ADDIS ABABA UNIVERSITY
SCHOOL OF GRADUATE STUDIES

INSTITUTE OF TECHNOLOGY
DEPARTMENT OF CIVIL ENGINEERING

COMPARATIVE EVALUATION OF CONCENTRICALLY BRACING SYSTEM FOR LATERAL LOADS ON MEDIUM RISE STEEL BUILDING STRUCTURES

A thesis submitted to the School of Graduate Studies in Partial fulfillment of the Requirements for the Degree of Master of Science in Civil Engineering (Structural)

By

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Advisor: **Dr. Asnake Adamu**

February, 2015

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Approved by Board of Examiners

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DECLARATION

I, the undersigned, declare that this thesis is my original work and has not been presented for a degree in any other university. All sources of materials used for the thesis have been duly acknowledged.

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ABSTRACT

Lateral load resisting systems are structural elements providing basic lateral strength and stiffness, without which the structure would be laterally unstable. The unstable nature of structures is solved by appropriate provision of bracings systems. In this study, steel bracing is provided at the periphery of four, six and ten story buildings to determine the efficiencies of structural systems under five bracing types is considered. These bracings are V, inverted V, combination of V and inverted V which forms X bracing in two successive stories, K, mirror of K bracings.

The buildings with incorporated bracing systems are analyzed using ETABS analysis software as per Eurocode 3 and 8. Then, the lateral displacement value under each of the bracing type is evaluated. The result shows that chevron (inverted V) bracing gives lesser lateral displacement. This indicates that this bracing gives better performance for lateral load resistance.

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ABBREVIATIONS AND NOTATIONS

AISC- American institute of steel construction

ASCE- American society of civil engineering

CBFs - Concentrically braced frames

CP -Collapse Prevention

EBFs- Eccentrically braced frames

EBCS- Ethiopian building code of standards

EN- European standard

ETABS- Extended Three-Dimensional Analysis of Building Systems

IO-Immediate Occupancy,

LFRS- lateral force resisting systems

LL- Live load

LS- Life Safety

MRFs- Moment resisting frames

NA- National annex

SCBF-Special concentrically braced frames

ULS- Ultimate limit state

SLS- Serviceability limit state

f_y- Nominal yield strength

f_u- Ultimate tensile strength

H- Height of the building from basement

K- Effective length factor

q- Structural behavior factor

R-Response modification factor

T_1-Fundamental period of vibration

1. INTRODUCTION

1.1 Background of the Study

Structural systems of medium and tall buildings need resistance mechanism especially in areas of high seismic regions to sustain its stability without sudden collapse. Bracing structures are the most widely utilized in steel buildings to increase the resistance of the overall structural systems. But the resistance capacities of bracing are different for different orientation of bracing systems. Previous studies said that X bracing performs better than any other concentrically bracing type. But their criteria of measurement are not stipulated clearly. To minimize such set back, this study considers the weight of the bracing assumed to be a constant parameter for all selected bracing type.

To compare the efficiencies of bracings, five types of steel bracings are selected in this study. These are:

- ✓ V-bracings,
- ✓ Inverted V (chevron) bracings,
- ✓ Combination of V and chevron bracing which forms X bracing in two stories of the buildings,
- ✓ K bracings, and
- ✓ Mirror of K bracings.

Each of these five bracing are provided on each of the three types of buildings consisting of:

- ✓ 4 story,
- ✓ 6 story, and
- ✓ 10 story steel frame building structures.

Then these buildings are modeled and analyzed using finite element software's, ETABs version 9.

1.2 Objective and Scope of the Study

The main objective of this thesis is to compare and evaluate the effectiveness of bracing systems of steel structure for medium steel buildings under earthquake lateral loads. It is to select the most efficient earthquake lateral loads resistant bracing types which gives the minimum lateral displacements under the selected groups of bracings types.

The study considers on concentrically type of bracing systems having a structural resistance capacity for lateral loads through a vertical concentric truss system. The axes of the members are made to align concentrically at the joints.

This study is limited to only on comparison of concentrically bracing types with different types of orientation (V, inverted V, K and mirror of K-bracings). While comparison is made the study doesn't consider any aesthetical effects of the bracing for provision of doors and windows.

1.3 Content of the Thesis

The study comprises a steel structure of four, six and ten story is analyzed with five different bracing type namely; V-bracing, inverted V-bracing (chevron bracing), K-bracing and mirror of K-bracing. The general classification of bracings based on their geometrical arrangements is grouped in to two (i.e. concentrically bracing and eccentrically bracing). In this study, the bracing considered are concentrical types of classification.

The story height and bay width of the building is assumed to be equal with 4m dimensions for equal treatment of bracing which do not alter behavior of bracings. The weight of each bracing is assumed to be equal which is constant parameter in this work.

