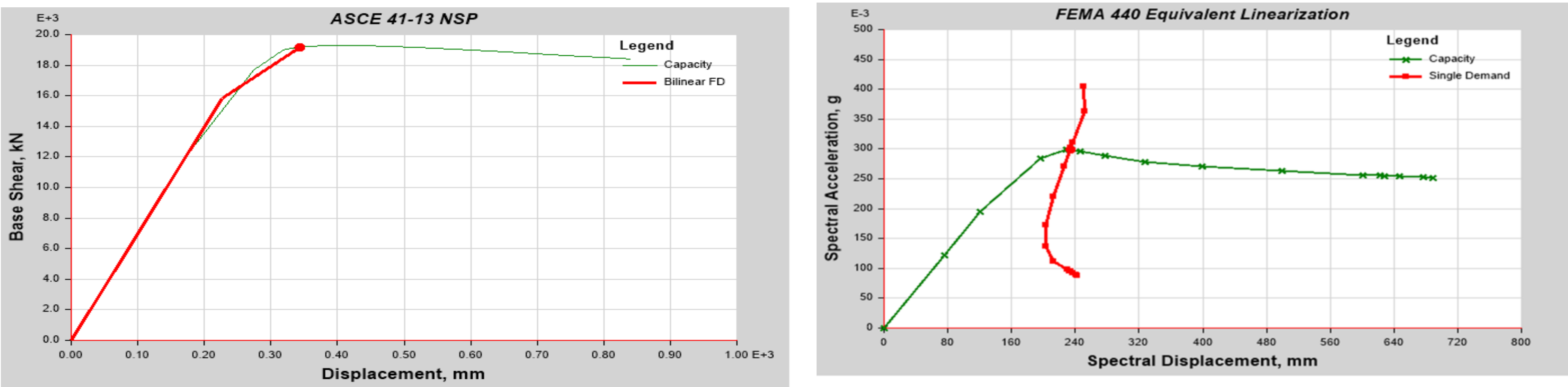

Figure 49: Modification and Demand-Capacity Curve for 12-Story H Plan Eccentric Diagonal Braced Frame for on X-axis

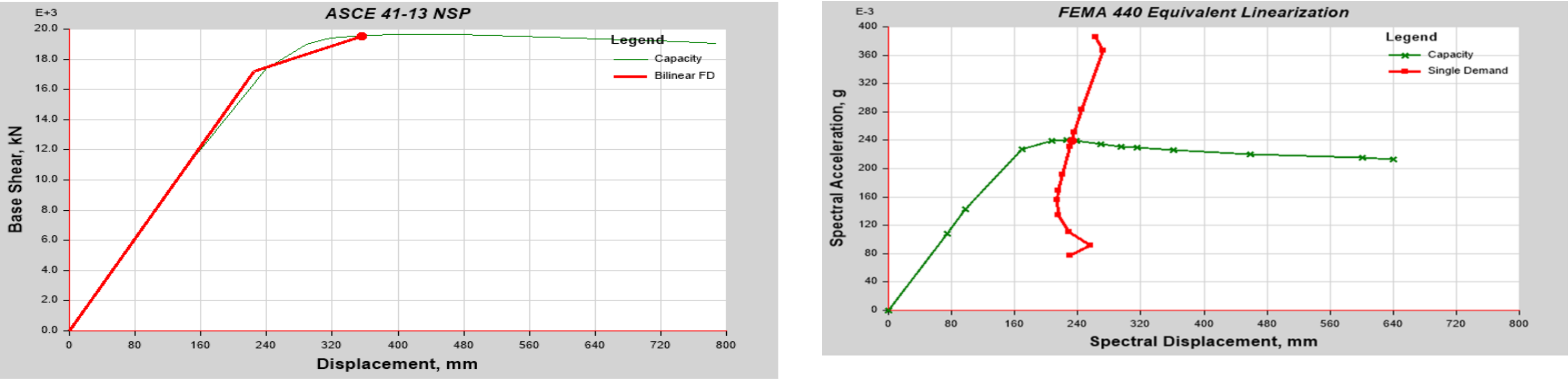

Figure 50: Modification and Demand-Capacity Curve for 12-Story Square Plan Eccentric Diagonal Braced Frame for on X-axis

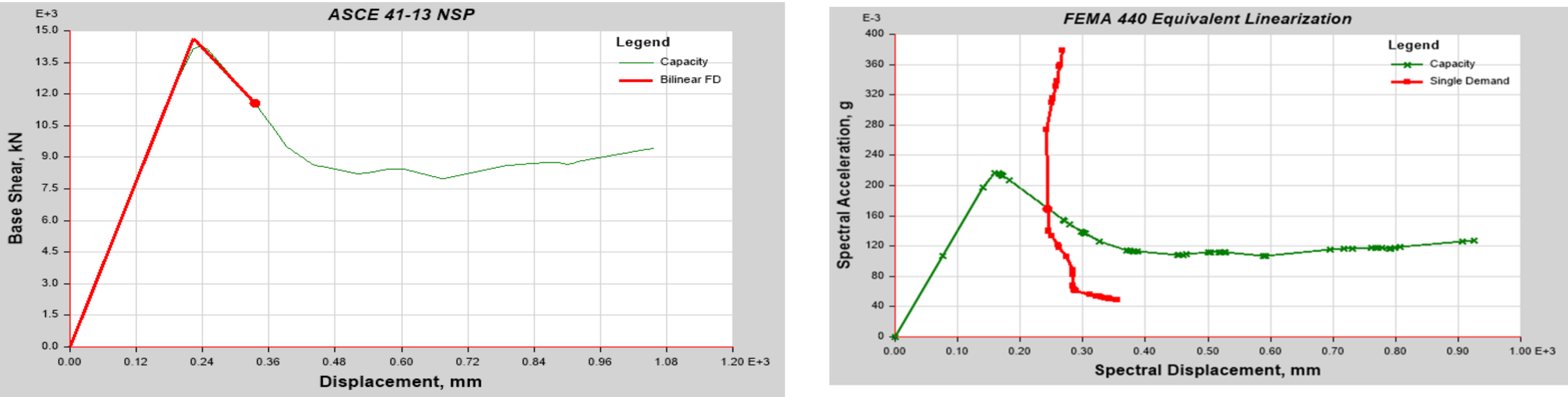

Figure 51: Modification and Demand-Capacity Curve for 12-Story H Plan Eccentric Inverted V Braced Frame for on X-axis

Performance point is intersection between the capacity curves with demand curves in as mentioned in the previous figures.

Table 18: Performance Point for 4-Story H Plan on X-axis

Type of Braced Frame	Performance Point Occurrence	Target Displacement (mm)	Shear (Kn)	Step Number	Monitored Displacem-ent (mm)	A-IO	IO-LS	LS-CP	>CP
Concentric Diagonal	YES	76.8	12963.4	4	88.3	1288	8	8	8
Concentric Inverted Chevron V	YES	98.4	676.2	5	110.2	1268	6	6	8
Eccentric Diagonal	YES	132.2	10680.6	5	168.7	1200	2	8	8
Eccentric Inverted Chevron V	YES	145.6	4222.8	6	196.1	1224	17	31	8

Table 19: Performance Point for 4-Story H Plan on Y-axis

Type of Braced Frame	Performance Point Occurrence	Target Displacement (mm)	Shear (Kn)	Step Number	Monitored Displacem-ent (mm)	A-IO	IO-LS	LS-CP	>CP
Concentric Diagonal	YES	57.6	18042.9	4	151.1	1280	16	12	12
Concentric Inverted Chevron V	YES	151.7	3355.5	6	204.3	1280	0	0	8
Eccentric Diagonal	YES	257.2	2788.5	5	300.4	1212	4	4	4
Eccentric Inverted Chevron V	YES	188.9	2867.1	6	222.9	1280	0	0	8

Table 20: Performance Point for 4-Story Square Plan on X-axis

Type of Braced Frame	Performance Point Occurrence	Target Displacement (mm)	Shear (Kn)	Step Number	Monitored Displacement (mm)	A-IO	IO-LS	LS-CP	>CP
Concentric Diagonal	YES	37.7	16945	2	43.9	1476	4	0	4
Concentric Inverted Chevron V	YES	112.8	5626.6	6	125.3	1452	8	12	8
Eccentric Diagonal	YES	140.1	10842	4	191.4	1390	4	10	12
Eccentric Inverted Chevron V	YES	162.1	4388.7	6	212.9	1415	12	37	16

Table 21: Performance Point for 4-Story Square Plan on Y-axis

Type of Braced Frame	Performance Point Occurrence	Target Displacement (mm)	Shear (Kn)	Step Number	Monitored Displacement (mm)	A-IO	IO-LS	LS-CP	>CP
Concentric Diagonal	YES	16327.6	82.2	4	124.2	1444	20	0	8
Concentric Inverted Chevron V	YES	160.4	3961.3	6	206.7	1472	0	0	8
Eccentric Diagonal	NO								
Eccentric Inverted Chevron V	YES	209.7	3006.9	6	223.5	1472	0	0	8

Table 22: Performance Point for 12-Story H Plan on X-axis

Type of Braced Frame	Performance Point Occurrence	Target Displacement (mm)	Shear (Kn)	Step Number	Monitored Displacement (mm)	A-IO	IO-LS	LS-CP	>CP
Concentric Diagonal	YES	270.3	26415.9	4	289.8	3876	42	0	42
Concentric Inverted Chevron V	YES	243.4	17391.9	8	14927.4	4384	0	0	56
Eccentric Diagonal	YES	324.8	19063.2	6	379.23872	3872	32	8	48
Eccentric Inverted Chevron V	YES	312.3	12225.6	11	337.3	4380	8	0	52

Table 23: Performance Point for 12-Story H Plan on Y-axis

Type of Braced Frame	Performance Point Occurrence	Target Displacement (mm)	Shear (Kn)	Step Number	Monitored Displacement (mm)	A-IO	IO-LS	LS-CP	>CP
Concentric Diagonal	YES	252.6	32185.94	3	278.7	3888	38	0	34
Concentric Inverted Chevron V	YES	233.3	28730.3	6	237.2	4366	50	0	24
Eccentric Diagonal	NO								
Eccentric Inverted Chevron V	NO								

Table 24: Performance Point for 12-Story Square Plan on X-axis

Type of Braced Frame	Performance Point Occurrence	Target Displacement (mm)	Shear (Kn)	Step Number	Monitored Displacement (mm)	A-IO	IO-LS	LS-CP	>CP
Concentric Diagonal	YES	296.8	22897.1	5	297.4	4332	38	8	58
Concentric Inverted Chevron V	YES	267.4	16700.1	11	304.8	4760	0	0	64
Eccentric Diagonal	YES	321.5	19395.9	6	327.9	4346	24	18	52
Eccentric Inverted Chevron V	YES	337.5	12356	8	359.6	4760	0	0	64

Table 25: Performance Point for 12-Story Square Plan on Y-axis

Type of Braced Frame	Performance Point Occurrence	Target Displacement (mm)	Shear (Kn)	Step Number	Monitored Displacement (mm)	A-IO	IO-LS	LS-CP	>CP
Concentric Diagonal	YES	305.9	21593.5	6	334.3	4344	22	16	58
Concentric Inverted Chevron V	YES	329	6212	18	390.6	4784	8	0	32
Eccentric Diagonal	NO								
Eccentric Inverted Chevron V	YES	432.4	5780.8	14	438.3	4792	0	0	32

Tables between 18 to 25 have listed the target displacement for every excited performance point with its shear force that lead to the target displacement, as well as the step that performance point occurred in with the plastic hinges condition for this step.

The target displacement of performance point of a structure that subjected to pushover analysis, has been selected at the top point of the structure, which is the roof story, by pushing the structure until it reach the target displacement after. These target displacements have been by exerting the inverted triangular form of loading to the structure.

As shown in table from 18 to 25 that obtained from modification curve and capacity-demand curve the largest target displacement values according to performance points were with all eccentric diagonal and inverted V braced frames. In some cases, the target displacement of performance point of eccentric diagonal braced frames does not exceed.

Therefore, from the previous analysis that has been done for four types of braced frames using pushover analysis, the eccentric diagonal braced frames were the best selection for 4 and 12 story, H and square plane.

4.2 Discussion of Response Spectrum Analysis

Discussion of the obtained results of braced frames under response spectrum analysis will be described in this section. The comparison between the selected types of bracing systems using response spectrum analysis will be through three methods. These method will depends on the performance of braced frames and the structures in order to identify the best bracing system for 4- and 12 story structure, and H and square plan.

4.2.1 Comparison Using Story Stiffness

The story stiffness is an important factor. In each story the stiffness will have a different value depends and the plan shape and elevation of the story. When the stiffness is larger, the smaller internal forces will be act on the structure as well as resisting the lateral loads in response spectrum analysis.

Examples about the results of modeling structure using response spectrum analysis taken from ETABS and listed below.

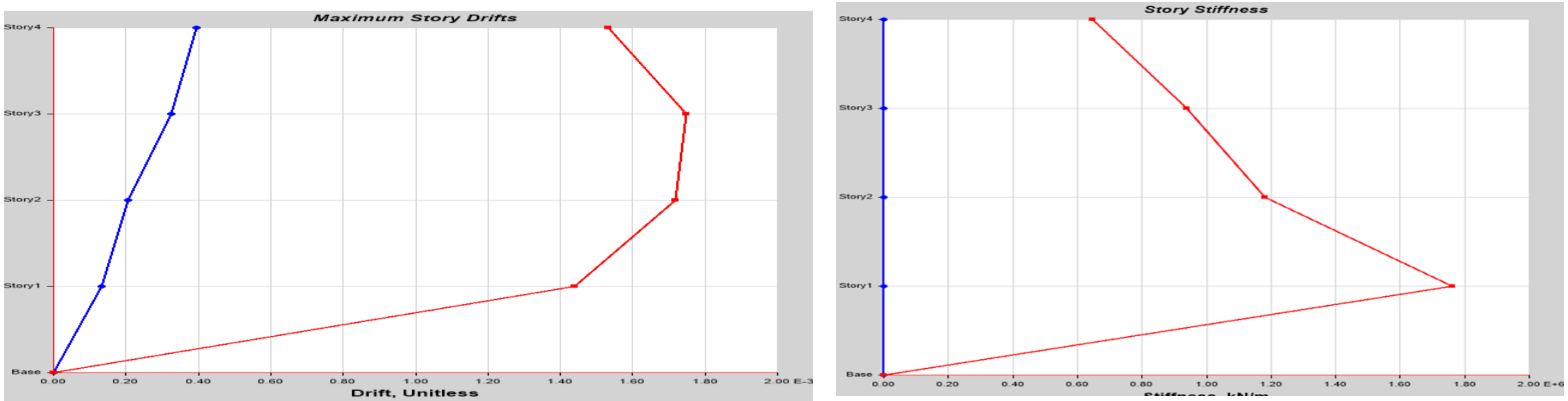

Figure 52: Story-Drift and Story Stiffness Curve for 4-Story H Plan Concentric Diagonal Braced Frame for on Y-axis

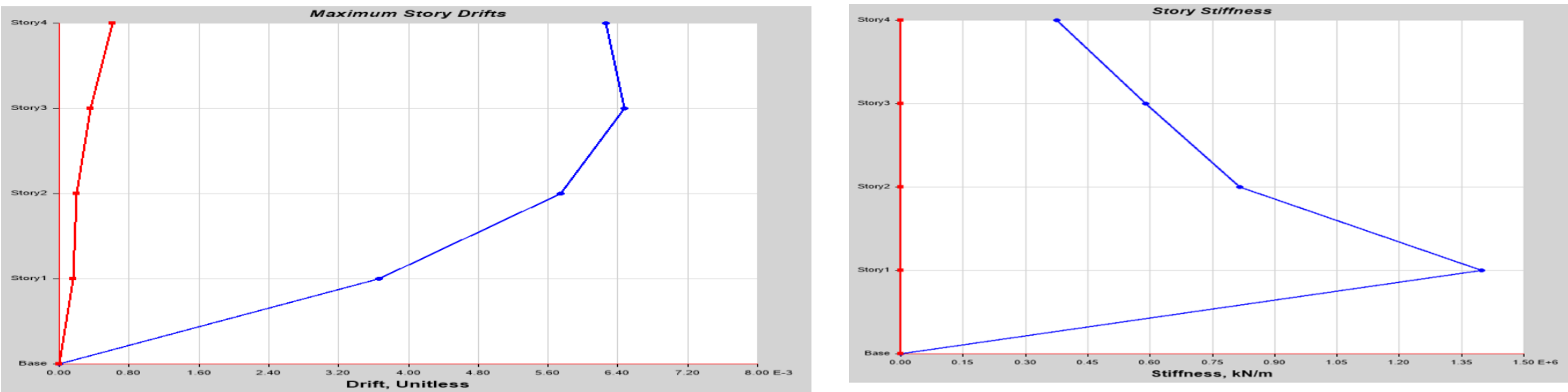

Figure 53: Story-Drift Curve and Story Stiffness for 4-Story Square Plan Concentric Inverted V Braced Frame for on X-axis

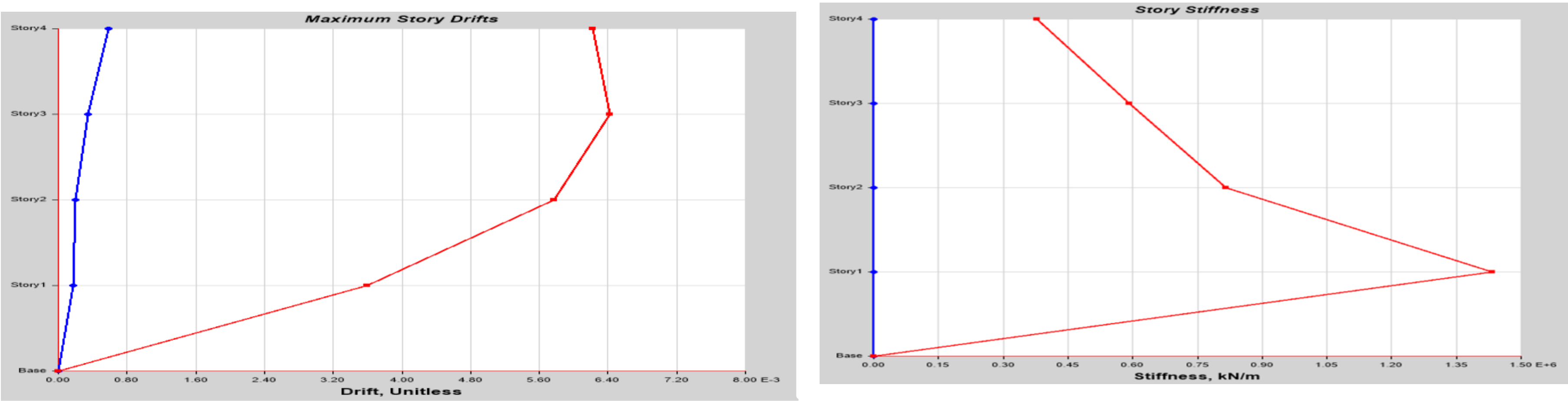

Figure 54: Story-Drift Curve and Story Stiffness for 4-Story Square Plan Concentric Inverted V Braced Frame for on Y-axis

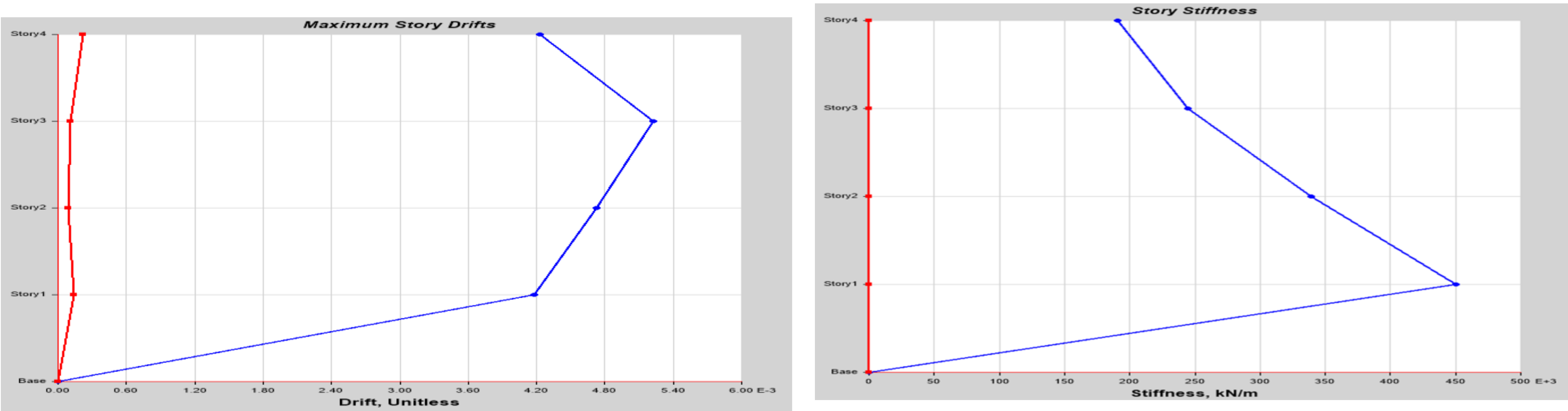

Figure 55: Story-Drift and Story Stiffness Curve for 4-Story Square Plan Eccentric Diagonal Braced Frame for on X-axis

In the following tables the maximum story stiffness of all braced frames for 4 and 12 stories and in H and square plane will be listed.