For analysis of these steel building, Eurocode 3- Design of steel structures; and Eurocode 8-Design of structures for earthquake resistance, is used. These codes are direct similarity to that of the new EBCS 3 and EBCS 8 of 2013 version.

The thesis is organized in different sections which are arranged as follows:

- Section one deals with an introductory part which include background, objective and contents of the thesis.
- Section two briefly reviews theoretical background of steel bracing systems, classifications, principles and design approaches are considered.
- Section three discusses about the modeling software and loading consideration in the frame geometry is highlighted.
- Section four tells about the analysis of structural systems for the given loading under consideration.
- Section five presents comparison and discussion for lateral displacement for each of the bracing type is made using Microsoft excel programme with the help of graphs.
- Finally, conclusions drawn and recommendation is forwarded to show research areas for the next researchers.

2. LITERATURE REVIEW

2.1 Behavior of Steel Structures

Steel is a versatile construction material widely used in the construction of high rise structures, bridges, airport hangers, shopping complex, rope car pylons, recreational structures, steel arch, etc. It has high strength and ductility, which is the primary requirement under seismic action because the structure has to absorb the vibration energy imparted to it during shaking of ground. Duggal (2007) discussed on the large ductility and high strength to weight ratio of structural steel [1], which make it an ideal material for earthquake resistance.

The large ductility and the high strength-to-weight ratio of structural steel make it an ideal material for earthquake resistance. In general, steel buildings are more flexible than RCC buildings, but also they display more lateral displacement than RCC buildings which can be controlled by providing lateral support mechanism like bracing structures. Structural planning of steel buildings should conform to that the beams yield prior to the columns, and the strength of a connection should be greater than the strength of beams and columns framing into the connection members and connections should guarantee high strength, ductility, and energy dissipation capacity, and an excessive lateral sway should be avoided. Multi-storey buildings are generally constructed in steel as framed structures. A ductile frame can undergo important inelastic deformations, localized in the neighborhood of sections with maximum bending moment. This eventually leads to the formation and rotation of plastic hinges and redistribution of plastic moments, allowing the structure to resist higher loads than those predicted by the elastic analysis [2]. Un-braced steel buildings are ductile and possess large energy dissipation capacity but tend to deform greatly, causing serious damage to non-structural elements during small to medium-size earthquakes. Braced frames can resist large amounts of lateral forces and have reduced lateral deflection and thus reduced P-Δ effect. However, a uniform distribution of bracing throughout the structure is desirable.

2.2 Causes and Failure Modes of Steel Structures

Although steel is highly ductile, inelastic ductility is not necessarily retained in the finished structure. Hence, care must be taken during design and construction to avoid losing this property. Considerable care is also needed to check failures due to instability and brittle fracture to ensure the development of full ductility and energy dissipation capacity under earthquake loading.

The causes of instability are [2]:

(i) Local buckling of plate elements (e.g., web, flange, etc.) with large width to-thickness ratios: A steel member containing plate elements with a large width-to thickness ratio is unable to reach its yield strength, because of prior local buckling. Even if the yield strength is attained, ductility will be inadequate. Under cyclic loading, it is observed that strength and ductility decrease with increasing width-to-thickness ratio, and local buckling of web causes further degradation.

(ii) Flexural buckling of long columns and braces: Long columns may fail by buckling. This mode of instability is sudden and can occur when the axial load in a column may never reach the yield. Even a small lateral force in such condition will produce a substantial deflection leading to instability and the phenomenon is called flexural buckling. The capacity of slender columns is, therefore, limited by the stiffness of the member rather than the strength of the material. The lateral stiffness of the frames, therefore, is increased by bracing the frames. However, buckling of braces is a potential source of instability of steel frames. Steel bracing dissipate considerable energy by yielding under tension but buckle without much energy dissipation in compression. Therefore, the energy dissipation capacity of concentrically braced frames is marked less, due to buckling of braces than that of the moment frames.

(iii) Lateral-torsional buckling of beams: During moderate to strong shaking of the ground, additional forces are developed in various members of a structure. For a beam loaded in flexure, the load bearing the side (generally the top) carries the load in compression, whereas the non-load bearing side (generally the bottom) will be in tension. If the beam is not supported in the opposite direction of bending, and the flexural load increases to a critical limit, the beam will fail due to local buckling on the compression side in wide-flange sections designed for flexure only. If the top flange buckles laterally, the rest of the section will twist resulting in a failure mode known as lateral-torsional buckling.

(iv) P-Δ effects in frames subjected to large vertical loads: If the lateral stiffness is inadequately high, the building as a whole, or one or more stories, can fail due to the P-Δ effect. This is because of the secondary effect on shears and moments of the frame members, due to the action of the vertical loads, which interact with the lateral displacement of the building resulting from seismic forces.

(v) Uplift of braced frames: Earthquakes have a vertical component of movement in addition to the traditionally considered horizontal effects. The stresses produced due to vertical motion are generally considered not to be significant to cause instability. However, due to the horizontal component of movement, the overturning moments produce additional longitudinal stresses in walls and columns and additional upward (uplifting) and downward (thrust) forces in foundations causing instability.

(vi) Connection failure: The failures of bolted and welded connections are to be avoided.

The causes of brittle failure in steel buildings are that brittle failure is more frequent in welded steel structures, particularly, those that are fillet welded, than it is in structures connected by mechanical fasteners. This is due to a combination of possible weld defects, high residual stresses, stress concentration, which reduce the possibility of crack arrest, tension failure at net sections of bolted or riveted connection, and Lamellar tearing of plates in which the through-thickness strain due to weld metal shrinkage is large and highly restrained.

It is evident that the main objectives to achieve adequate performance of steel buildings are: the use of sufficiently ductile steel, and the ductile design and fabrication of framed members and connections [2]. All frame instability, especially the excessive sway leading to higher levels of damage to non-structural components and to higher secondary stresses due to P-Δ effect, should be avoided; all forms of brittle failures should be avoided; and also failure mechanism should provide maximum redundancy, i.e., the possibility of failure by local collapse should be avoided. All portions of the building should be tied well together.

2.3 Lateral Load Resisting Systems

The resistance of tall buildings to wind as well as to earthquake is the main determinant in the formulation of new structural systems that evolve by the continuous efforts of structural engineers to increase building height while keeping the deflection within acceptable limits and minimizing the amount of materials. Thanks to the sophisticated computer technology, modern materials and innovative structural concepts, structural systems have gone beyond the traditional frame construction of the home insurance building and have allowed skyscrapers to grow to the greater heights now a day.

Most of the tallest buildings in the world have steel structural system, due to its high strength-to-weight ratio, ease of assembly and installation, economy in transport to the site, availability of various strength levels, and wider selection of sections. Innovative framing systems and modern design methods, improved fire protection, corrosion resistance, fabrication, and erection techniques combined with the advanced analytical techniques made possible by computers, have also permitted the use of steel in just any rational structural system for tall buildings.

Lateral load resisting systems are structural elements that provide its basic lateral strength and stiffness, and without which the structure would be laterally unstable. The LFRS is used to resist forces resulting from wind or seismic activity. Buildings are basically big cantilever beams which are supported on one end only and the loads are perpendicular to the beam. As in a beam, buildings are designed for strength (shear and flexure) and serviceability (deflection). Structural engineering of tall buildings requires the use of different systems for different building heights. Each system, therefore, has an economical height range, beyond which a different system is required. The requirements of these systems and their ranges are somewhat imprecise because the demands imposed on the structure significantly influence these systems. However, knowledge of different structural systems, their approximate ranges of application, and the premium that would result in extending their range is indispensable for a successful solution of a tall building project.

2.3.1 Moment Resisting-Rigid Frame Systems

Moment frames develop their resistance to lateral forces through the flexural strength and continuity of beam and column elements. They are utilized in both steel and reinforced concrete construction. Rigid frame systems for resisting lateral and vertical loads have long been accepted for the design of the buildings. Rigid framing, namely moment framing, is based on the fact that beam-to-column connections have enough rigidity to hold the nearly unchanged original angles between intersecting components. Owing to the natural monolithical behavior, hence the inherent stiffness of the joist, rigid framing is ideally suitable for reinforced concrete buildings. On the other hand, for steel buildings, rigid framing is done by modifying the joints by increasing the stiffness in order to maintain enough rigidity in the joints.

The fundamental requirements for all ductile moment frames are that:

i. They have sufficient strength to resist seismic demands,
ii. They have sufficient stiffness to limit inter-story drift,
iii. Beam-column joints have the ductility to sustain the rotations they are subjected to,
iv. Elements can form plastic hinges, and
v. Beams will develop hinges before the columns at locations distributed throughout the structure as shown in figure 2.1 (the strong column/weak beam concept).

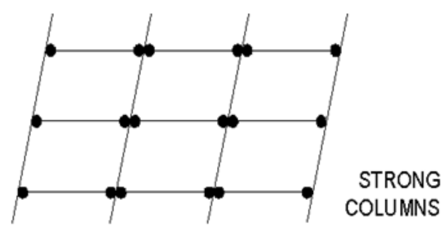


Figure 2.1: Plastic hinge formations

For a rigid frame, the strength and stiffness are proportional to the dimension of the beam and the column dimension, and inversely proportional to the column spacing. Columns are placed where they are least disturbing to the architecture, but at spacing close enough to allow a minimum depth of floor. Thus, in order to obtain an efficient frame action, closely spaced columns and deep beams at the building exterior must be used. Especially for the buildings constructed in seismic zones, special attention should be given to the design and detailing of joints, since rigid frames are more ductile and less vulnerable to severe earthquakes when compared to steel-braced[3].