Table 26: Story Stiffness of 4-Story H Plan

Type of Braced Frame	Story Number	Elevation	X-Dir kN/m	Y-Dir (kN/m)
Concentric Diagonal	1	3	1360113.342	1760152.65
Concentric Inverted Chevron V	1	3	1299220.194	1354919.94
Eccentric Diagonal	1	3	418323.794	209319.832
Eccentric Inverted Chevron V	1	3	352324.061	224847.692

For table 26, the concentric diagonal braced frames was the stiffer structure compared with the other braced frames for the 4-story H plan. The total stiffness of the first story of the concentric braced frame is 3.12×10^6 kN/m. On the other hand, eccentric inverted V has the lowest story stiffness with 5.8×10^5 kN/m.

Table 27: Story Stiffness of 4-Story Square Plan

Type of Braced Frame	Story Number	Elevation	X-Dir kN/m	Y-Dir (kN/m)
Concentric Diagonal	1	3	570253.333	1541049.639
Concentric Inverted Chevron V	1	3	1397269.02	1432204.008
Eccentric Diagonal	1	3	450211.745	245200.293
Eccentric Inverted Chevron V	1	3	355447.358	226687.436

As shown in table 27 concentric inverted V has the maximum stiffness of the first story for all braced frames of 4-story square plan. The stiffness of the first story of the concentric inverted V is 2.8×10^6 kN/m, while eccentric inverted V has the lowest story stiffness of the first story with 5.8×10^5 kN/m, which is similar to the value in 4-story H plan.

Table 28: Story Stiffness of 12-Story H Plan

Type of Braced Frame	Story Number	Elevation	X-Dir kN/m	Y-Dir (kN/m)
Concentric Diagonal	1	3	2288001.888	3255945.468
Concentric Inverted Chevron V	1	3	2388587.58	4047353.282
Eccentric Diagonal	1	3	1219184.862	979083.644
Eccentric Inverted Chevron V	1	3	706607.28	758577.054

Table 29: Story Stiffness of 12-Story Square Plan

Type of Braced Frame	Story Number	Elevation	X-Dir kN/m	Y-Dir (kN/m)
Concentric Diagonal	1	3	2082775.122	2090029.096
Concentric Inverted Chevron V	1	3	2397205.412	2618959.879
Eccentric Diagonal	1	3	1353665.086	661481.608
Eccentric Inverted Chevron V	s	3	703511.698	444314.988

.For both tables 28 and 29, the highest stiffness of the first story for 12-story H and square plan achieved the concentric inverted V braced frames with values reached $6.4 \times$ kN/m and 5×10^6kn/m respectively.

4.2.2 Story Displacement Comparison

Response spectrum analysis has a big effect on the stories within the structure. As mentioned in section 3.7.3 the displacement at each story of the structure should be minimized as much as possible by an efficient braced frames in order to prevent this displacement to be more the allowable value.

According to Eurocode 8, the displacement at each story during linear analysis should not be more than: $h/300$, (Eq. 5.1)

Where h is the elevation of the story.

The maximum story displacement for 4 and 12 story and H and square plan of braced frames will be listed in the tables below.

Table 30: Displacement of 4-Story H Plan for All Braced Frames

Type of Braced Frame	Number of Story	Elevation (m)	Displacement of Story Due to applied Response Spectrum on X-axis (mm)	Displacement of Story Due to applied Response Spectrum on Y-axis (mm)	Allowable Displacement (mm)
Concentric Diagonal	4	12	21.4	18.6	40
Concentric Inverted Chevron V	4	12	57.9	47.7	40
Eccentric Diagonal	4	12	44.1	52.4	40
Eccentric Inverted Chevron V	4	12	98.8	113.1	40

In table 30, all braced frames that have been selected for this study has displacement value bigger than the allowable value, except the concentric diagonal braced frames, which remain safe during the application of response spectrum analysis on both axis X and Y. All the other braced frames fulfill the allowable displacement criteria.

Table 31: Displacement of 4-Story Square Plan for All Braced Frames

Type of Braced Frame	Number of Story	Elevation (m)	Displacement of Story Due to applied Response Spectrum on X-axis (mm)	Displacement of Story Due to applied Response Spectrum on Y-axis (mm)	Allowable Displacement (mm)
Concentric Diagonal	4	12	24.8	26.7	40
Concentric Inverted Chevron V	4	12	65	64.5	40
Eccentric Diagonal	4	12	52.7	65.2	40
Eccentric Inverted Chevron V	4	12	107.6	129	40

In table 31, the concentric diagonal braced frames is the only braced frame that remain in the safe side for the allowable displacement criteria while all the other braced system has been fulfill the criteria.

Table 32: Displacement of 12-Story H Plan for All Braced Frames

Type of Braced Frame	Story Number	Elevation (m)	Displacement of Story Due to applied Response Spectrum on X-axis (mm)	Displacement of Story Due to applied Response Spectrum on Y-axis (mm)	Allowable Displacement (mm)
Concentric Diagonal	12	36	95.4	86.6	120
Concentric Inverted Chevron V	12	36	187.2	173.7	120
Eccentric Diagonal	12	36	119	115.4	120
Eccentric Inverted Chevron V	12	36	241.4	236.1	120

The result of analyzing the 12-story H plan was different that than the results that obtained from 4-story H and square plan. As shown in table 32, the displacement values of concentric diagonal braced frames and eccentric diagonal braced frames were under the allowable displacement, which is 120 mm for each direction.

Table 33: Displacement of 12-Story Square Plan for All Braced Frames

Type of Braced Frame	Story Number	Elevation (m)	Displacement of Story Due to applied Response Spectrum on X-axis (mm)	Displacement of Story Due to applied Response Spectrum on Y-axis (mm)	Allowable Displacement (mm)
Concentric Diagonal	12	36	110.7	114.4	120
Concentric Inverted Chevron V	12	36	206.4	201.9	120
Eccentric Diagonal	12	36	125.2	157.1	120
Eccentric Inverted Chevron V	12	36	267.8	305.2	120

In the results analysis of response spectrum for12-story square plan was similar with 4-story H and square plan. Table 33 shows that the displacement value of concentric diagonal braced frame is also the only bracing system that satisfy the allowable displacement criteria with 110.7 mm on X-direction and 114.4 mm on Y-direction.

4.2.3 Comparison of Story Drift

As mentioned in section 3.7.3.3, accidental drift or story drift of a structure can be obtained by using response spectrum analysis. The drift has a unitless value that should be not more than the allowable drift value that mentioned in the Eurocode.

The allowable drift value can be obtained from the formula:

$$\theta \leq 0.01 \times h \qquad \text{(Eq 5.2)}$$

Where θ is the story drift, and h is the elevation of the story.

Tables below contains the maximum story drift in the structure on X and Y direction. Due to the applied response spectrum on X or Y axis, only the drifts within the same direction will be considered and neglecting the other value because it will be small compared with its value when the response spectrum applied on its direction.

Table 34: Drift Values of 4-Story H Plan During Response Spectrum on X-axis

Type of braced Frame	Story	Elevation	X-axis	Allowable Drift
Concentric Diagonal	2	6	0.002046	0.04
Concentric Inverted V	3	9	0.005851	0.04
Eccentric Diagonal	3	9	0.004279	0.04
Eccentric Inverted V	2	6	0.009198	0.04

Table 35: Drift Values of 4-StoryH Plan During Response Spectrum on Y-axis

Type of braced Frame	Story	Elevation	Y-axis	Allowable Drift
Concentric Diagonal	3	9	0.001749	0.04
Concentric Inverted V	3	9	0.004547	0.04
Eccentric Diagonal	1	3	0.005677	0.04
Eccentric Inverted V	1	3	0.012373	0.04

As shown in tables 34 and 35, all braced frames are below the allowable drift value, which is 0.04. The best braced frame in resisting story drift for 4-sory H plan is the concentric diagonal braced frame for both X and Y axis.

Table 36: Drift Values of 4-Story Square Plan During Response Spectrum on X-axis

Type of braced Frame	Story	Elevation	X-axis	Allowable Drift
Concentric Diagonal	3	9	0.002455	0.04
Concentric Inverted V	3	9	0.006468	0.04
Eccentric Diagonal	3	9	0.005225	0.04
Eccentric Inverted V	2	6	0.010181	0.04

Table 37: Drift Values of 4-Story Square Plan During Response Spectrum on Y-axis

Type of braced Frame	Story	Elevation	Y-axis	Allowable Drift
Concentric Diagonal	3	9	0.002671	0.04
Concentric Inverted V	3	9	0.006422	0.04
Eccentric Diagonal	1	3	0.006252	0.04
Eccentric Inverted V	1	3	0.01314	0.04

The results of story drift for 4-story square plan (Tables 34 and 35) shows that eccentric inverted V has the largest drift value compared with all braced frames. The best braced frame in resisting story drift for 4-sory H plan is the concentric diagonal braced frame for both X (0.002455) and Y (0.002671) axis.

Table 38: Drift Values of 12-Story H Plan During Response Spectrum on X-axis

Type of braced Frame	Story	Elevation	X-axis	Allowable Drift
Concentric Diagonal	10	30	0.003911	0.04
Concentric Inverted V	10	30	0.007253	0.04
Eccentric Diagonal	9	27	0.004479	0.04
Eccentric Inverted V	9	27	0.008113	0.04

Table 39: Drift Values of 12-Story H Plan During Response Spectrum on Y-axis

Type of braced Frame	Story	Elevation	Y-axis	Allowable Drift
Concentric Diagonal	11	33	0.00335	0.04
Concentric Inverted V	11	33	0.007067	0.04
Eccentric Diagonal	10	30	0.004123	0.04
Eccentric Inverted V	9	27	0.008041	0.04

Table 40: Drift Values of 12-Story Square Plan During Response Spectrum on X-axis

Type of braced Frame	Story	Elevation	X-axis	Allowable Drift
Concentric Diagonal	11	33	0.004536	0.04
Concentric Inverted V	11	33	0.008371	0.04
Eccentric Diagonal	10	30	0.004897	0.04
Eccentric Inverted V	9	27	0.009087	0.04

Table 41: Drift Values of 12-Story Square Plan During Response Spectrum on Y-axis

Type of braced Frame	Story	Elevation	Y-axis	Allowable Drift
Concentric Diagonal	10	30	0.004581	0.04
Concentric Inverted V	11	33	0.007755	0.04
Eccentric Diagonal	9	27	0.005573	0.04
Eccentric Inverted V	8	24	0.009941	0.04

From tables 38 and 39, the braced system has approximately same value of story drift under applied response spectrum on X axis and Y axis. The values eccentric and concentric diagonal braced frame for both axis are same with an advantage for the concentric frame for 12-story H and square plans.

4.3 Comparison between Pushover and Response Spectrum Analysis

In fact, it is difficult to compare between two different types of analysis, especially if they are linear and nonlinear, because in nonlinear analysis, hinges should be assigned for all beams, columns and braced frames while there is no assigned hinges in the linear analysis. There are two simple way to compare between pushover and response spectrum analysis, and it will be described below.

4.3.1 Comparison by the Applied Base Shear

Figures below will compare only the shear force at the base due to the applied forces by pushover and response spectrum without considering any additional load (Dead load, Self-Wight….etc.).

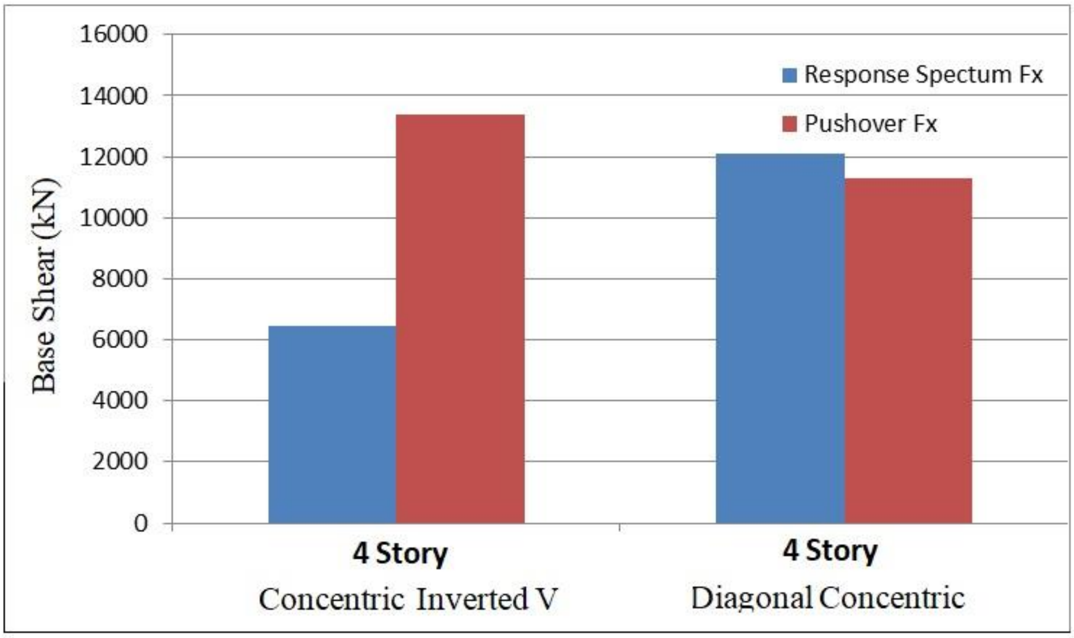

Figure 56: Base Shear on X-axis 4-Story H Plan Inverted V and Diagonal Concentric Braced Frame

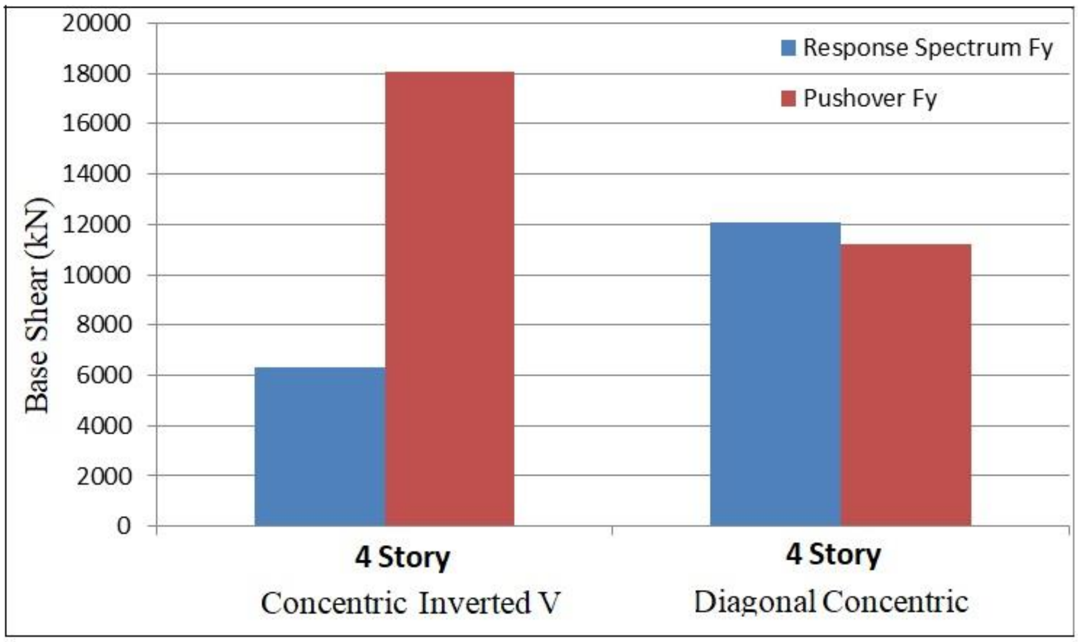

Figure 57: Base Shear on X-axis 4-Story Square Plan Inverted V and Diagonal Concentric Braced Frame

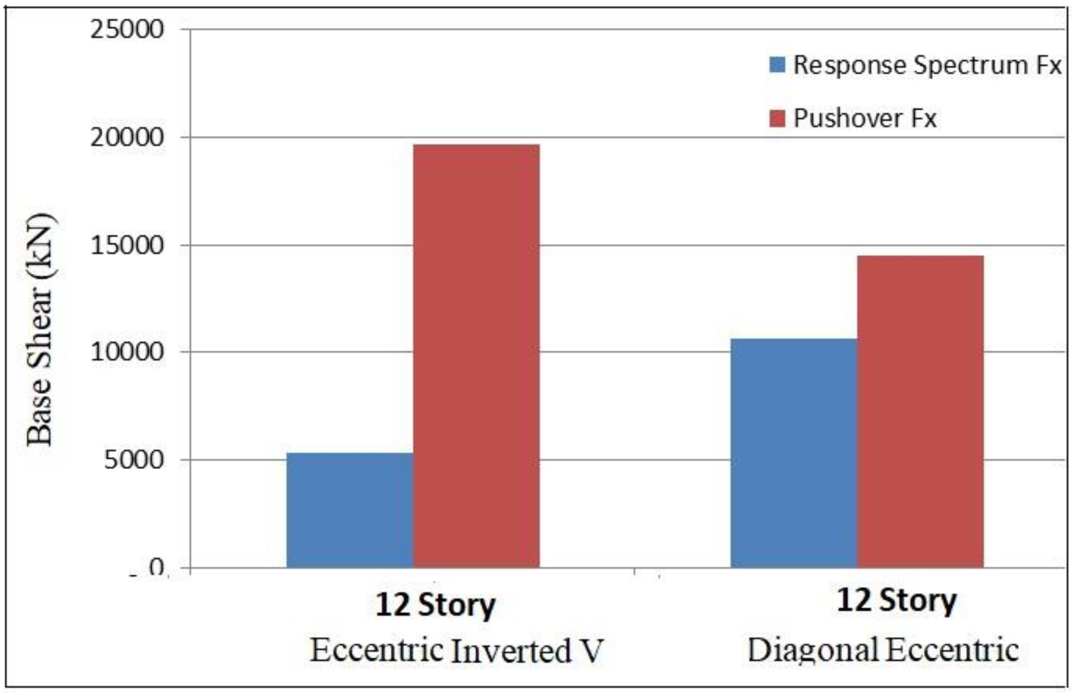

Figure 58: Base Shear on X-axis H Plan 12-Strory Eccentric Inverted V and Diagonal

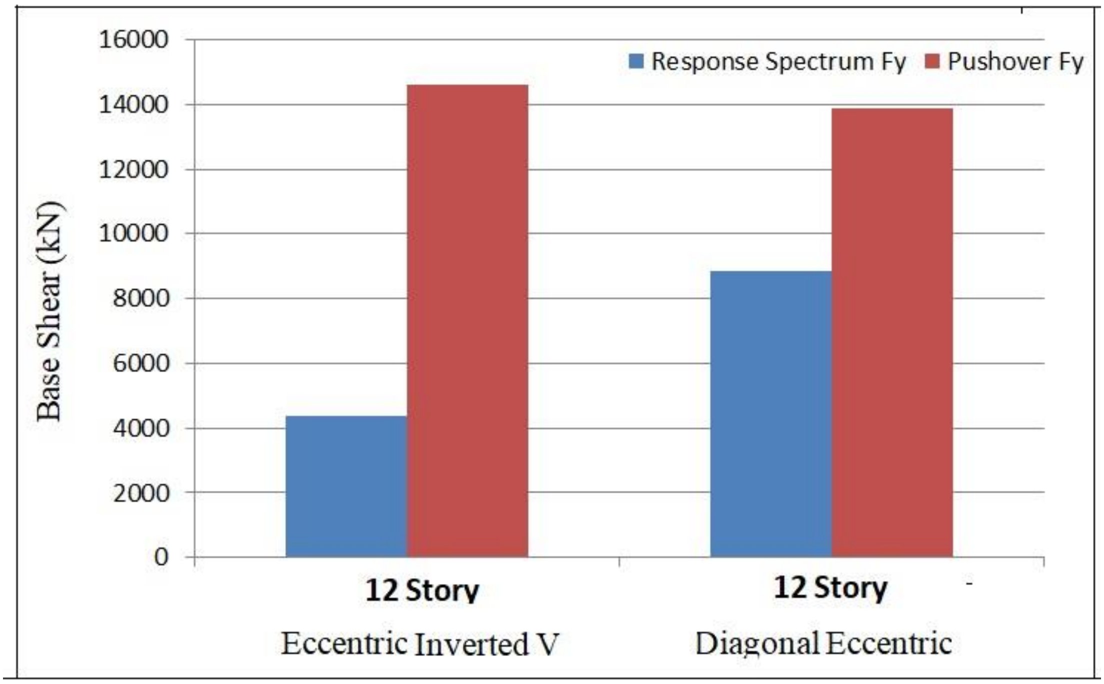

Figure 59: Base Shear on Y-axis Square Plan 12-Strory Eccentric Inverted V and Diagonal

Figures 85, 86, 87 and 88 compares between the base shear due to pushover and response spectrum analysis on X and Y directions for diagonal concentric and eccentric braced frames for 4 and 12 stories. The values of base shear during the pushover analysis for 12 stories are larger than base shear due to response spectrum analysis. While for 4-story, base shear during response spectrum and pushover analysis varies from X to Y directions and from H to square plan.

4.4 Comparison of the Weight of the Braced Framed

The weight for all beams, columns and bracing members will effects the economical side of this comparison as much as it is one of the most important factor that can effects the behaviour of the structure.