2.3.2 Braced Frame Systems

Braced frame systems are mostly utilized in steel buildings since the diagonal bracing has to resist tension for one or the other directions of lateral loading. Concrete bracing of the double diagonal form is sometimes used, however, with each diagonal designed as a compression member to carry the full external shear. Contrary to rigid frame, having less elastic stiffness and low energy dissipation capacity, this system is a highly efficient and economical for resisting horizontal loading and attempts to improve the effectiveness of a rigid frame by almost eliminating the bending of columns and girders, by the help of additional bracings. It behaves structurally like a vertical truss, and comprises of the usual columns and girders, essentially carrying the gravity loads, and diagonal bracing components so that the total set of members forms a vertical cantilever truss to resist the horizontal loading.

Bracing generally takes the form of steel rolled sections, circular bar sections, or tubes. The areas around elevator, stairs, and service shafts, where frame diagonals may be enclosed within permanent walls, are the most preferable places for the braces; and the arrangement of the bracing is generally dictated by the requirements for openings.

Bracings can cover two or more than two stories in a single run which gives high strength and ductility of the structure with number of stories. This configuration is well suited for tall, slender buildings and was firstly used in a steel building, the 100-storey-high John Hancock Center (1969) shown below in Figure 2.2.

Historically, bracing has been utilized to stabilize the building laterally in many of the world's tallest structures, including 77-storey-high Chrysler Building (1930) and 102-storey-high Empire State Building (1931) in New York.

Figure 2.2: John Hancock Tower

2.4 Bracing Systems

The outcome of an earthquake manifest great devastation due to unpredicted seismic motion striking extensive damage to innumerable buildings of varying degree, i.e. either full or partial. This damage to structures in turn causes irreparable loss of life with a large number of casualties. Strengthening of structures using bracing systems proves to be a better option. A bracing system improves the seismic performance of the frame by increasing its lateral stiffness and capacity. Through the addition of the bracing system, load could be transferred out of the frame and into the braces, bypassing the weak columns while increasing strength [4].

In braced frames the lateral resistance of the structure is provided by diagonal members that, together with the girders, form the "web" of the vertical truss, with the columns acting as the "chords". Because the horizontal shear on the building is resisted by the horizontal components of the axial tensile or compressive actions in the web members. Bracing systems are highly efficient in resisting lateral loads.

As per EBCS 3 definition a frame may be classified as braced if its sway resistance is supplied by a bracing system with a response to in-plane horizontal loads which is sufficiently stiff for it to be acceptably accurate to assume that all horizontal loads are resisted by the bracing system. This may be assumed to be the case if the frame attracts not more than 10% of the horizontal loads [5].

The efficiency of bracing, is being able to produce a laterally very stiff structure for a minimum of additional material, makes it an economical structural form for any height of building, up to the very tallest. An additional advantage of full triangulated bracing is that the girders usually participate only minimally in the lateral bracing action: consequently, the floor framing design is independent of its level in the structure and, therefore, can be repetitive up the height of the building with obvious economy in design and fabrication [6]. A major disadvantage of diagonal bracing is that it obstructs the internal planning and the location of windows and doors. For this reason, braced bents are usually incorporated internally along wall and partition lines, and especially around elevator, stair, and service shafts. Another drawback is that the diagonal connections are expensive to fabricate and erect.

Steel braced frame is one of the structural systems used to resist earthquake loads in multistoried buildings. It is an economical, easy to erect, occupies less space and has flexibility to design for meeting the required strength and stiffness.

Steel bracing is a highly efficient and economical method of resisting horizontal forces in a frame structure. Bracing has been used to stabilize laterally the majority of the world’s tallest building structures. Bracing is efficient because the diagonals work in axial stress and therefore call for minimum member sizes in providing stiffness and strength against horizontal shear.

It has immense advantages not only in high rise structures but also in single story steel buildings of industrial buildings, airplane hangars or warehouse buildings [7]. For the lateral load resisting systems for such tall structures, design engineers often use vertical braced frames with two or more bracing tiers or panels stacked between the ground and roof level with this configuration, braced length is reduced, which leads to smaller brace size as shorter brace are more effective in compression.

With increasing of experiences from the events of the last earthquakes and reviewing the behavior of the structures, more and more innovative topics in new buildings has been considered. The application of seismic systems using braces as one of the most effective methods in steel structures. The most important issues in the study of this kind of systems are to determine the appropriate types of bracing.

Lateral resistance in braced frames is provided by diagonal members which forms the vertical truss structure together with the main beams. Columns in this structure are basic members. Since the shear forces are supported by horizontal components of tensile or compressive axial forces, bracing systems are very efficient. The desired behavior of bracing system in generation of lateral stiffness with minimum amount of materials, reveal it as an economic solution for a variety of buildings with arbitrary height. Another advantage of diagonal bracings is that the main beams have minimum participation in resisting of lateral loads and therefore of deck systems in different stories can be designed in a repetitive manner that is more desirable in economical point of view [8].

In braced frames, the primary source of drift capacity is through buckling and yielding of diagonal brace members. Proportioning and detailing rules for braces ensure adequate axial ductility, which translates into lateral drift capacity for the system. Special design and detailing rules for connections, beams and columns attempt to preclude less ductile modes of response that might result in reduced lateral drift capacity. [9]

2.4.1 Types of Bracings

Today braces in the constructions play a major role in supporting and integrating the whole structures of the buildings which minimizes the failure cases of structures. Furthermore various types of braces embrace different strength of force. In a multi-storey building, the beams and columns are generally arranged in an orthogonal pattern in both elevation and on plan. In a braced frame building, the resistance to horizontal forces is provided by two orthogonal bracing systems:

- Horizontal bracing: At each floor level, bracing in a horizontal plane, generally provided by floor plate action, provides a load path to transfer the horizontal forces (mainly from the perimeter columns, due to wind pressure on the cladding) to the planes of vertical bracing.

- Vertical bracing: Bracing in vertical planes (between lines of columns) provides load paths to transfer horizontal forces to ground level and provide a stiff resistance against overall sway.

Here in this thesis work emphasis is given more on vertical bracing systems.

2.4.1.1 Horizontal Bracing

A horizontal bracing system is needed at each floor level, to transfer horizontal forces (chiefly the forces transferred from the perimeter columns) to the planes of vertical bracing that provide resistance to horizontal forces.

There are two types of horizontal bracing system that are used in multi-storey braced frames[10].

- Diaphragms
- Discrete triangulated bracing.

Usually, the floor system will be sufficient to act as a diaphragm without the need for additional steel bracing. At roof level, bracing, often known as a wind girder, may be required to carry the horizontal forces at the top of the columns, if there is no diaphragm which shown in Figure 2.3.

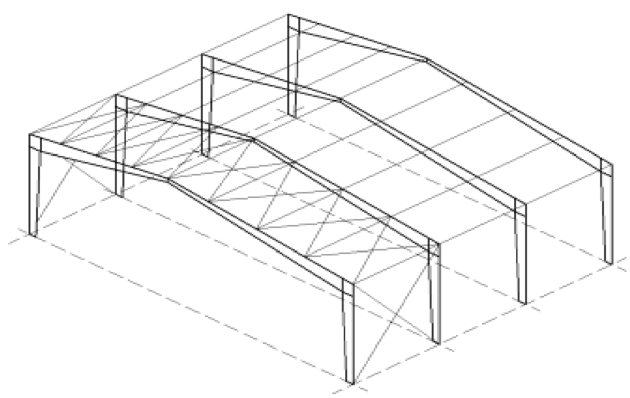


Figure 2.3: Horizontal bracing (in the roof) in a single storey building

Horizontal Diaphragms

All floor solutions involving permanent formwork such as metal decking fixed by through-deck stud welding to the beams, with in-situ concrete infill, provide an excellent rigid diaphragm to carry horizontal forces to the bracing system.

Floor systems involving precast concrete planks require proper consideration to ensure adequate transfer of forces if they are to act as a diaphragm. The coefficient of friction between planks and steelwork may be as low as 0.1, and even lower if the steel is painted. This will allow the slabs to move relative to each other, and to slide over the steelwork. Grouting between the slabs will only partially overcome this problem, and for large shears, a more positive tying system will be required between the slabs and from the slabs to the steelwork.

Connection between slabs may be achieved by reinforcement in the topping. This may be mesh, or ties may be placed along both ends of a set of planks to ensure the whole panel acts as one. Typically, a 10 mm bar at half depth of the topping will be satisfactory.

Connection to the steelwork may be achieved by one of two methods:

- Enclose the slabs by a steel frame (on shelf angles, or specially provided constraint) and fill the gap with concrete.
- Provide ties between the topping and an in-situ topping to the steelwork (known as an 'edge strip'). Provide the steel beam with some form of shear connectors to transfer forces between the in-situ edge strip and the steelwork.

If plan diaphragm forces are transferred to the steelwork via direct bearing (typically the slab may bear on the face of a column), the capacity of the connection should be checked. The capacity is generally limited by local crushing of the plank. In every case, the gap between the plank and the steel should be made good with in-situ concrete.

Timber floors and floors constructed from precast concreted inverted tee beams and infill blocks (often known as 'beam and pot' floors) are not considered to provide an adequate diaphragm without special measures.

Discrete Triangulated Bracing

Where diaphragm action from the floor cannot be relied upon, a horizontal system of triangulated steel bracing is recommended. A horizontal bracing system may need to be provided in each orthogonal direction.

Typically, horizontal bracing systems span between the 'supports', which are the locations of the vertical bracing. This arrangement often leads to a truss spanning the full width of the building, with a depth equal to the bay centers, as shown in the Figure 2.4. This floor bracing is frequently arranged as a Warren truss, or as a Pratt truss, or with crossed members.