Table 42 : Total Structural Weight

Braced Frame	4-story H plan (Tonnes)	4-story Square Plan (Tonnes)	12-story H plan (Tonnes)	12-story Square Paln (Tonnes)
Concentric Diagonal	130.7	149.5	569.1	554.8
Concentric Inverted V	118.1	136.8	508.3	559
Eccentric Diagonal	115.2	136.9	494.3	548.6
Eccentric Inverted V	112.9	129.4	486.6	535.8

As shown in the table 42, the best selection of braced system for all systems 4-and 12-story, H plan and square plan will be the eccentric Inverted V braced frame, because it is the lighter weight between all bracing frame systems.

Chapter 5

CONCLUSION

5.1 Introduction

Earthquake actions leads to loss of live for many people every year in different regions on the world. For many years, lateral resisting systems were the main topic for many researchers in order to provide the best lateral resisting system. Steel braced frames are one of the lateral resisting systems for steel structure. It becomes one the most important system to resist the earthquake loads. Steel braced frames divided into two categorize concentric braced frames and eccentric braced frames.

The purpose of this study is to compare between two types of concentric braced frames with similar two types of eccentric braced frames. In addition, this study aimed to obtain the effects of regular and irregular 3D modeling with 6 bays each bay has length 5 meter. The study aimed also to check the effect rise in elevation of the structure, and to achieve this, 4- and 12 story steel structures used.

Two types of methods used for this study, pushover and response spectrum analysis in order to obtain the behaviour of these selected braced frames under linear and nonlinear analysis. Nonlinear analysis of steel structure provided by assigning hinges in beams, columns and bracing members in order to obtain the condition of these assigned hinges performance point with its target displacement for pushover curves.

While linear analysis using response spectrum aimed to obtain the displacement, drift for each story, and check the stiffness and stability of the structure.

5.2 Overall Conclusion

After analyzing the structure for two types of concentric braced frames and two types of eccentric braced frames, for H and square plan, 4- and 12-story using pushover and response spectrum analysis, the following are the overall conclusions:

- Few numbers of assigned hinges in beams changed to immediate occupancy in most of the case.
- The eccentric diagonal braced frame was more efficient for the pushover analysis with the lowest number of collapsed hinges.
- The eccentric diagonal braced frame was also the best selected braced frames for 2D study according to Nourbakhs, S. M. (2011).
- ETABS (using default hinges) do not provide detailed info on hinges for columns, beams and bracing separately. Therefore, the decision can be reached by considering the total given information of collapsed hinges in the structures.
- Concentric diagonal braced frames has more stiffness than other frames, and concentric inverted V braced frames has more stiffness than the eccentric braced frames.
- Concentric diagonal braced frames was the only frames that provided the displacement allowable criteria, while the other frames fulfill the allowable displacement for all plans and elevations, except the eccentric diagonal braced frame in 4-story square plan.
- All braced frames were below the allowable value for story drift for all axis, elevations and plans.

- The lowest value of story drift achieved by the concentric diagonal braced frames for all axis, elevations and plans.
- In general, eccentric braced shows better results for the structural analysis using pushover method, while concentric braced frames shows the better results during the response spectrum analysis.
- The pushover analysis has larger shear force at the base for 12-story H and square plans than response spectrum analysis.
- The most efficient braced frames during the economical point of view were the eccentric inverted chevron V braced frames for all axis, elevations and plans with lowest weight of structural members.

5.3 Recommendation for Future Studies

Increasing in the elevation and using of the nonlinear dynamic analysis with asymmetric plan shape using the diagonal eccentric and concentric braced frames is to be recommended for the future studies.

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APPENDIX

Appendix A

4-Story Concentric Diagonal Braced Frame for H-plan

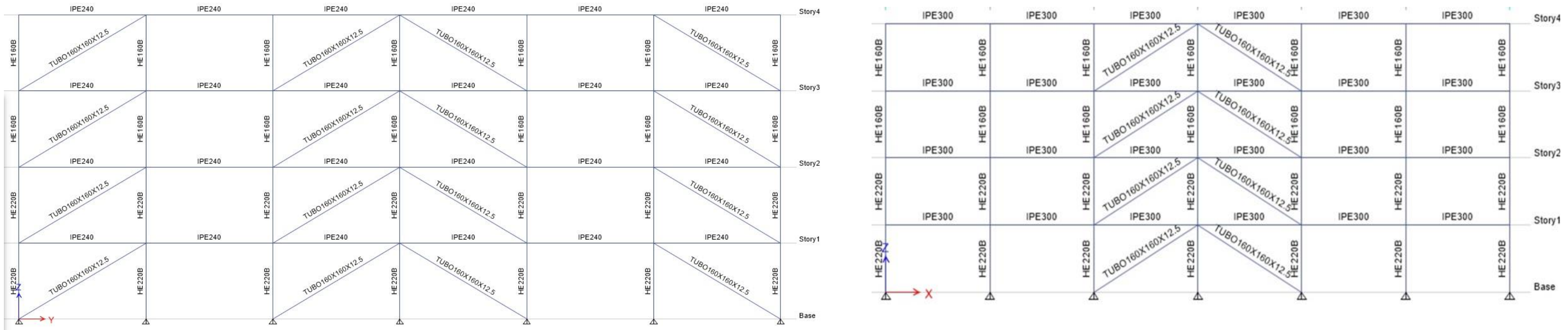


Figure A60: Design Section 4-Story H Plan Concentric Diagonal Braced Frame External Section on X-axis

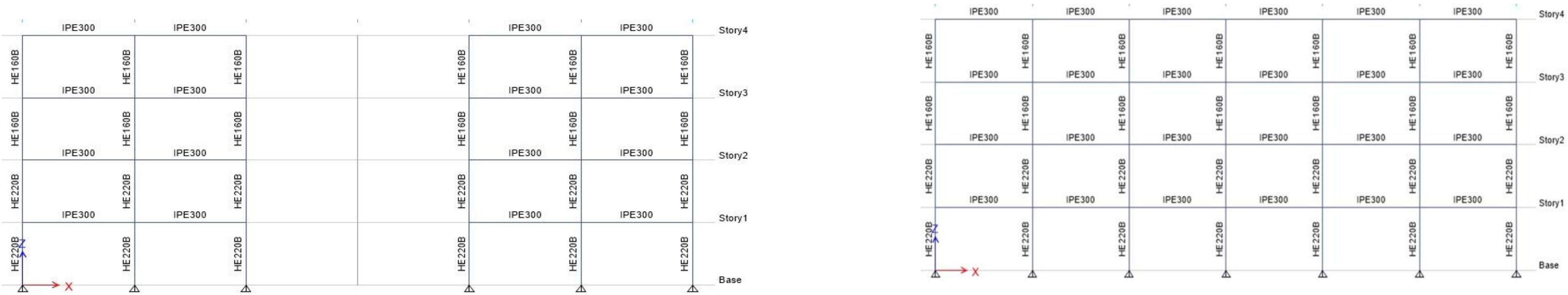
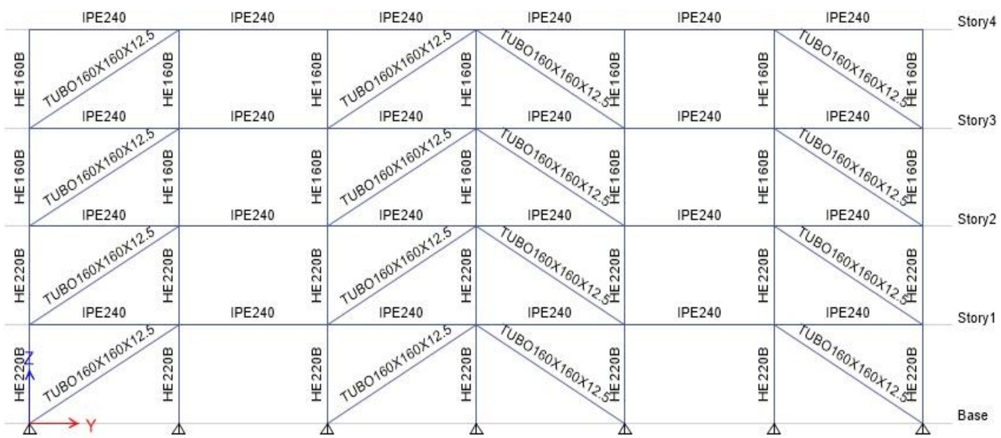


Figure A61: Design Section 4-Story H Plan Concentric Diagonal Braced Frame Internal Section on X-axis



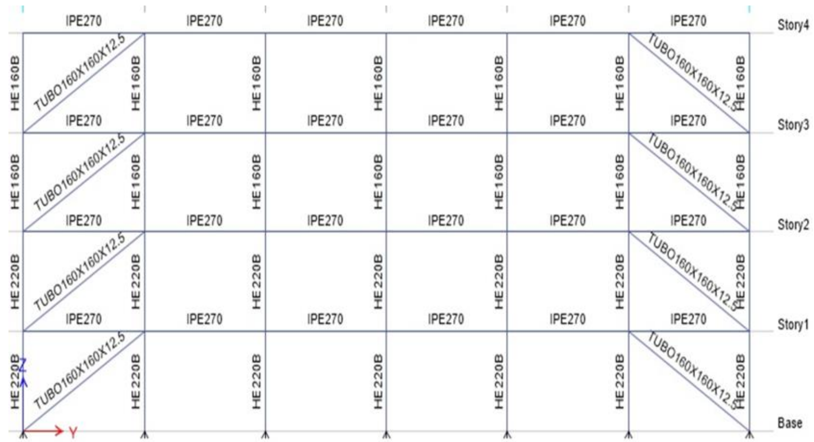
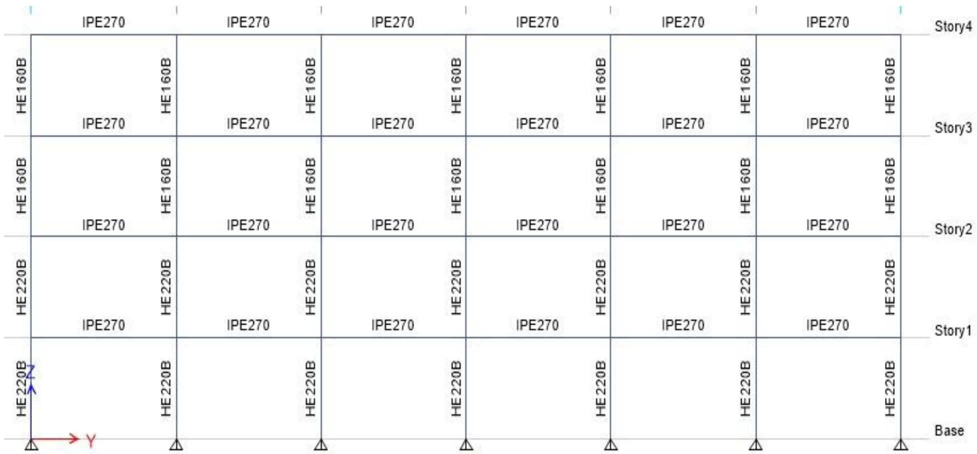


Figure A62: Design Section 4-Story H Plan Concentric Diagonal Braced Frame External Section on Y-axis



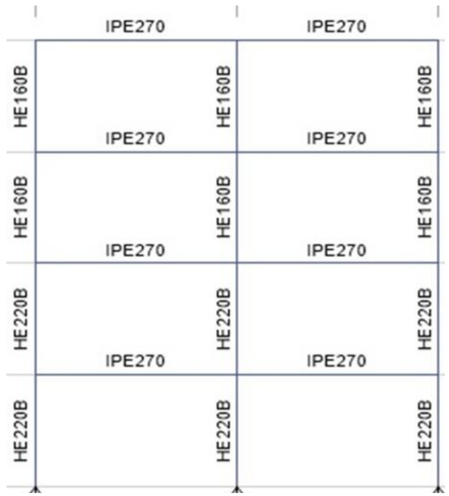


Figure A63: Design Section 4-Story H Plan Concentric Diagonal Braced Frame Internal Section on Y-axis

4-Story Concentric Diagonal Braced Frame for Square plan

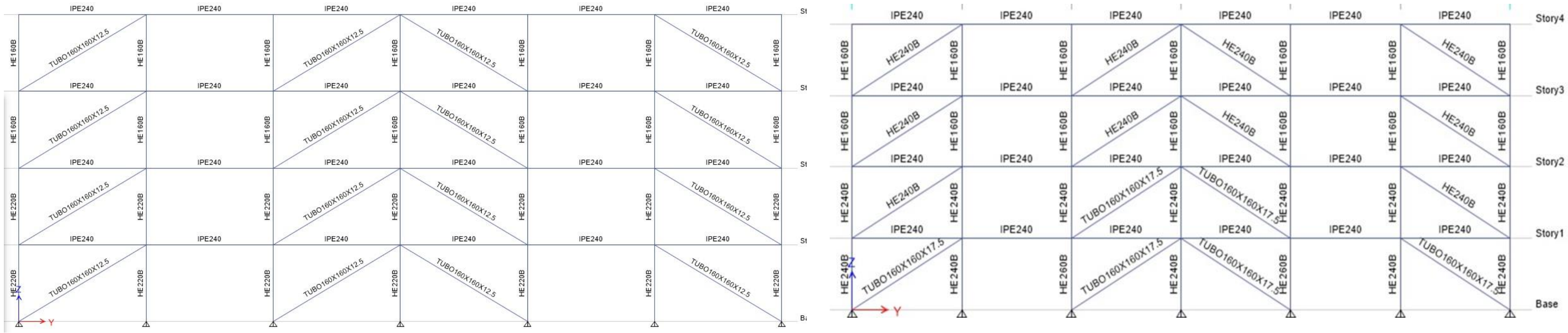


Figure A64: Design Section 4-Story Square Plan Concentric Diagonal Braced Frame External Section on X and Y axis

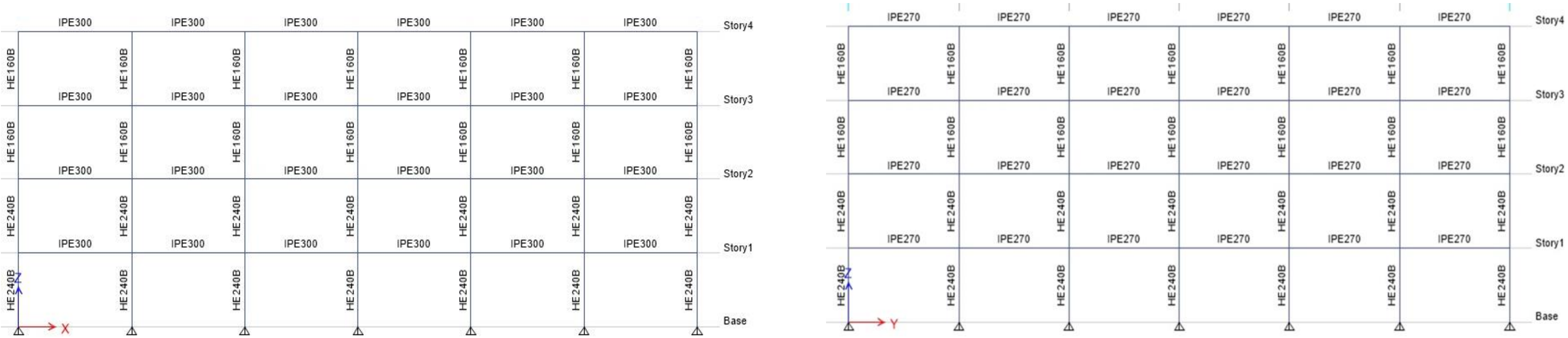


Figure A65: Design Section 4-Story Square Plan Concentric Diagonal Braced Frame Internal Section on X and Y axis

4-Story Concentric Inverted Chevron-V Braced Frame for H-plan

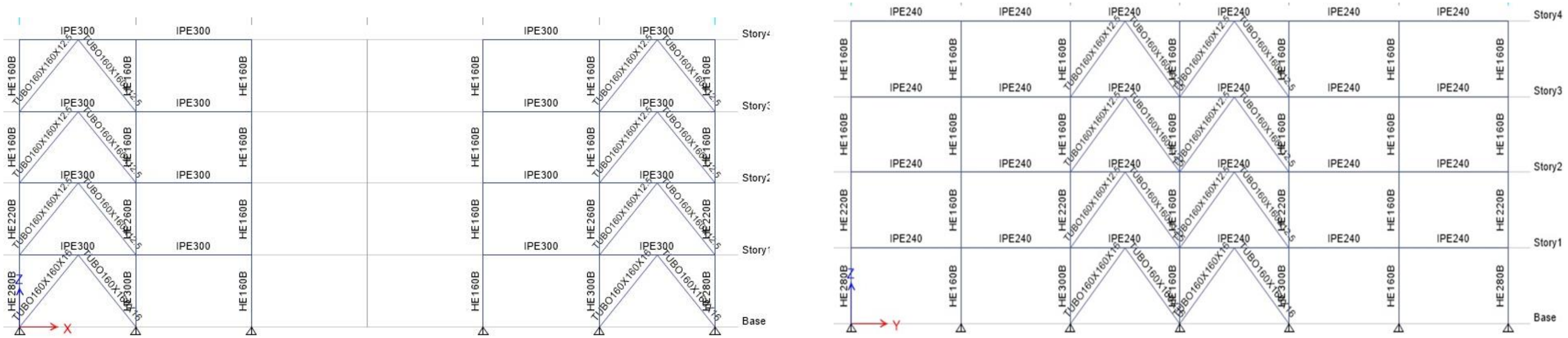


Figure A66: Design Section 4-Story H Plan Concentric Inverted V Braced Frame External Section on X and Y axis

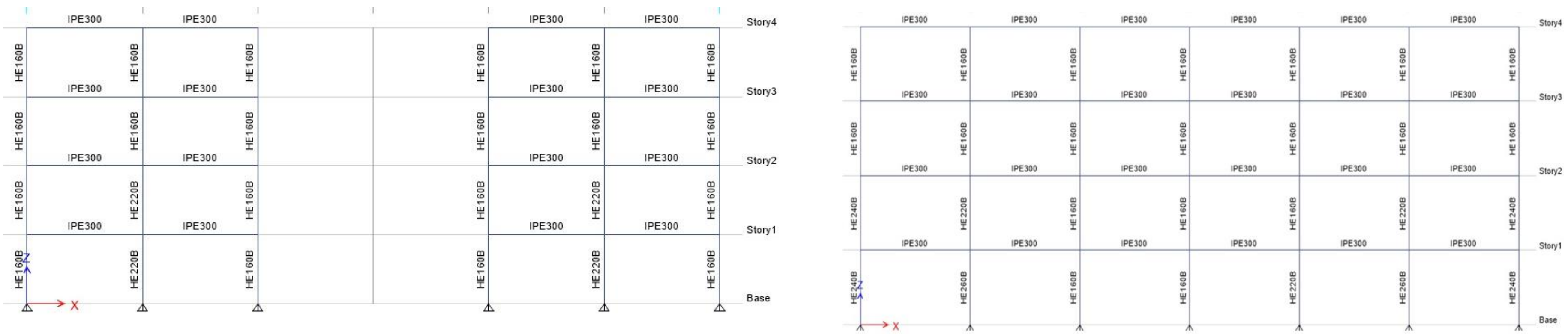
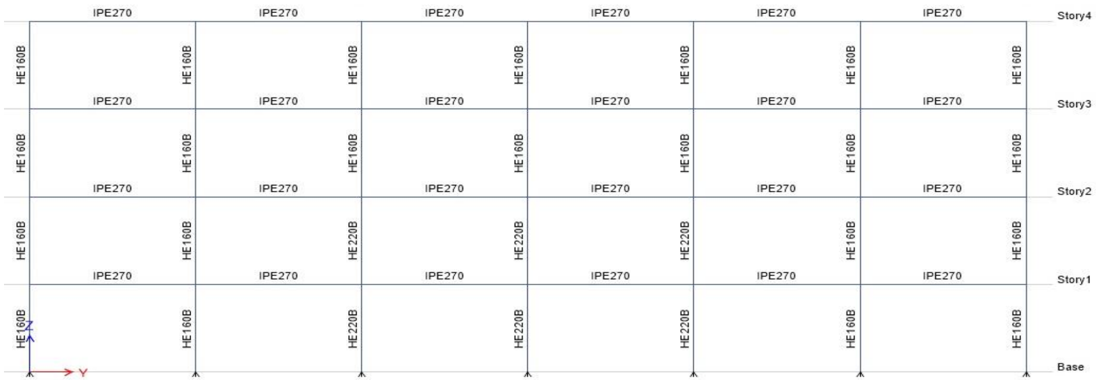


Figure A67: Design Section 4-Story H Plan Concentric Inverted V Braced Frame Internal Section on X-axis



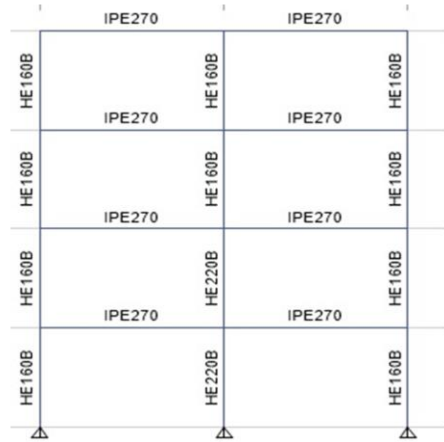
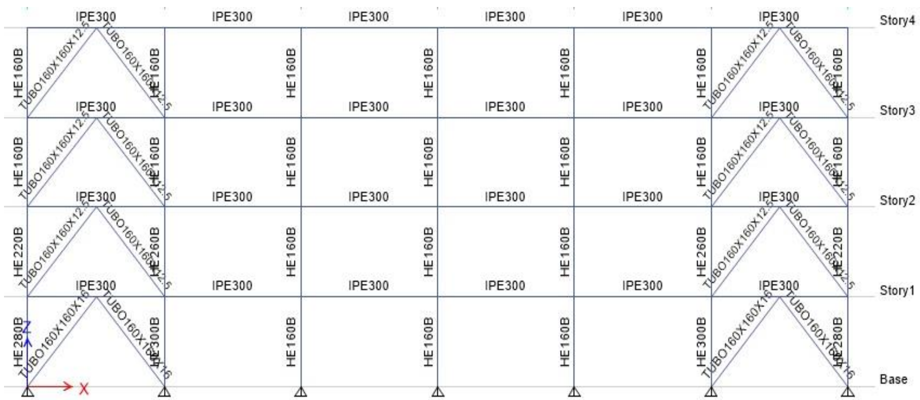


Figure A68: Design Section 4-Story H Plan Concentric Inverted V Braced Frame Internal Section on X-axis

4-Story Concentric Inverted Chevron-V Braced Frame for Square plan



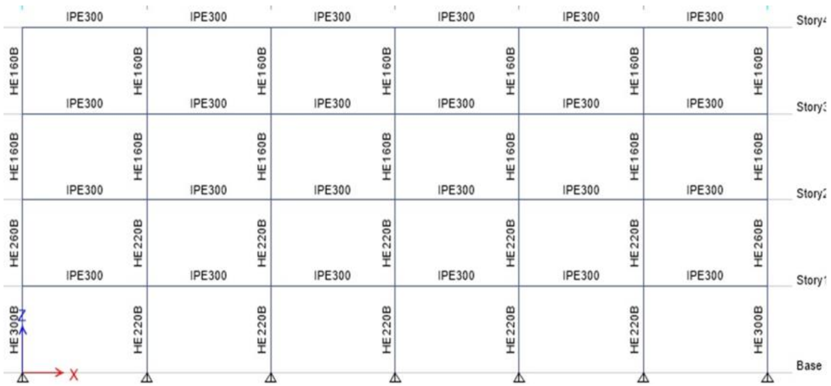


Figure A69: Design Section 4-Story Square Plan Concentric Inverted V Braced Frame External Section on X-axis

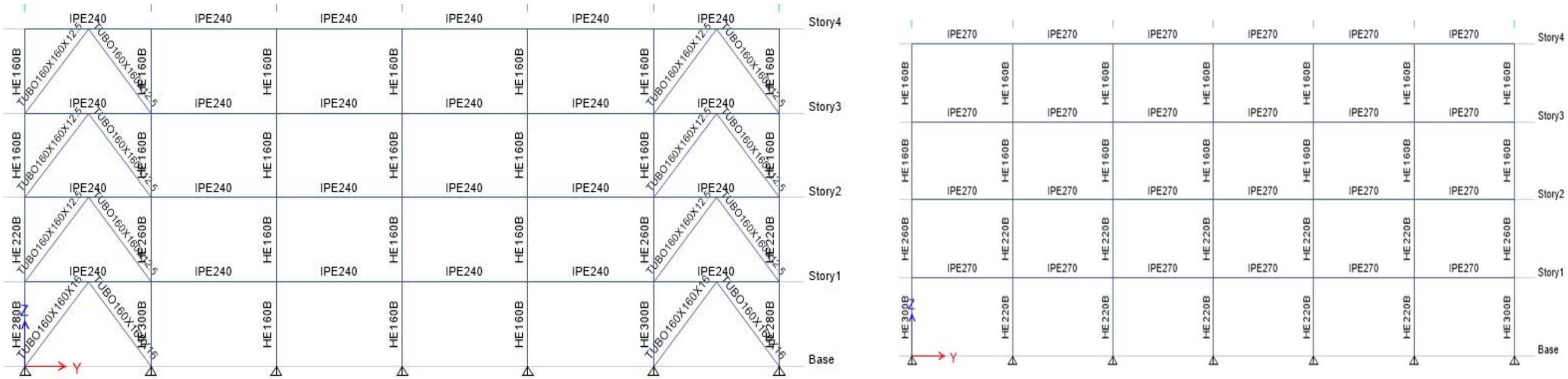


Figure A70: Design Section 4-Story Square Plan Concentric Inverted V Braced Frame External and Internal Section on Y-axis

4-Story Eccentric Diagonal Braced Frame for H-plan

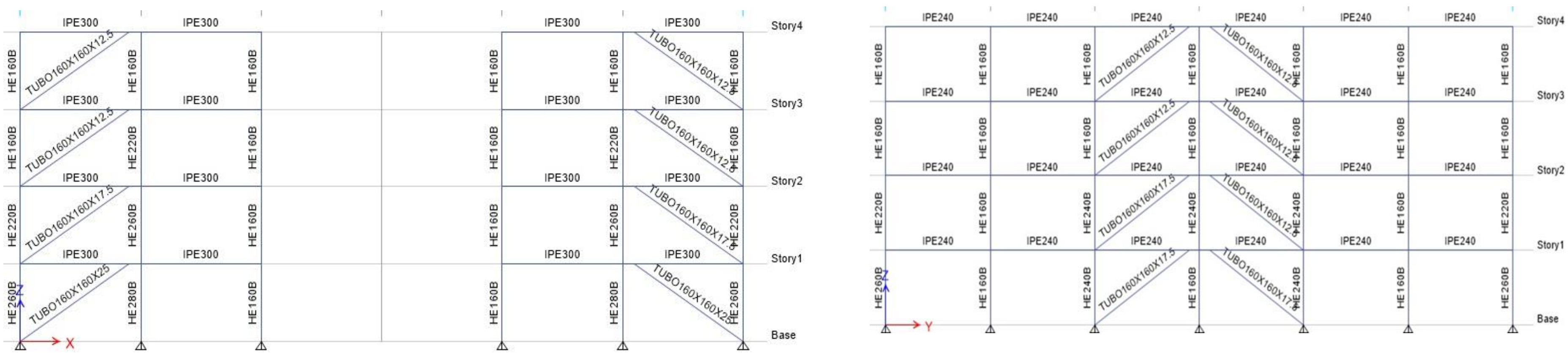
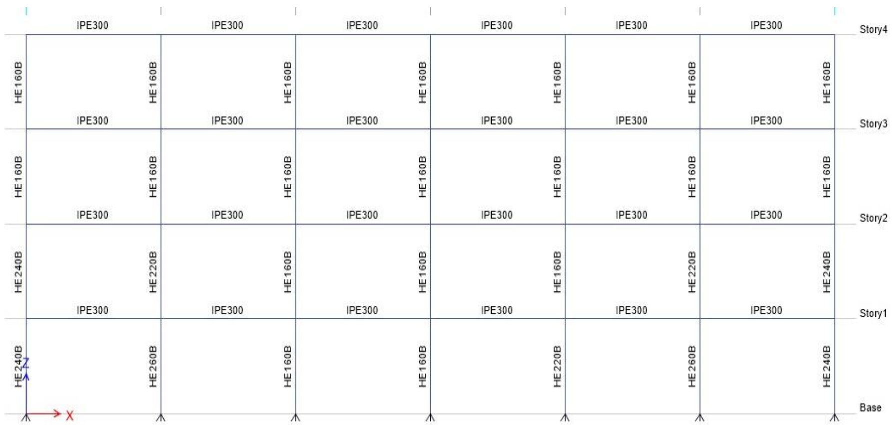


Figure A71: Design Section 4-Story H Plan Eccentric Diagonal Braced Frame External Section on X and Y axis



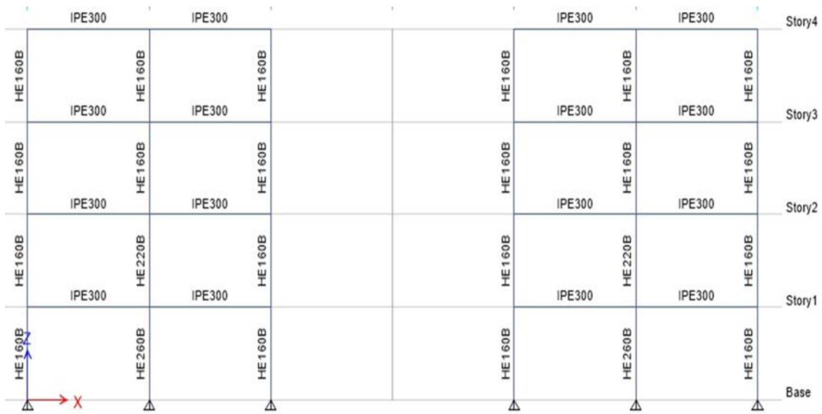
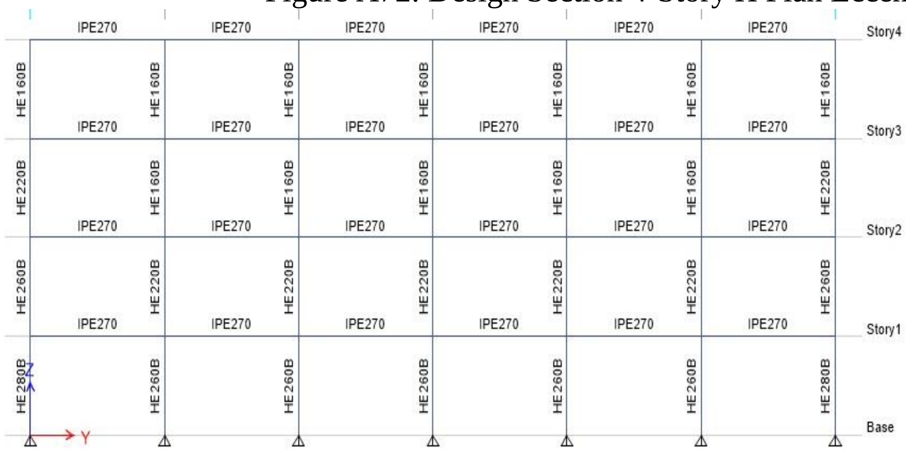


Figure A72: Design Section 4-Story H Plan Eccentric Diagonal Braced Frame Internal Section on X-axis



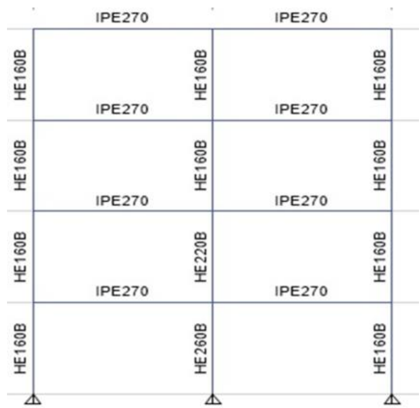


Figure A73: Design Section 4-Story H Plan Eccentric Diagonal Braced Frame External Section on Y-axis

4-Story Eccentric Diagonal Braced Frame for Square plan

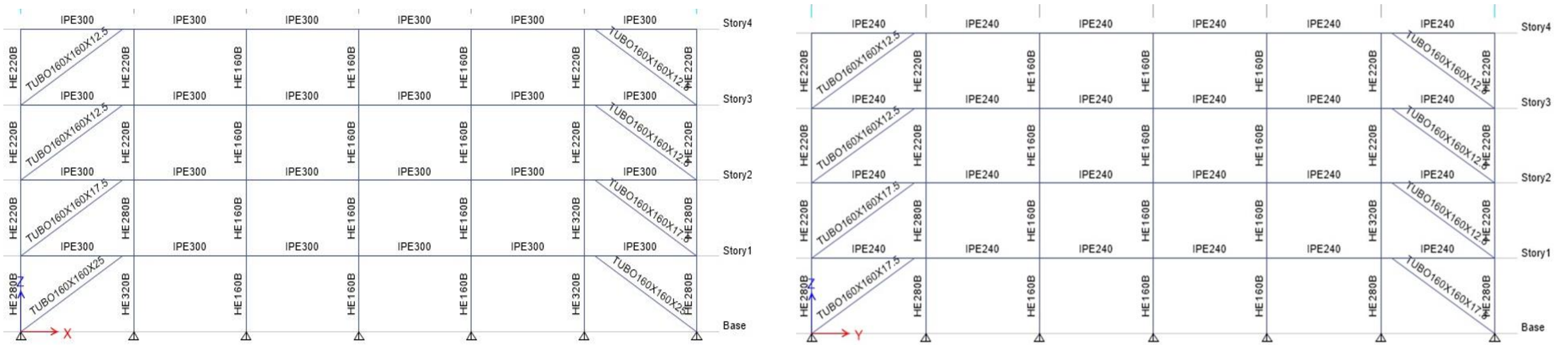


Figure A74: Design Section 4-Story Square Plan Eccentric Diagonal Braced Frame External Section on X and Y axis

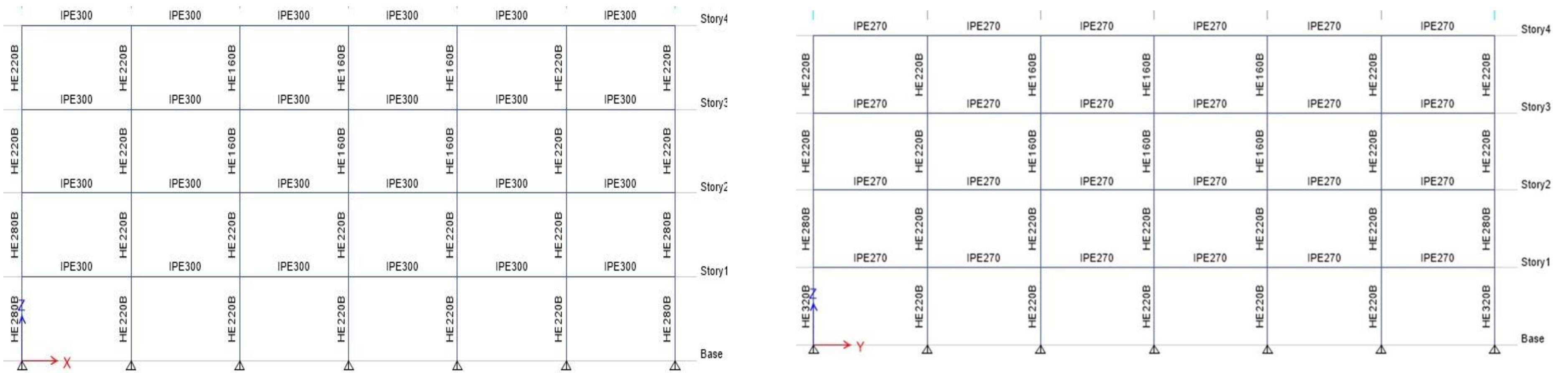


Figure A75: Design Section 4-Story Square Plan Eccentric Diagonal Braced Frame Internal Section on X and Y axis

4-Story Eccentric Inverted Chevron-V Braced Frame for H-plan

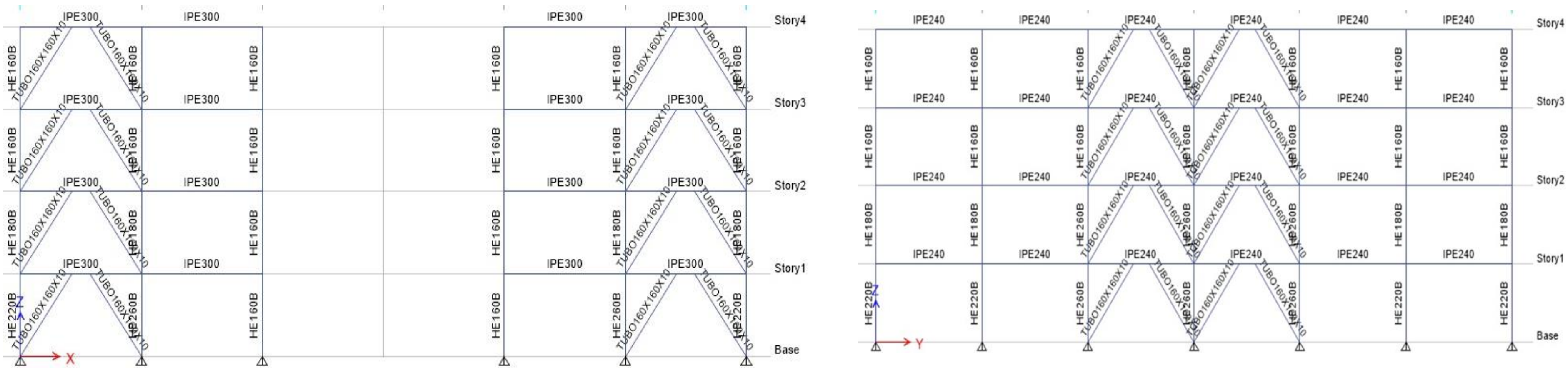


Figure A76: Design Section 4-Story H Plan Eccentric Inverted V Braced Frame External Section on X and Y axis

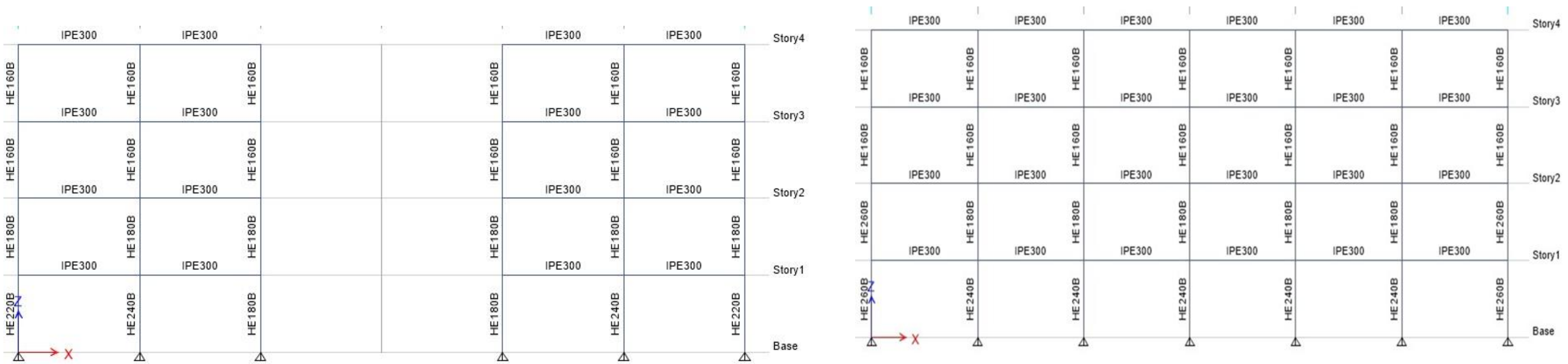


Figure A77: Design Section 4-Story H Plan Eccentric Inverted V Braced Frame Internal Section on X -axis

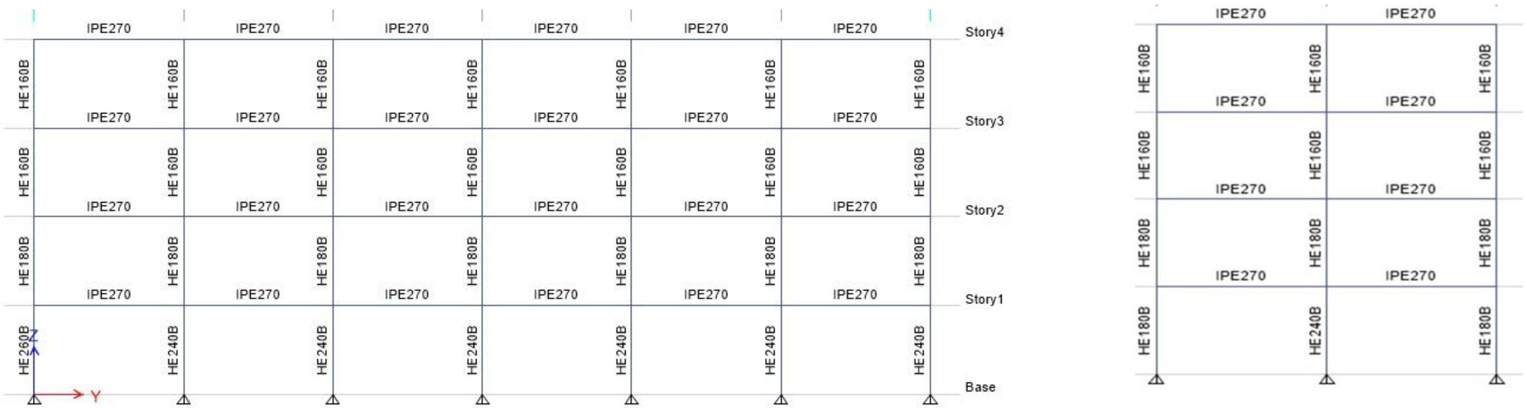


Figure A78: Design Section 4-Story H Plan Eccentric Inverted V Braced Frame Internal Section on Y-axis