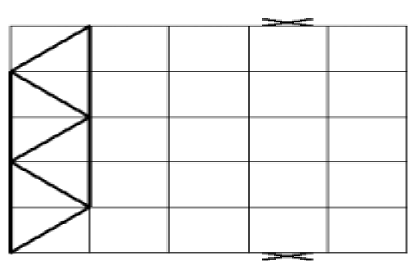


Figure 2.4: Typical floor bracing arrangement (plan view)

2.4.1.2 Vertical Bracing [8]

In a braced multi-storey building, the planes of vertical bracing are usually provided by diagonal bracing between two lines of columns. Either single diagonal is provided (in which case they must be designed for either tension or compression) or crossed diagonals are provided (in which case slender bracing members carrying only tension may be provided). This system allows obtaining a great increase of stiffness with a minimal added weight, and so it is very effective for existing structure for which the poor lateral stiffness is the main problem.

Note that when crossed diagonals are used and it is assumed that only the tensile diagonals provide resistance, the floor beams participate as part of the bracing system (in effect a vertical Pratt truss is created, with diagonals in tension and posts in compression).

The vertical bracing must be designed to resist the forces due to the following:

- Wind loads
- Equivalent horizontal forces, representing the effect of initial imperfections
- Second order effects due to sway (if the frame is flexible).

Forces in the individual members of the bracing system must be determined for the appropriate combinations of actions. For bracing members, design forces at ULS due to the combination where wind load is the leading action are likely to be the most difficult ones.

2.4.2 Classification of Vertical Bracings

Even though the shape and arrangements of bracings are various, based on its geometrical arrangements of the member, it can be classified as in to two types called concentrically bracing and eccentrically bracing as shown below in Figures 2.5 and 2.6.

Both types of bracings run diagonally from vertical member to the horizontal members (i.e. columns to beams) or from beam-column joint to other joint diagonally. This system allows obtaining a great increase of stiffness with a minimal added weight, and so it is very effective for existing structure for which the poor lateral stiffness is the main problem.

Figure 2.5: Concentric Bracings

Figure 2.6: Eccentric Bracings

These bracings may run in a single story and repeat itself on the succeeding story until it reaches to the final story or it can cover more than two stories at a single run as in the case of braced systems for mega structures shown previously in Figure 2 of John Hancock tower.

2.4.2.1 Eccentrically Braced Frames

Eccentrically braced frames (EBFs) are a lateral load resisting systems for steel building that can be considered a hybrid between conventional moment –resisting frames (MRFs) and concentrically braced frames (CBFs). EBFs are in effect an attempt to combine the individual advantages of MRFs and CBFs, while minimizing their respective disadvantages. Figure 2.7, illustrates several common EBF arrangements. Many other satisfactory arrangements can be devised. [11]

The distinguishing characteristics of an EBF is that at least one end of every brace is connected so that the brace force is transmitted either to another brace or to a column through shear and bending in a beam segment called a link. The link length in figure 2.7 is identified by the letter e.

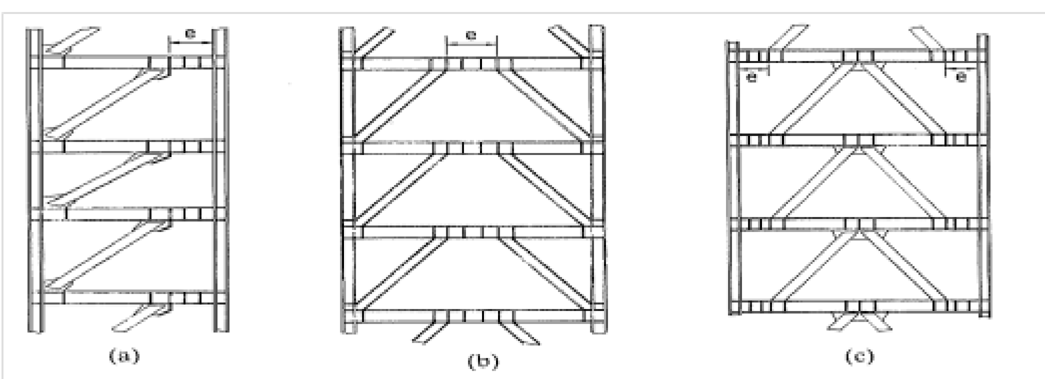


Figure 2.7: Typical arrangements for EBFs

Although eccentric bracing has been long known for wind bracing, its application to seismic resistant construction is only very resent. The excellent performance of EBFs under severe earthquake loading was demonstrated on one- third –scale model frames at the University of California in 1977. Soon after this study, several major buildings were constructed incorporating EBFs as part of their lateral seismic resisting systems, including the nineteen story bank of America building in San Diego [12] and the forty-seven story embarcadero four building in San Francisco[13], which is constructed in 1981. Since that time, numerous applications of these systems have been adopted in practice.