4-Story Eccentric Inverted Chevron-V Braced Frame for Square plan

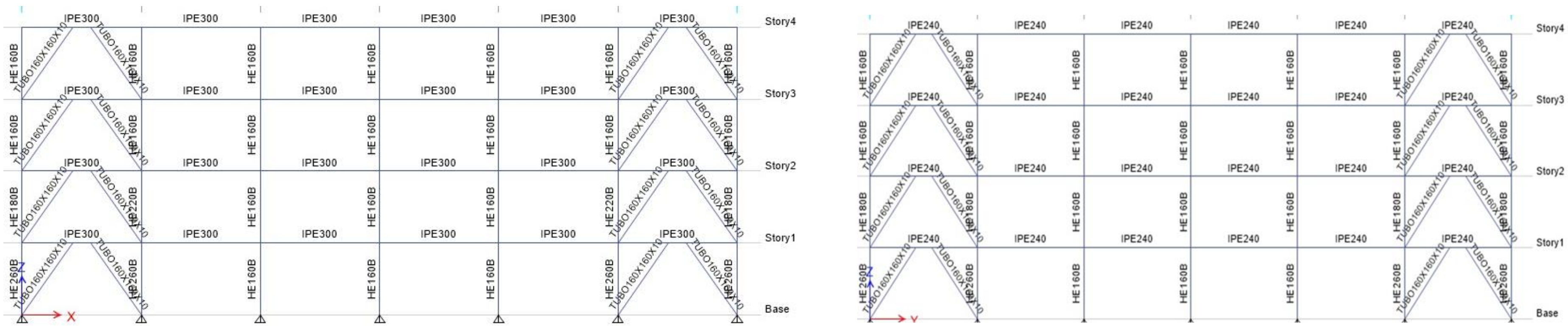


Figure A79: Design Section 4-Story Square Plan Eccentric Inverted V Braced Frame External Section on X and Y axis

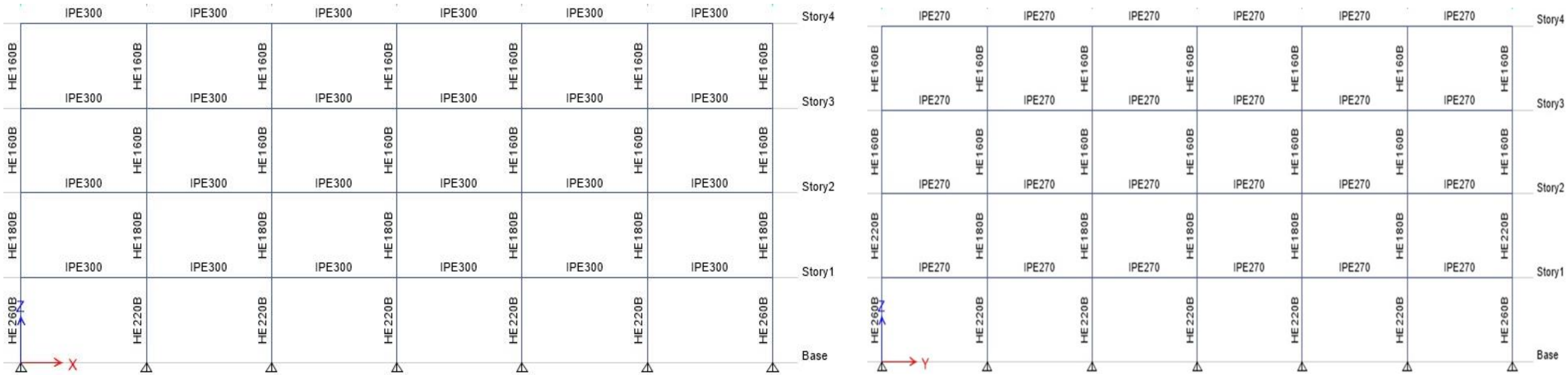


Figure A80: Design Section 4-Story Square Plan Eccentric Inverted V Braced Frame Internal Section on X and Y axis

12-Story Concentric Diagonal Braced Frame for H-plan

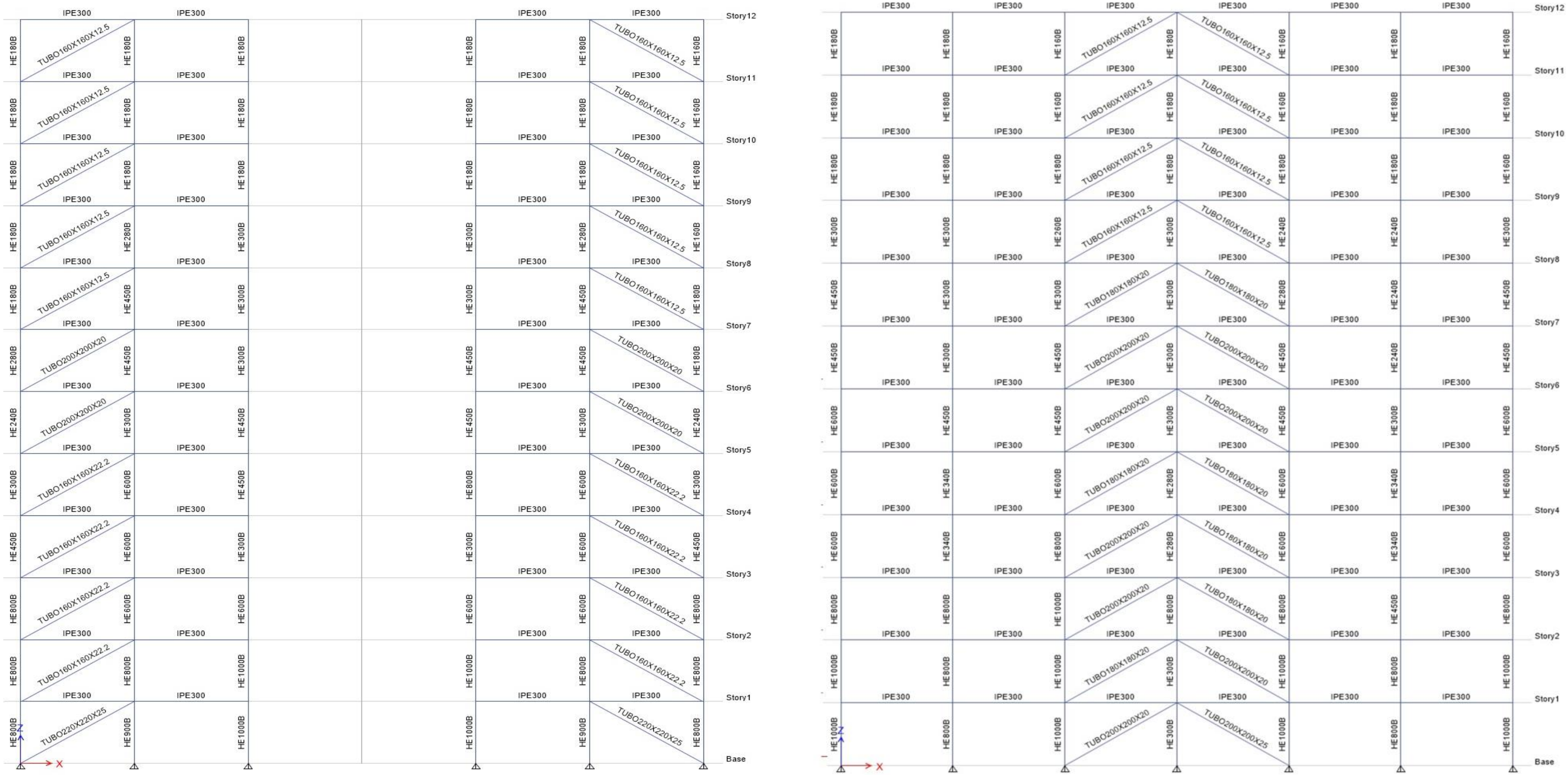


Figure A81: Design Section 12-Story H Plan Diagonal Concentric Braced Frame External Section on X-axis

Figure A82: Design Section 12-Story H Plan Diagonal Concentric Braced Frame Internal Section on X-axis

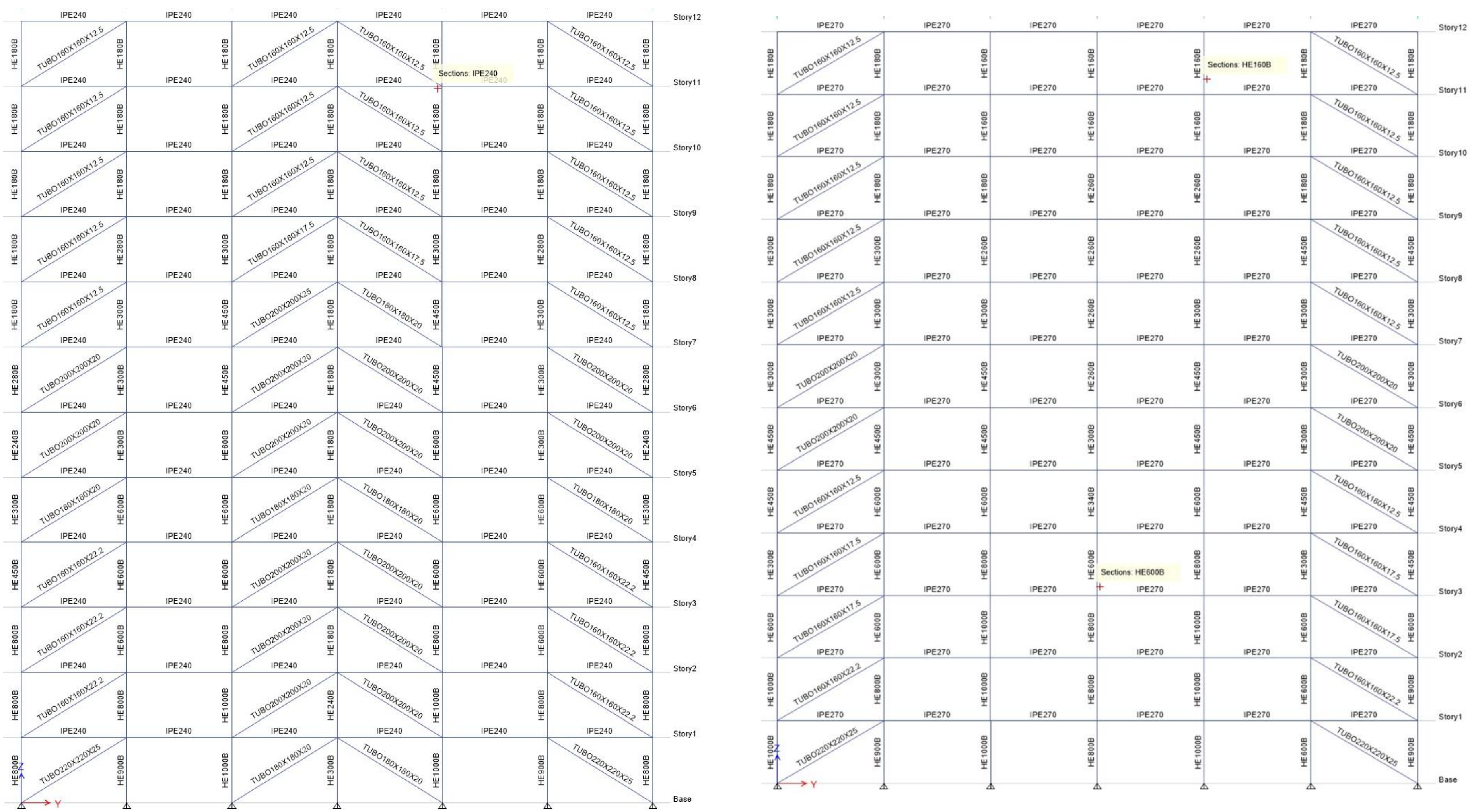


Figure A83: Design Section 12-Story H Plan Diagonal Concentric Braced Frame External Section on Y-axis

Figure A84: Design Section 12-Story H Plan Diagonal Concentric Braced Frame Internal Section on Y-axis

12-Story Concentric Diagonal Braced Frame for Square plan

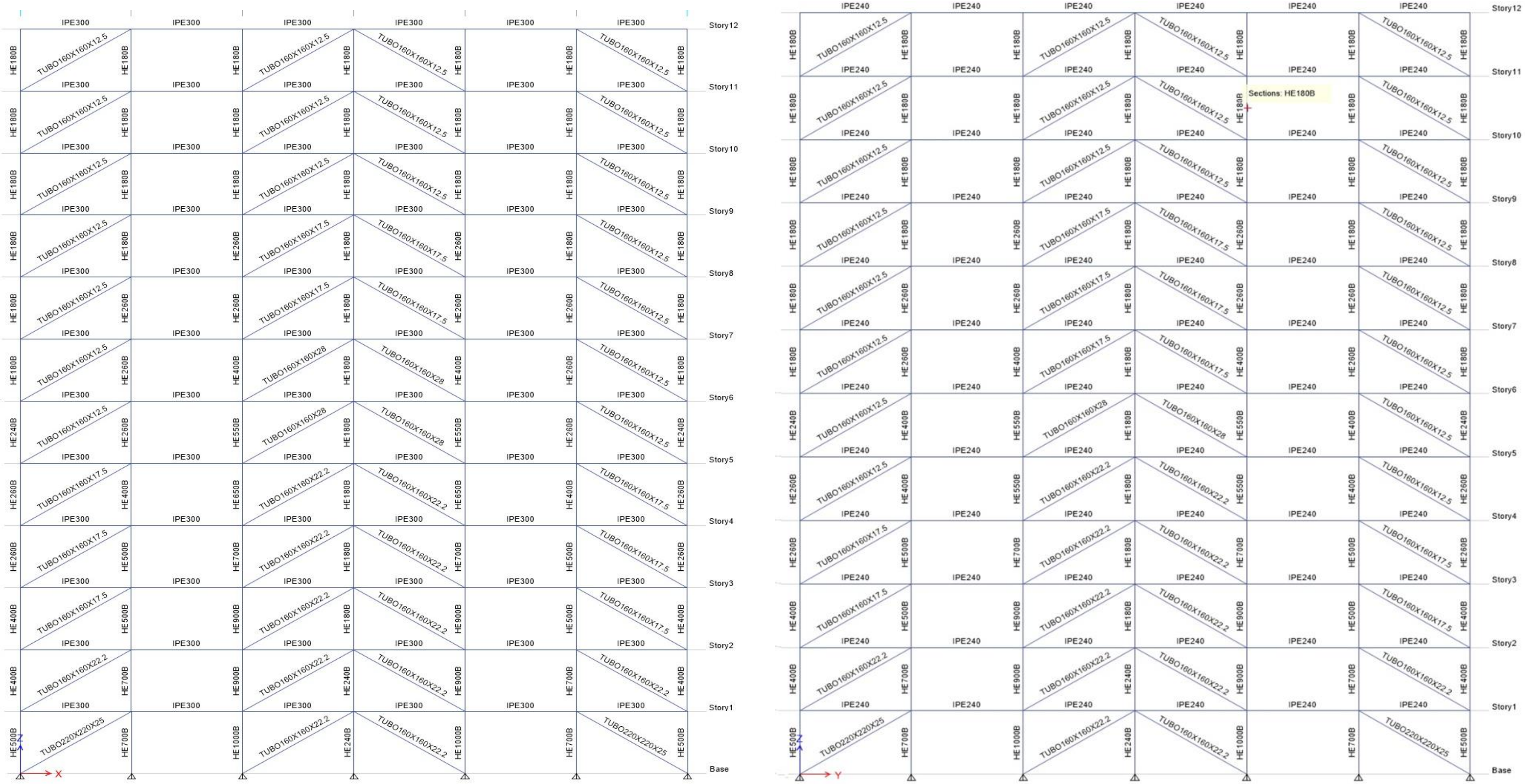
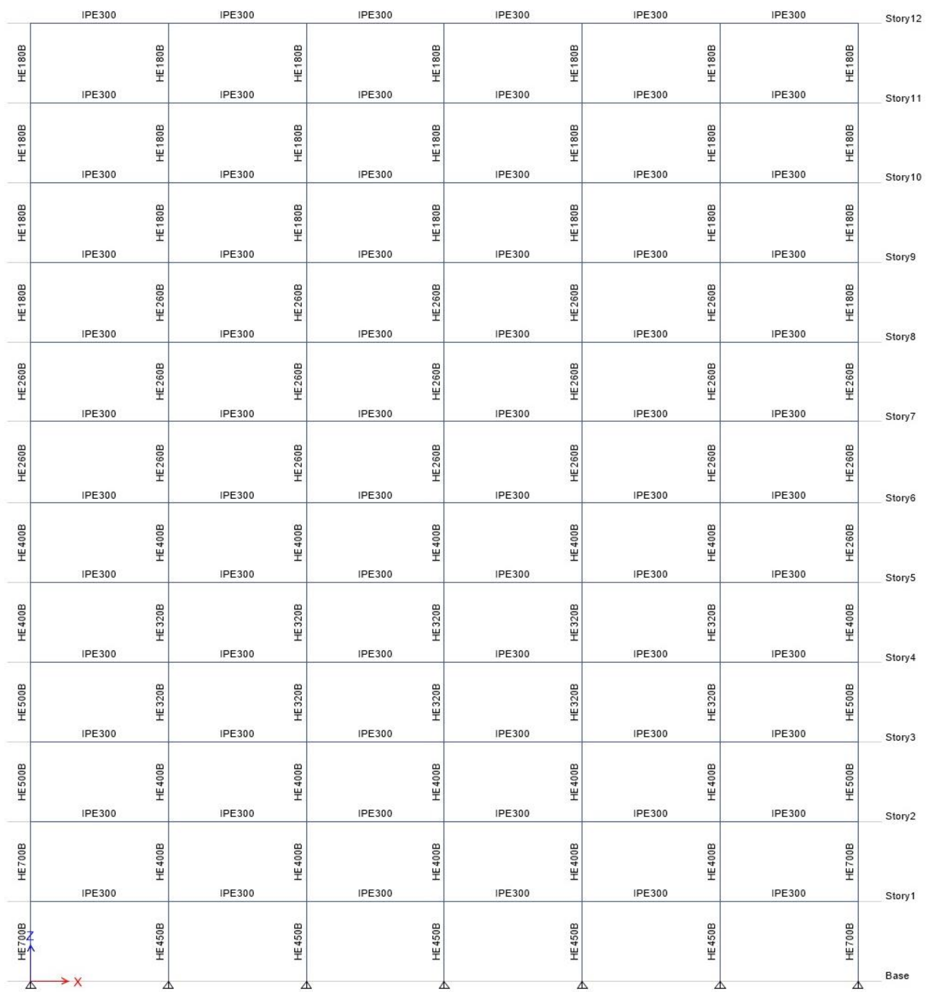


Figure A85: Design Section 12-Story Square Plan Diagonal Concentric Braced Frame External Section on X-axis



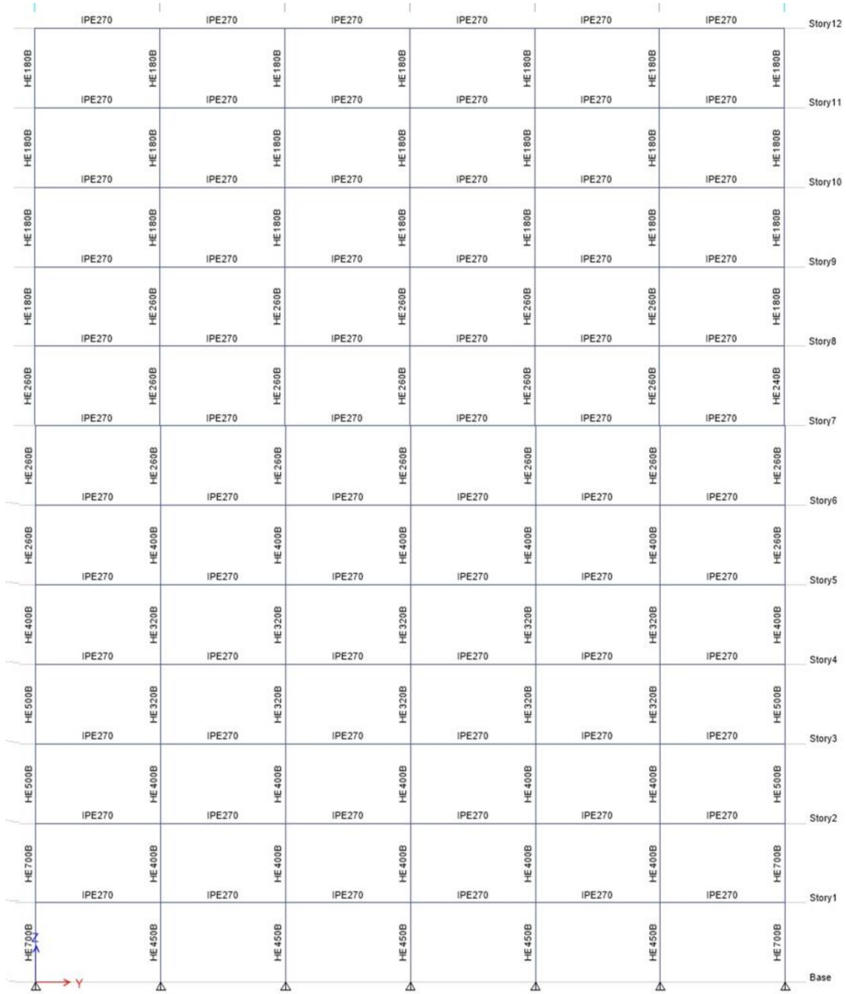


Figure A86: Design Section 12-Story Square Plan Diagonal Concentric Braced Frame External Section on Y-axis