Eccentric Bracings reduce the lateral stiffness of the system and improve the energy dissipation capacity. Due to eccentric connection of the braces to beams, the lateral stiffness of the system depends upon the flexural stiffness of the beams and columns, thus reducing the lateral stiffness of the frame. The vertical component of the bracing forces due to earthquake causes lateral concentrated load on the beams at the point of connection of the eccentric bracings.

The most attractive features of EBFs for seismic-resistance design is their high stiffness combined with excellent ductility and energy-dissipation capacity. The bracing members in EBFs provide the high elastic stiffness characteristic of CBFs, permitting code drift requirements to be met economically. Yet, under very severe earthquake loading, properly designed and detailed EBFs provide the ductility and energy dissipation capacity characteristics of moment resisting frames (MRFs).

The excellent ductility of EBFs can be attributed to two factors,

- First, inelastic activity under severe cyclic loading is restricted primarily to the links, which are designed and detailed to sustain large inelastic deformations without loss of strength.
- Secondly, braces are designed not to buckle, regardless of the severity of lateral loading on the frame.

The yielding of the links in EBFs serves to limit the maximum force transferred to the brace, acting, in effect, as a fuse for bracing member loads. The ultimate strength of the link can be accurately estimated. Thus, by designing the brace to be stronger than the link the designer can be assured with a high degree of confidence that the brace will not buckle, regardless of the severity of the earthquake load. The rapid deterioration of buckled brace under cyclic loading is well documented. Thus the avoidance of brace buckling in EBFs permits stable hysteretic behavior under the most severe cyclic loading conditions. Note that the link not only limit brace forces, but also the load transmitted to the columns, permitting reliable design for column stability, and offering some possible advantages for difficult foundation design problems[11].

The ductility and energy dissipation capacity of EBFs is proved experimentally under cyclic lateral loads applied on the structures. This is observed, EBFs ability to sustain large deformations without strength loss which is an indicative of excellent energy dissipation capacity of EBFs shown below in hysteretic loop of Figure 2.8, [11]. This is due to buckling of brace is prevented and the link can sustain large deformations without strength loss.

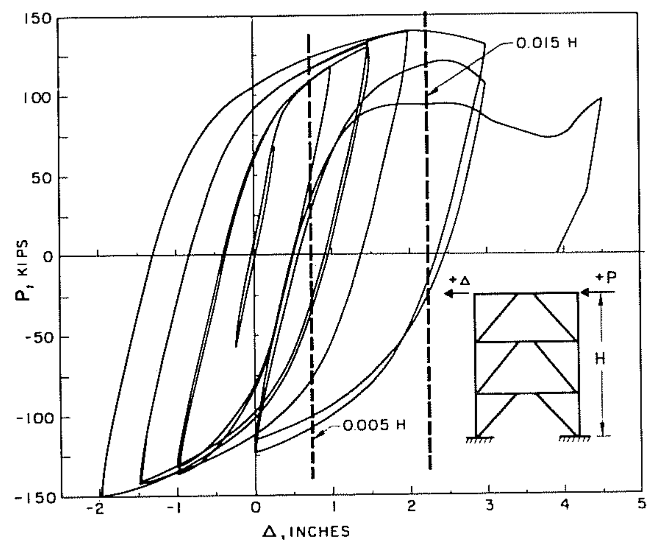


Figure 2.8: Typical experimental frame under cyclic lateral loads for EBFs.

The elastic lateral stiffness of an EBF will vary as a function of the link length e. This variation is illustrated for two simple ecentrical brace arrangement shown in Figure 2.9. When e=L, the frame has a moment resisting one and its elastic stiffness becomes minimal as shown in Figure 2.9. For e/L> 0.5 little stiffness is gained from the bracing. However as the length of the link decrease, a rapid increase in stiffness occurs. Maximum stiffness develops when e=0, corresponding to a concentrically braced frame. When e=0, there is no link present to act as a fuse for brace member forces. In order to gain maximum possible frame stiffness, the links must be kept short but too short link has excessive inelastic deformations.

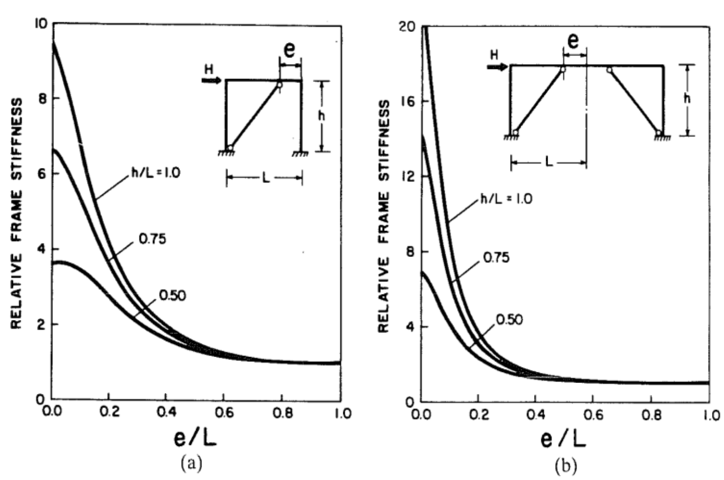


Figure 2.9: Variations of elastic lateral stiffness with e/L for two simple EBFs. [14]