12-Story Concentric Inverted Chevron-V Braced Frame for H-plan

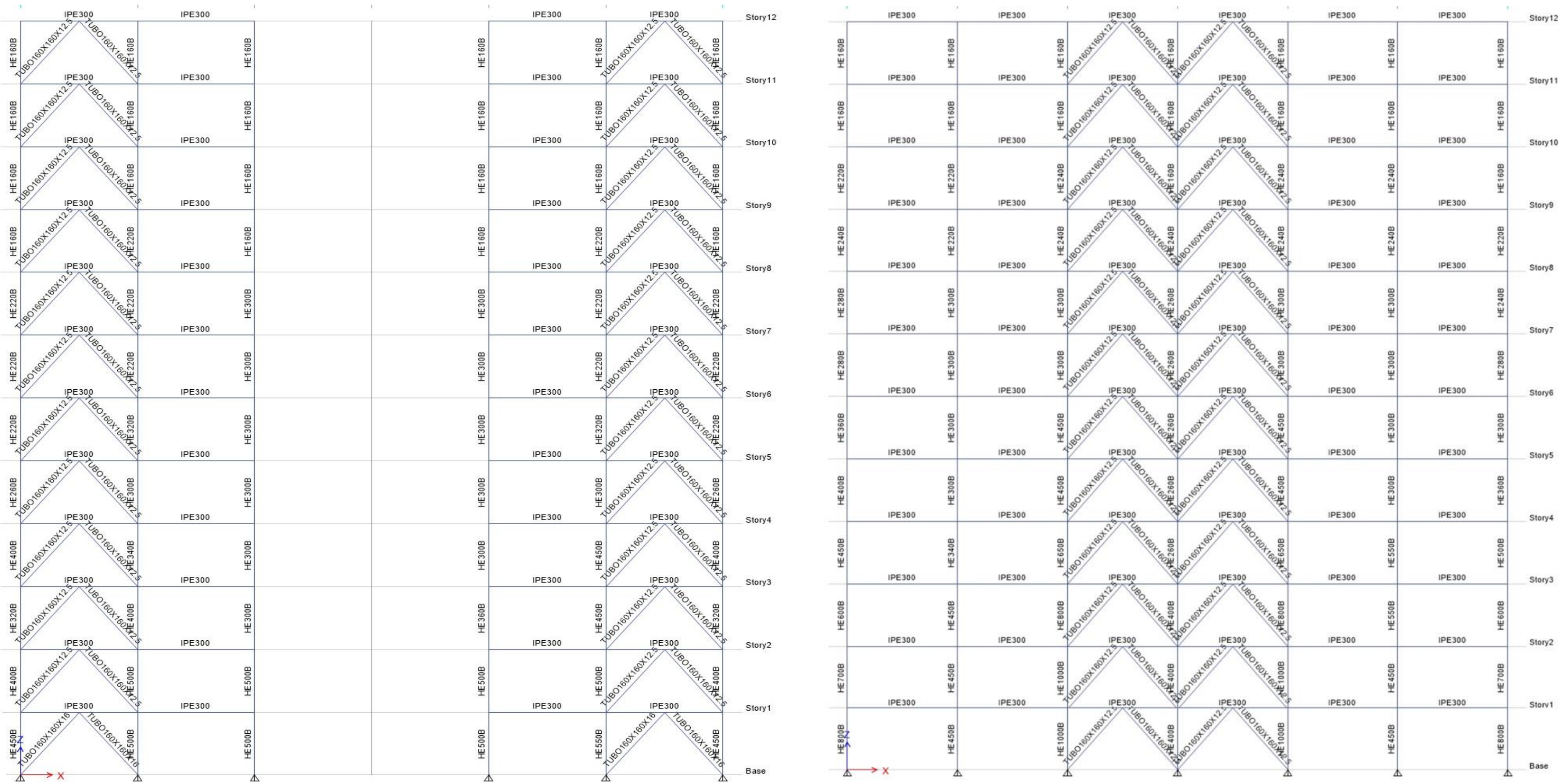
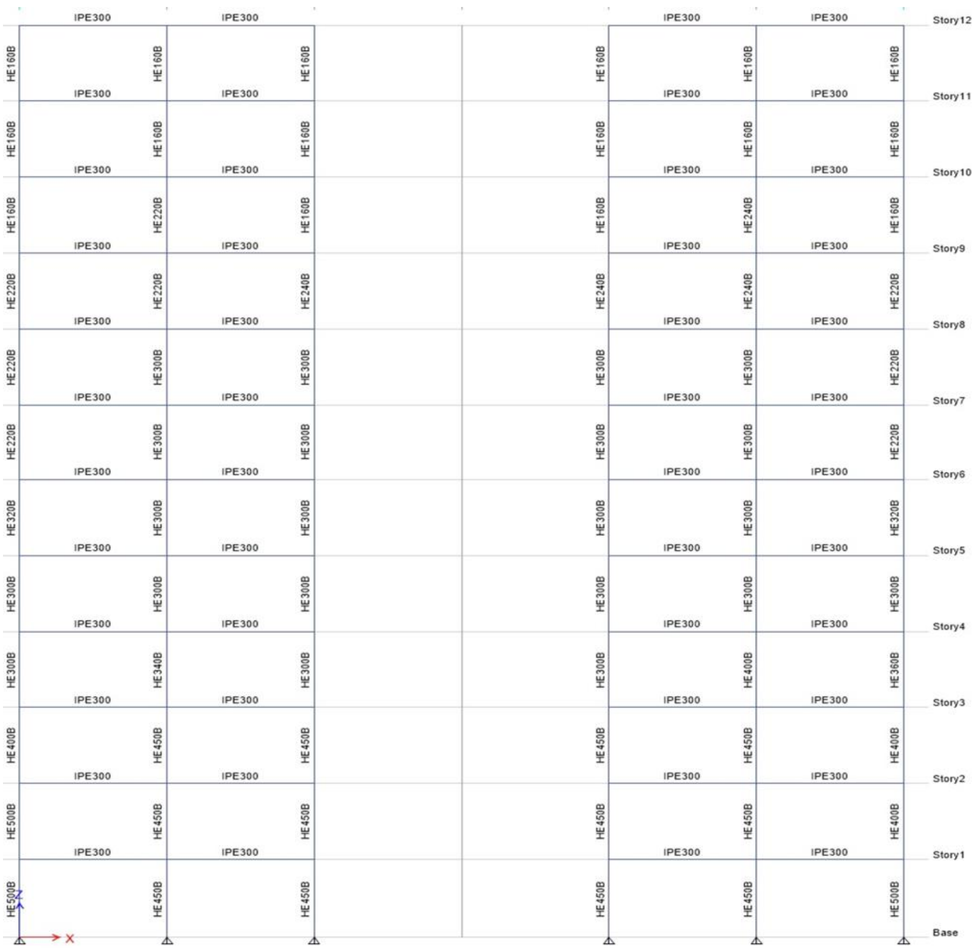


Figure A87: Design Section 12-Story H Plan Inverted V Concentric Braced Frame External Section on X-axis



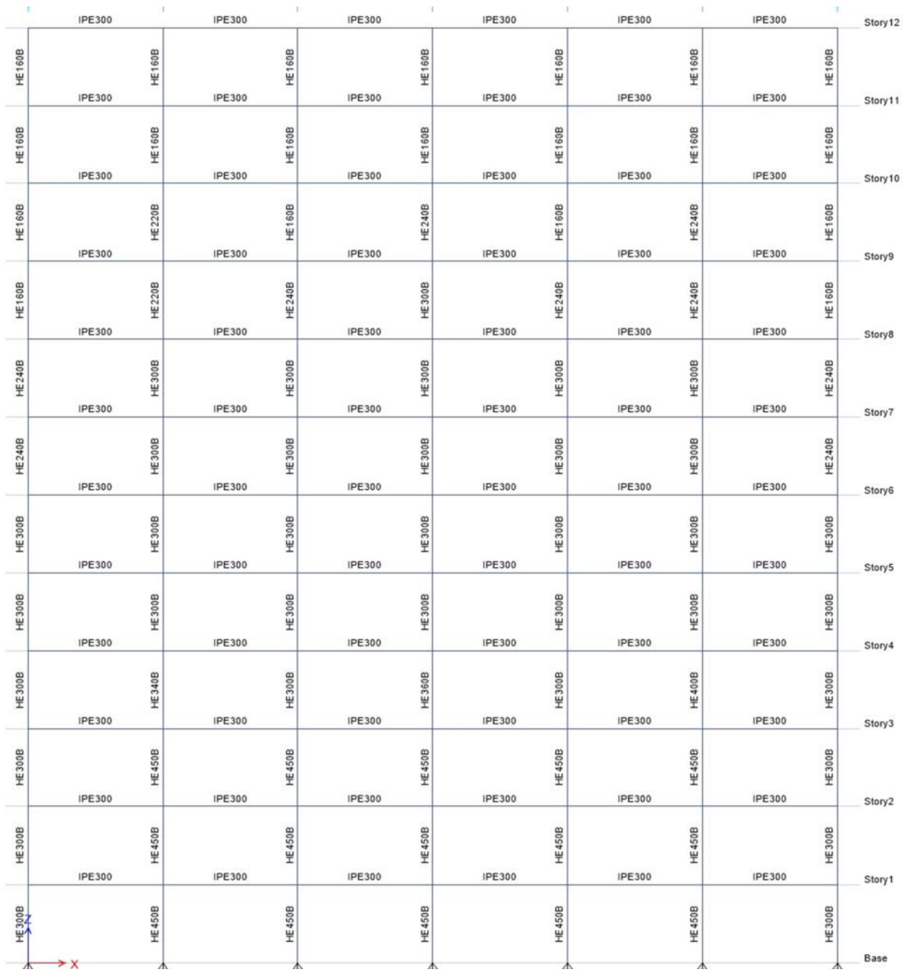


Figure A88: Design Section 12-Story H Plan Inverted V Concentric Braced Frame Internal Section on X-axis

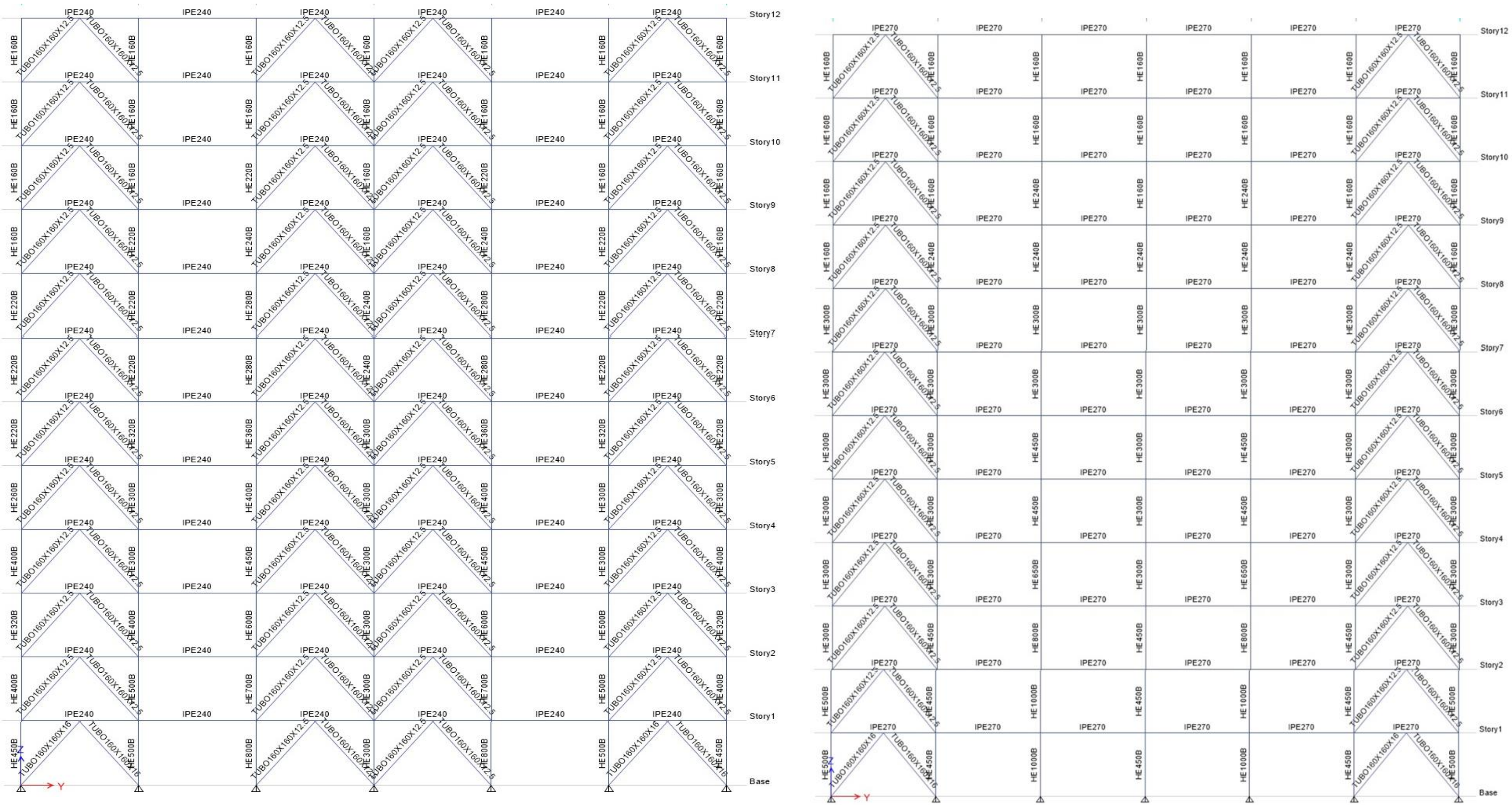


Figure A89: Design Section 12-Story H Plan Inverted V Concentric Braced Frame External Section on Y-axis

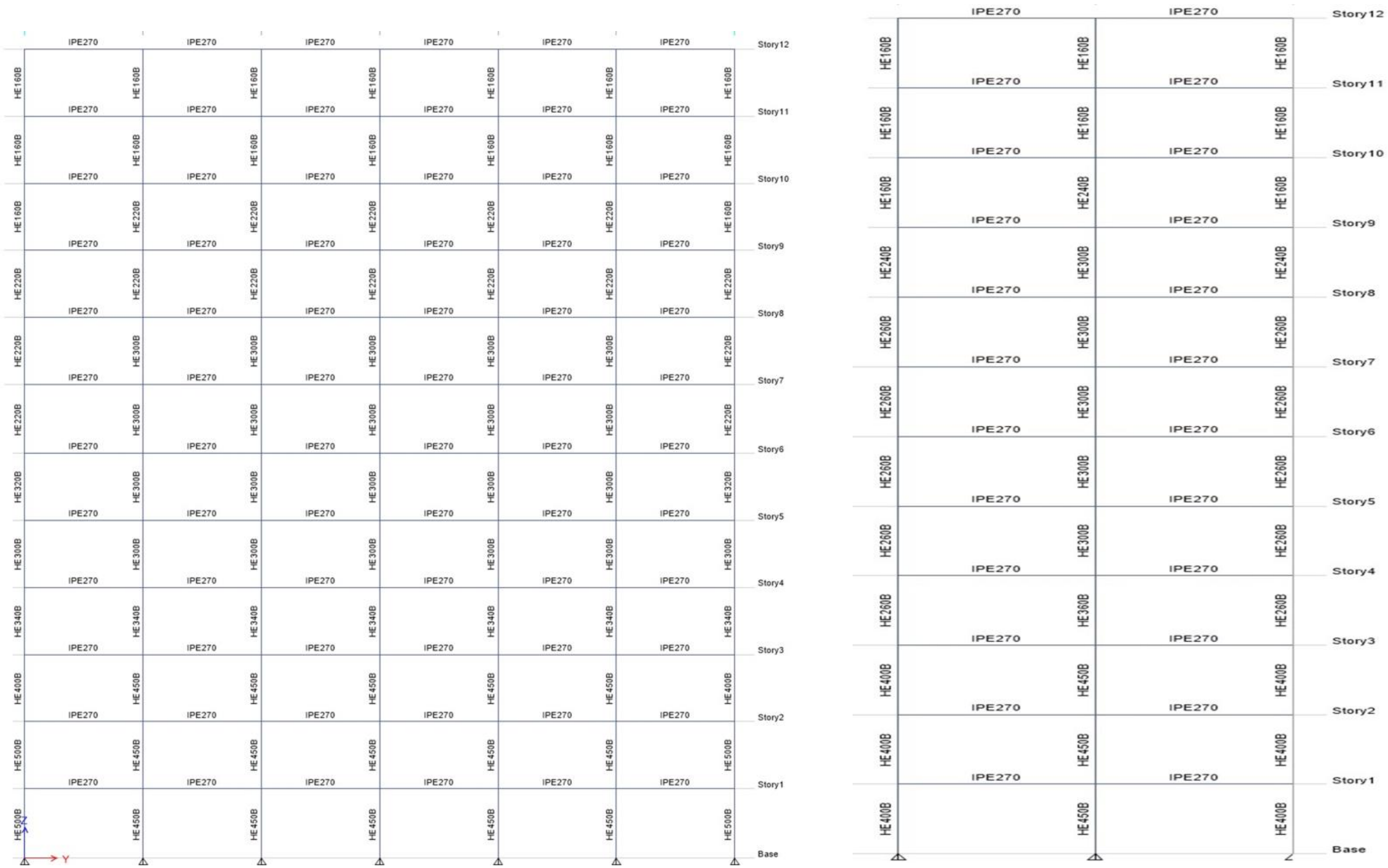


Figure A90: Design Section 12-Story H Plan Inverted V Concentric Braced Frame Internal Section on Y-axis

12-Story Concentric Inverted Chevron-V Braced Frame for Square plan

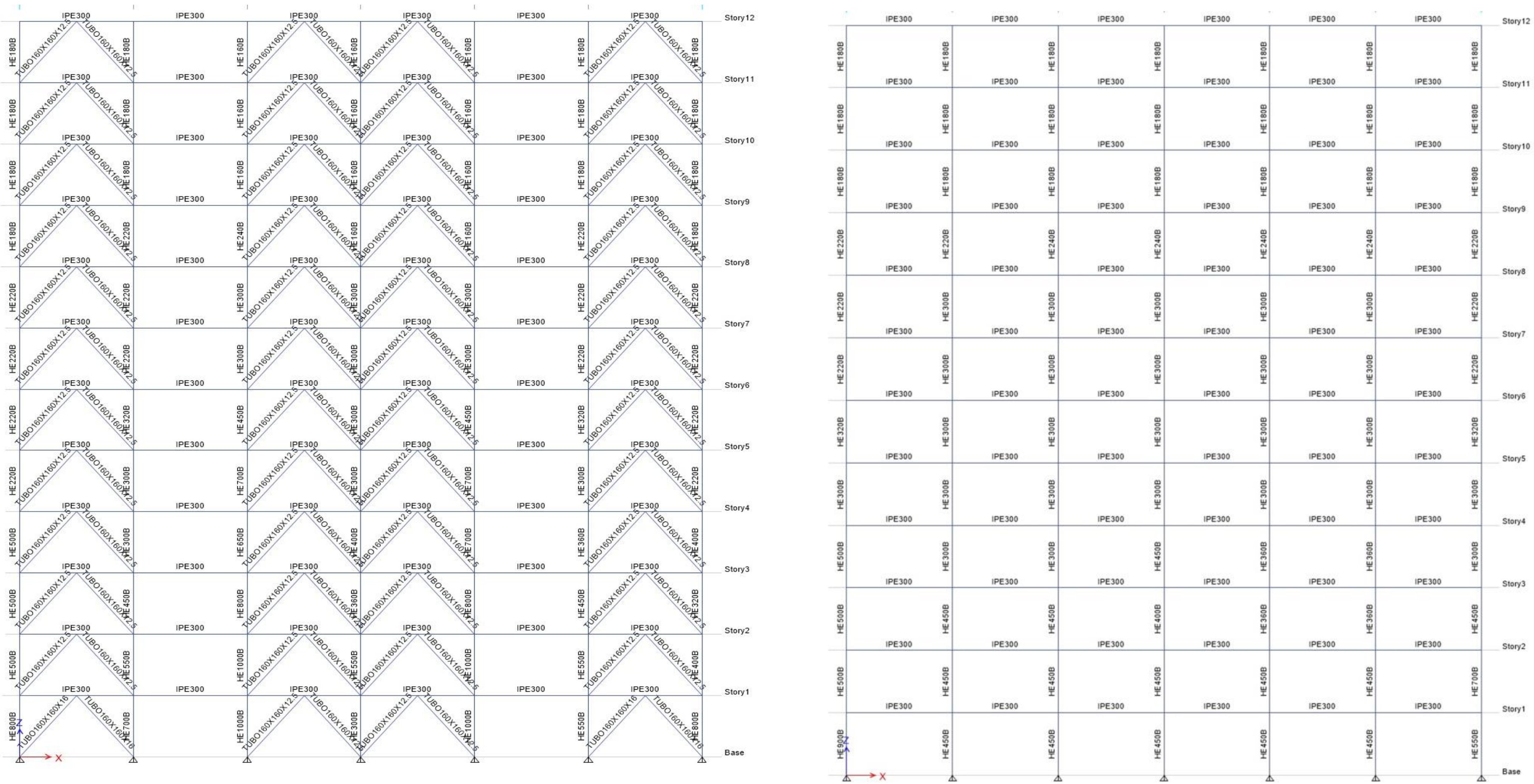


Figure A91: Design Section 12-Story Square Plan Inverted V Concentric Braced Frame External Section on X-axis

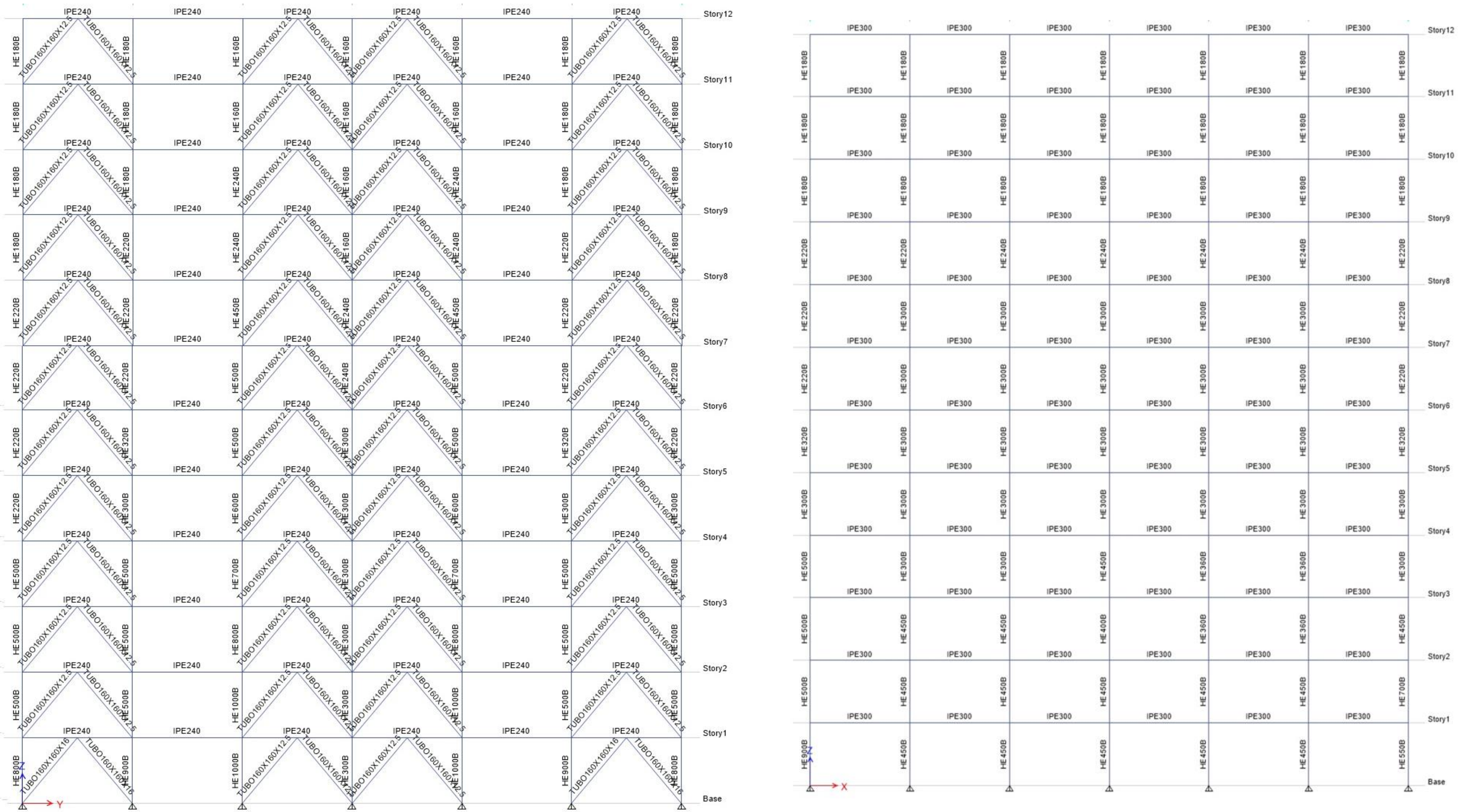


Figure A92: Design Section 12-Story Square Plan Inverted V Concentric Braced Frame External Section on Y-axis

12-Story Eccentric Diagonal Braced Frame for H-plan

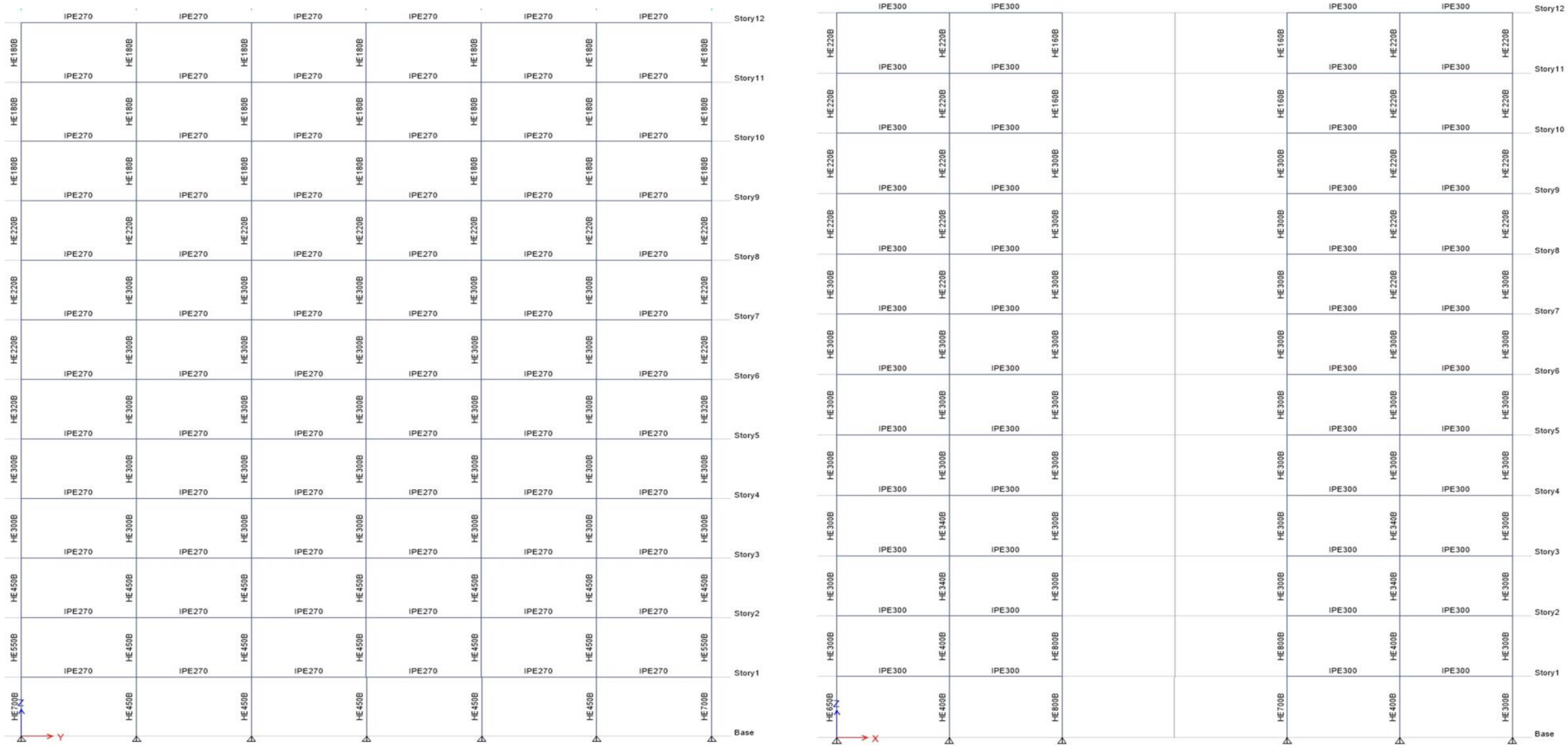


Figure A93: Design Section 12-Story H Plan Diagonal Eccentric Braced Frame Internal Section on X-axis

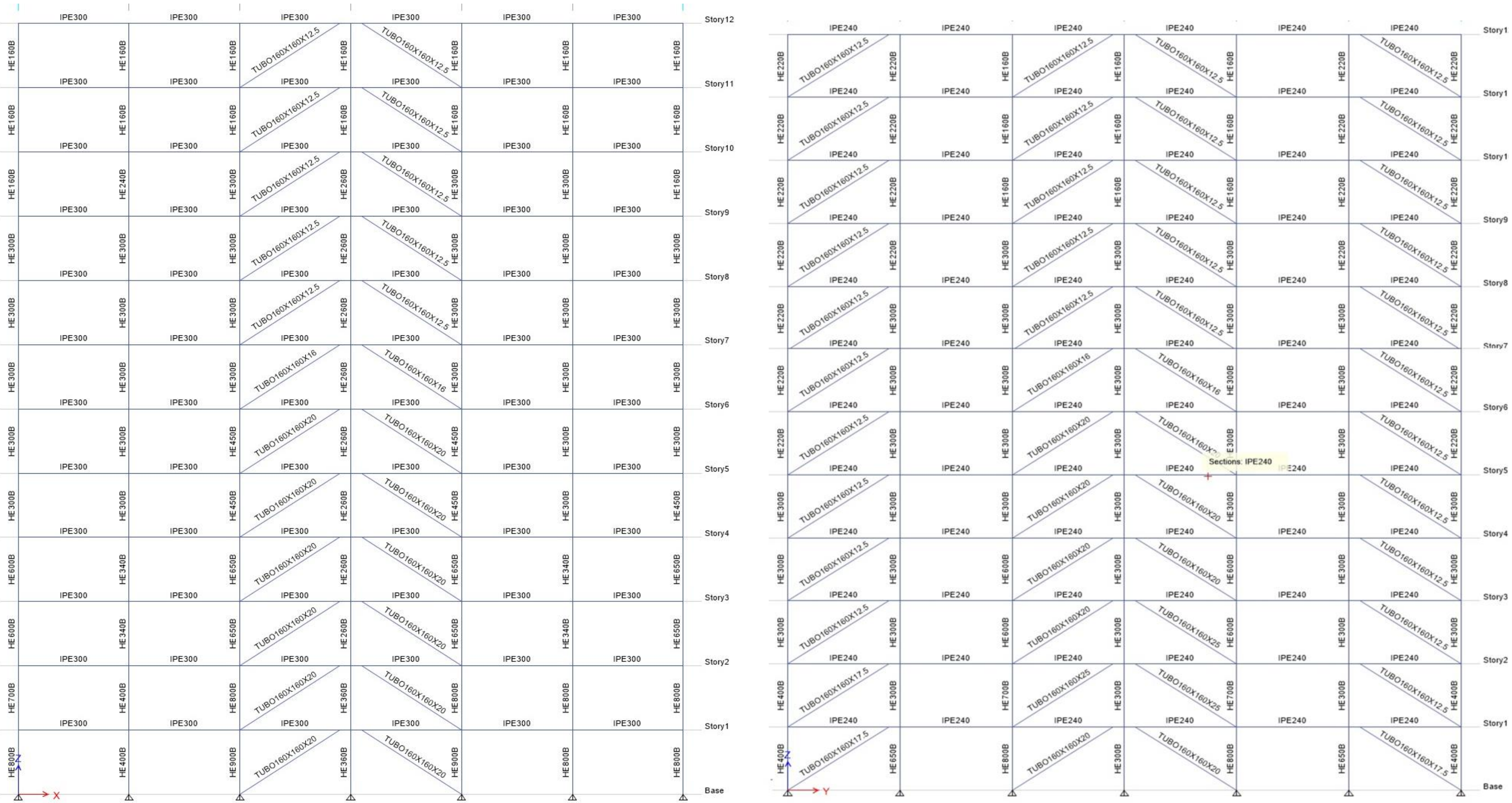


Figure A94: Design Section 12-Story H Plan Diagonal Eccentric Braced Frame External Section on X-axis

Figure A95: Design Section 12-Story H Plan Diagonal Eccentric Braced Frame Internal Section on X-axis

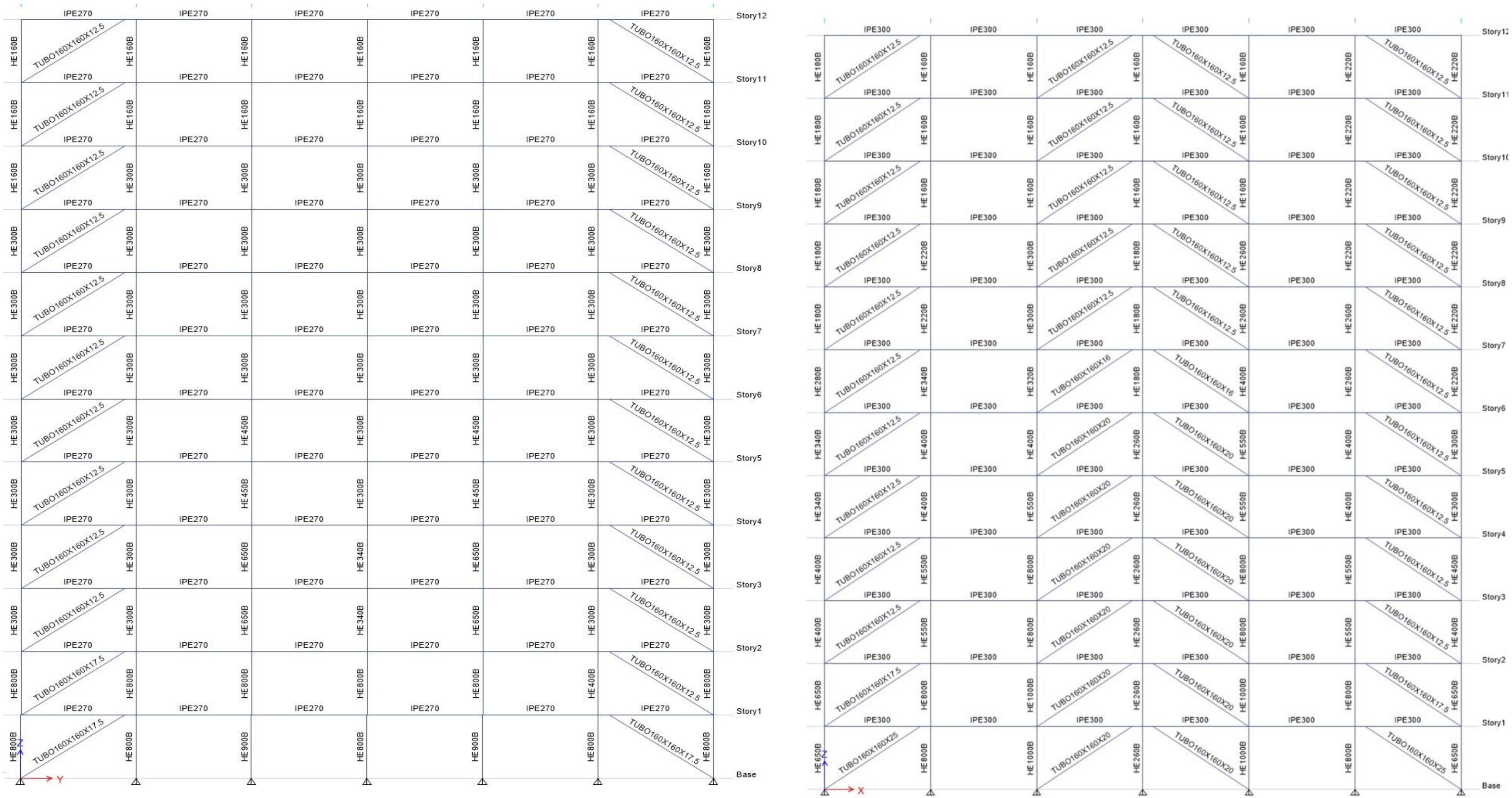


Figure A96: Design Section 12-Story H Plan Diagonal Eccentric Braced Frame External Section on X and Y axis

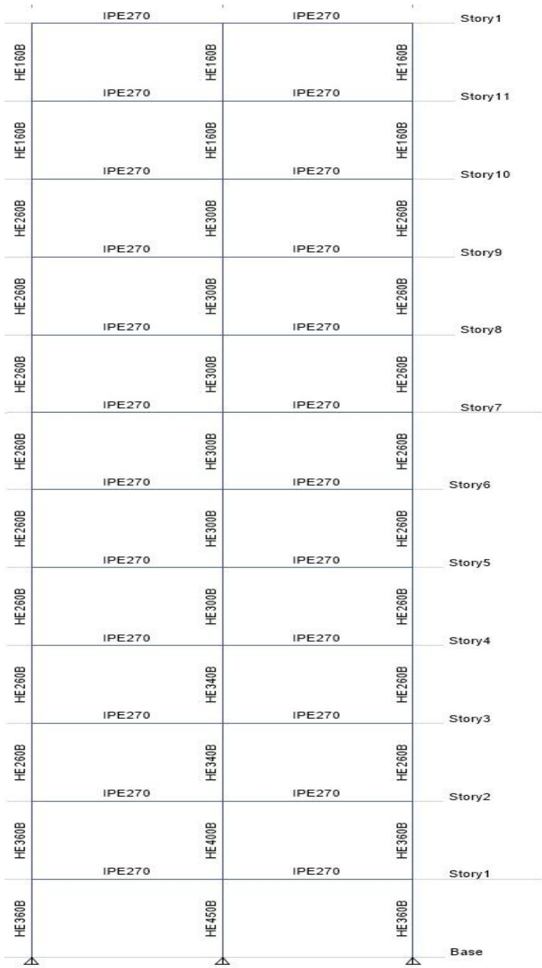


Figure A97: Design Section 12-Story H Plan Diagonal Eccentric Braced Frame Internal Section on Y-axis

12-Story Eccentric Diagonal Braced Frame for Square plan

Figure A98: Design Section 12-Story Square Plan Diagonal Eccentric Braced Frame Internal Section on X and Y axis

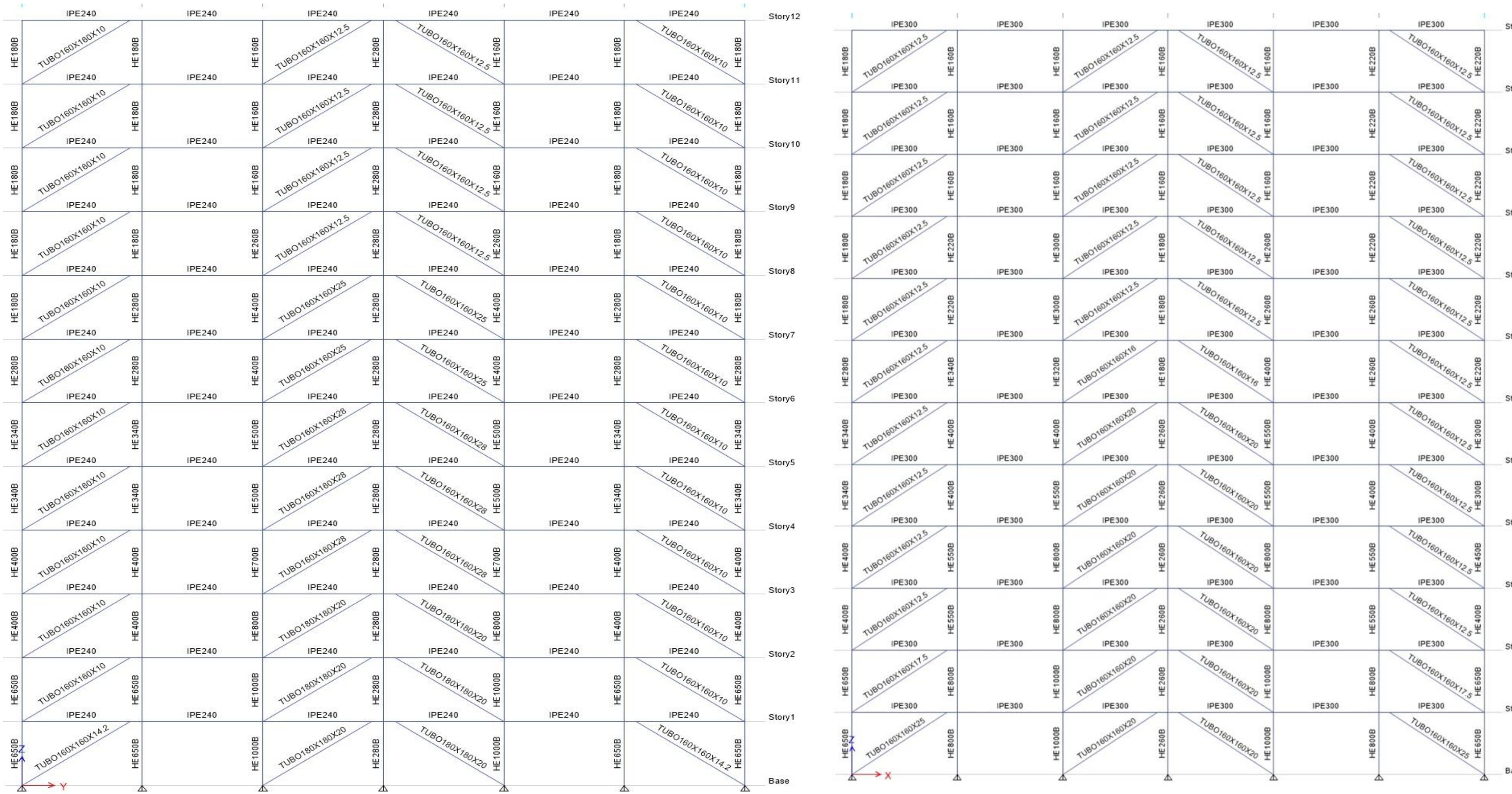
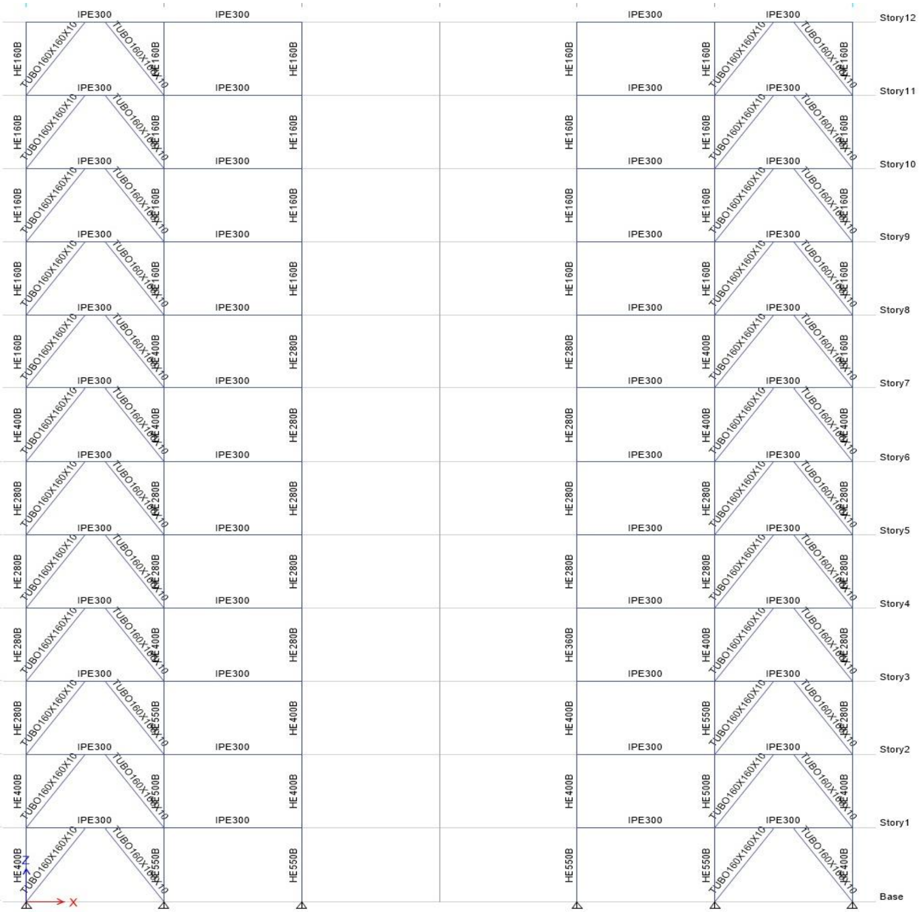


Figure A99: Design Section 12-Story Square Plan Diagonal Eccentric Braced Frame External Section on X and Y axis

12-Story Eccentric Inverted Chevron-V Braced Frame for H-plan



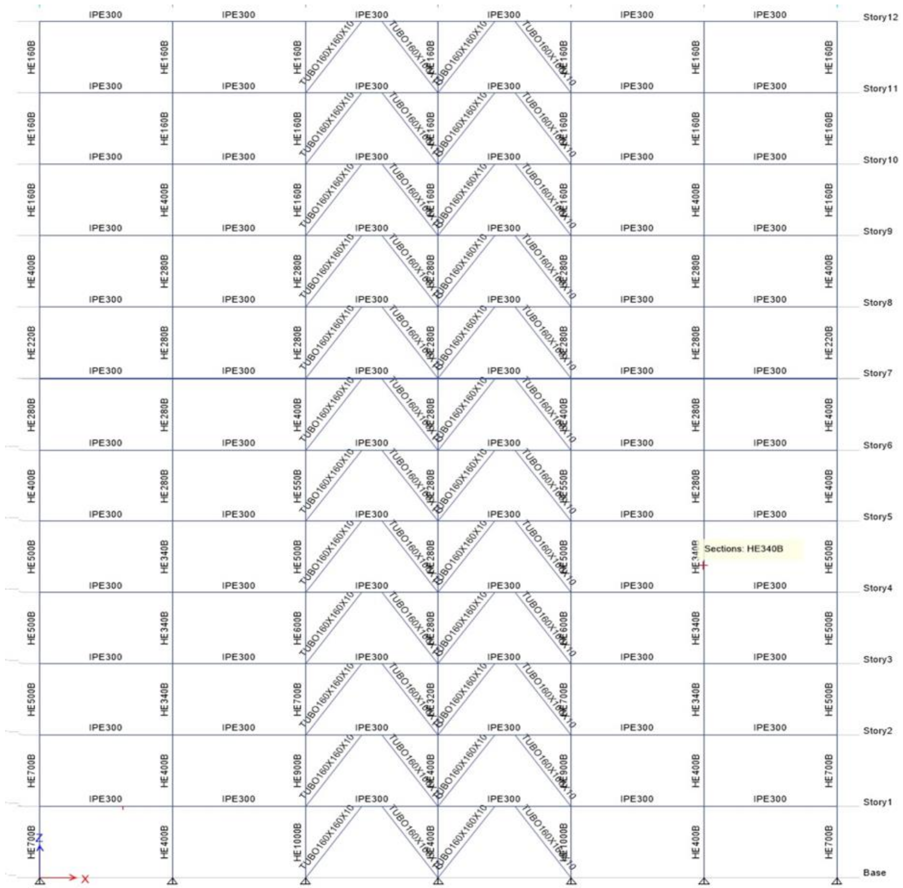


Figure A100: Design Section 12-Story H Plan Inverted √ Eccentric Braced Frame External Section on X-axis

Figure A101: Design Section 12-Story H Plan Inverted V Eccentric Braced Frame Internal Section on X-axis

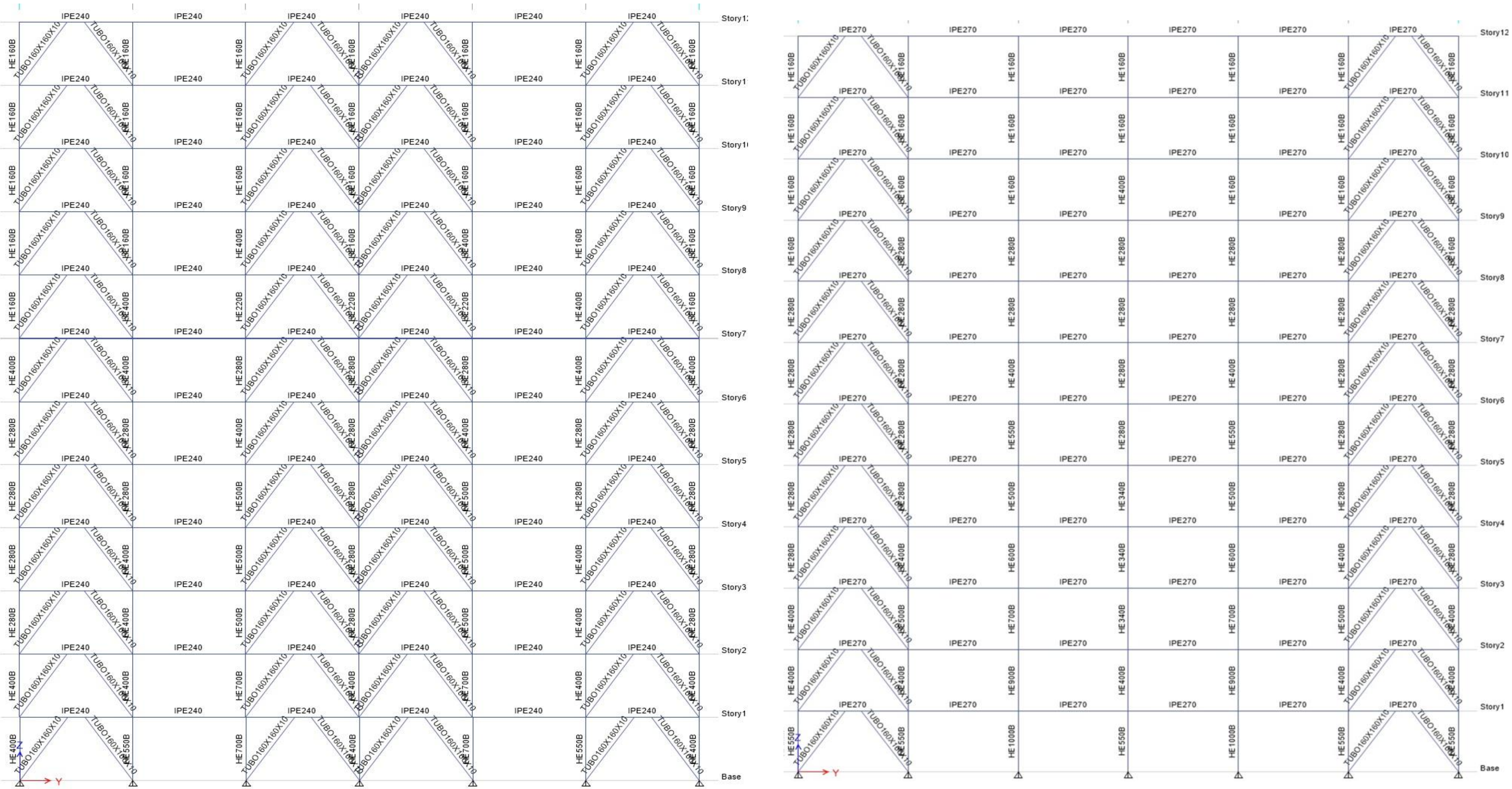


Figure A102: Design Section 12-Story H Plan Inverted V Eccentric Braced Frame External Section on Y-axis

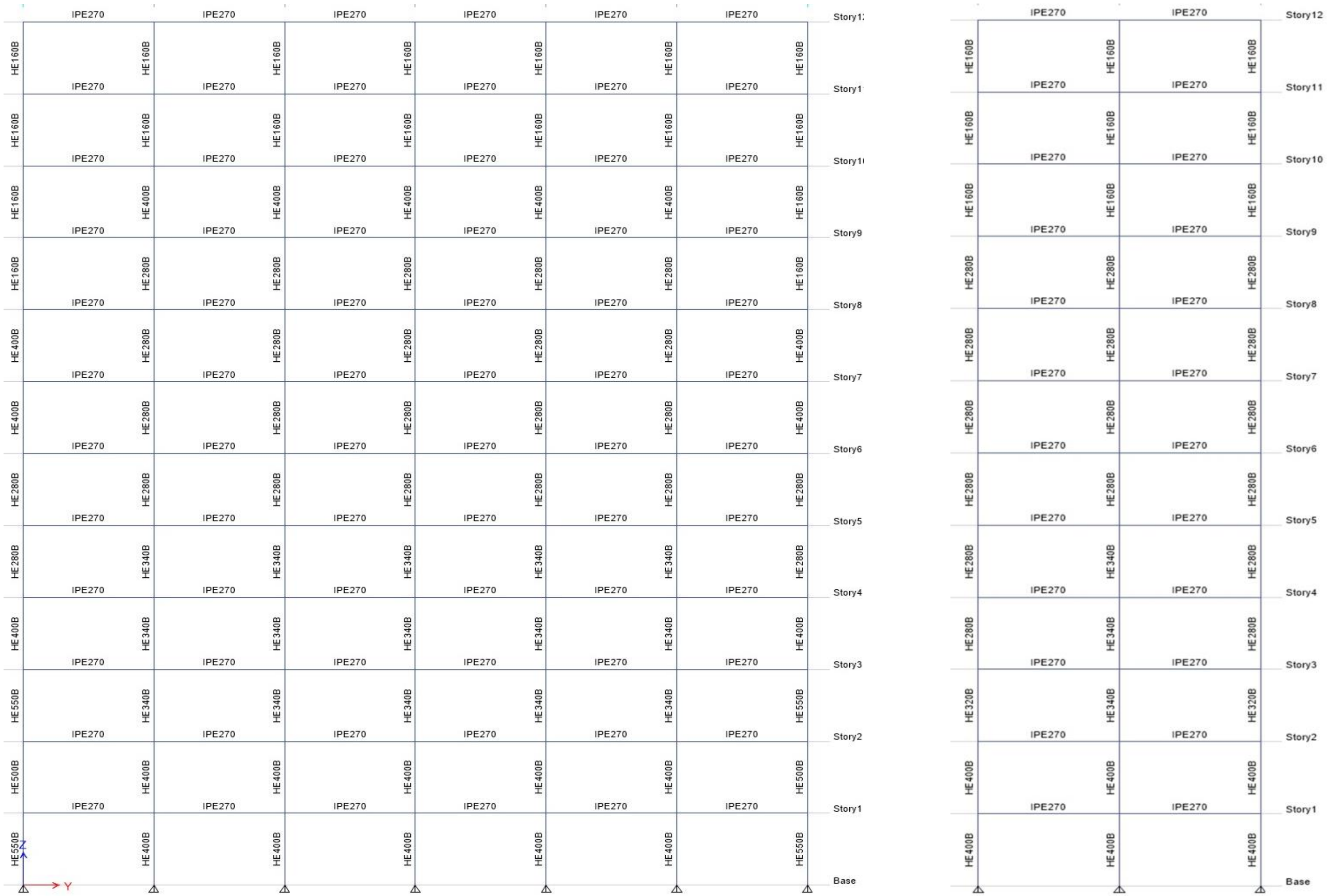


Figure A103: Design Section 12-Story H Plan Inverted V Eccentric Braced Frame Internal Section on Y-axis

12-Story Eccentric Inverted Chevron-V Braced Frame for Square plan

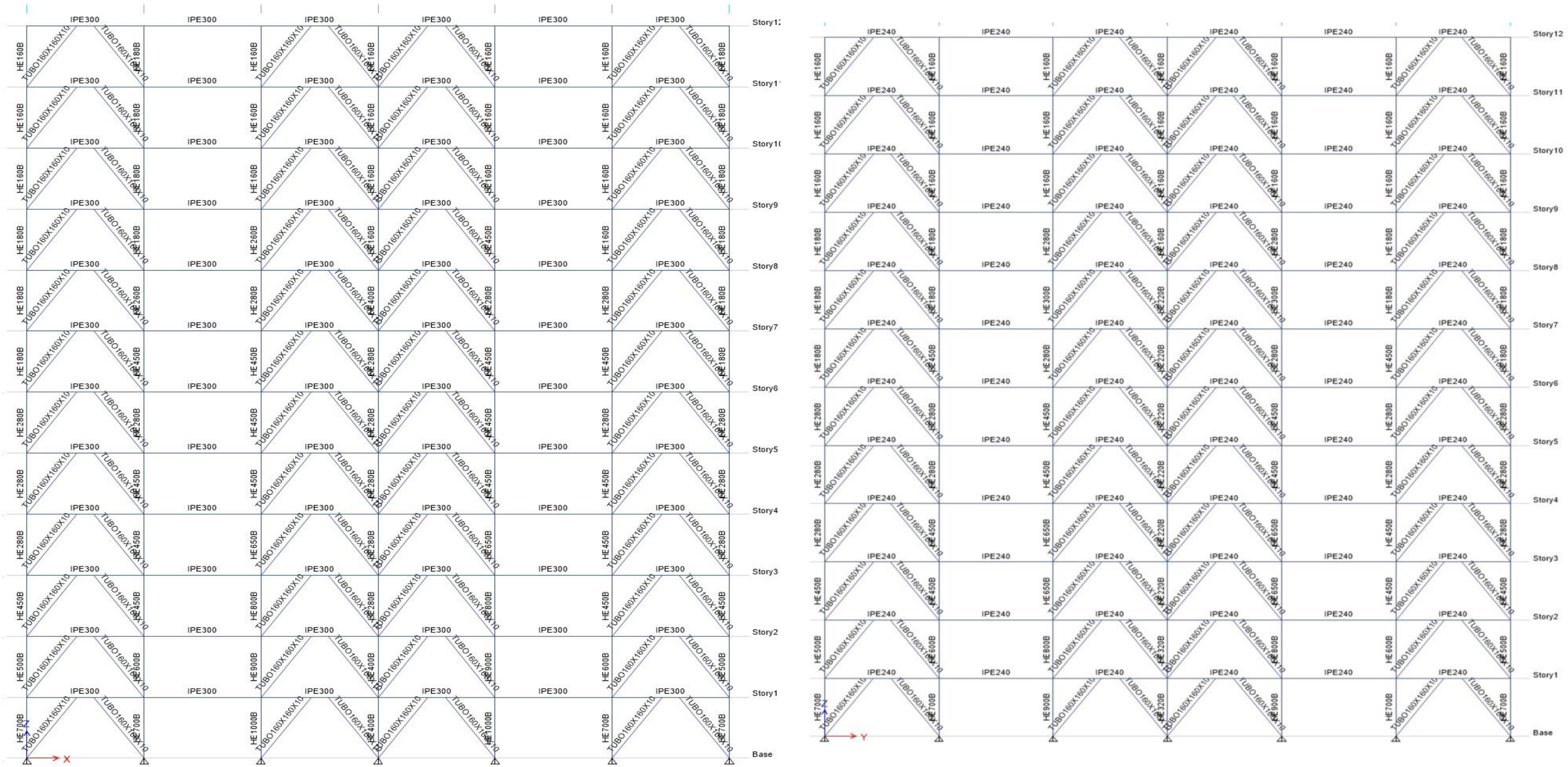


Figure A104: Design Section 12-Story Square Plan Inverted V Eccentric Braced Frame External Section on X-axis

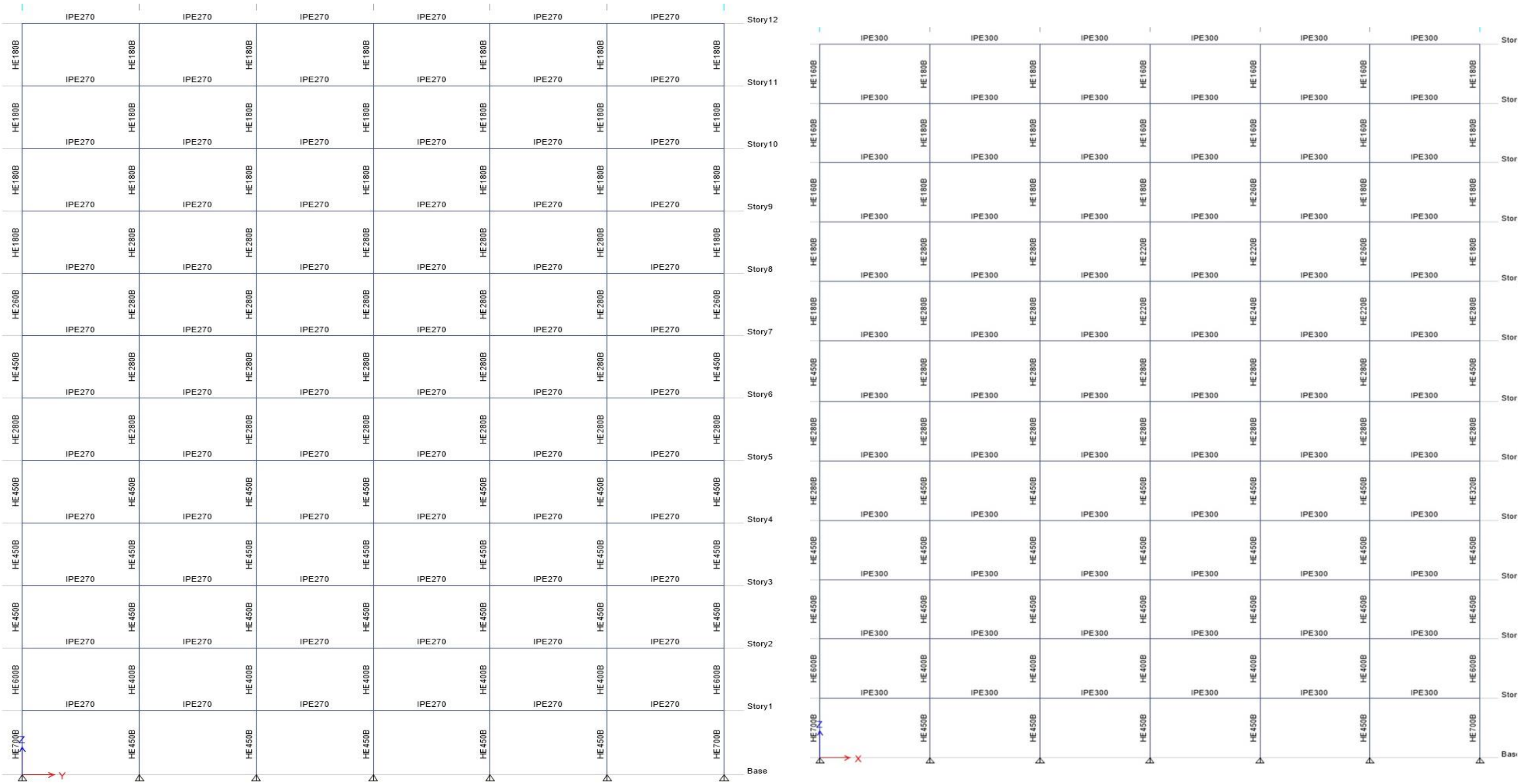


Figure A105: Design Section 12-Story Square Plan Inverted V Eccentric Braced Frame Internal Section on X and Y axis