Energy dissipation mechanism

In the design of seismic-resistant EBF, it is necessary to estimate the plastic rotation demand on the links. This is most accomplished through the use of energy dissipation mechanisms (collapse mechanisms), constructed by assuming rigid plastic behavior of the members. Mechanisms for a MRF and two types of EBF are illustrated in Figure 2.10. In each case, θ represents the overall frame drift. For the MRF, the rotation demand at the plastic hinges of the beam is also θ. However, for the EBFs the rotation demand on the link is much larger than θ, and from the geometry of the mechanisms can be determined to be as follows:

$$\gamma = \frac{L}{e}\theta$$

Link rotation, particularly for short links, is typically denoted by the symbol γ as a reminder of the importance of shear yielding supplying the link rotation. In Figure 2.10, the links are cross hatched to indicate it yields in shear and has formed shear hinge. The relationship between frame drift θ and the link rotation γ depends on the configuration of the EBF and must be determined from the appropriate mechanisms.

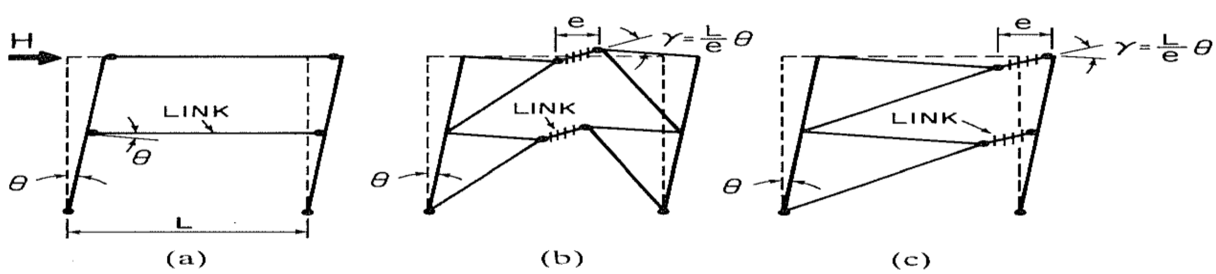


Figure 2.10: Energy dissipation mechanisms

2.4.2.2 Concentrically Braced Frames

Concentrically braced frames have suitable lateral stiffness to prevent relative drift due to lateral load impacts resulting from earthquake. Such braces are part of relatively stiff systems and compatible with common needs of architecture with varied forms as shown (Figure 2.11). Concentrically braced frames are used in different forms such as cross, diametric v-shape, Chevron (inverted-v), K shape, etc. [15]. Those types of braces have not any link length between the connection points of bracing and beams that differs from eccentrically bracing type.

The concentric bracings increase the lateral stiffness of the frame, thus increasing the natural frequency and also usually decreasing the lateral drift. However, increase in the stiffness may attract a larger inertia force due to earthquake. Further, while the bracings decrease the bending moments and shear forces in columns, they increase the axial compression in the columns to which they are connected. Since reinforced concrete columns are strong in compression, it may not pose a problem to retrofit in RC frame using concentric steel bracings.

It is a common phenomenon to use either steel or concrete materials for structural bracings as a lateral load resisting mechanisms in areas of high seismic zonal regions. Or it can be also used shear walls either at the periphery of the buildings or at the locations of lift as a core structure.

Generally, the use of steel concentrical bracing systems instead of Shear walls provides lower stiffness and resistance for a structure but it should not be forgotten that such a system has lower weight and more useful for architectural purposes.

For this research paper emphasis is given for concentrical type of steel bracing having five different types of geometrical arrangements with similar cross section for comparison purpose.

Different researchers' workout comparisons of the efficiencies of different forms of bracings, but their criteria of making assumptions for the selected group are not similar. Most Comparison of bracing was made by taking shape as the only criteria [8]. In any engineering problems formulation, criteria must be set for equal treatments of the phenomenon. Otherwise evaluations led to biased solutions and it creates also fallacy to have an optimum design type. If the designers/Engineers taking shape as the only criteria, the structure may be safe and stable but it may give unfair cost distribution to each type of bracings due to different weight of bracing members.

In this study, different forms of concentrically braced frames are taken to evaluate the performance of bracings by setting equal weight /volume as the criteria which has to be applied for each forms of bracings.

A sample of five type of steel bracing with different geometry is considered. Each bracing is tested under three different steel building having four, six and ten story. Three model types are taken to investigate the behavior of each bracing as the number of story changed. The sample building is symmetrical both in plan as well as elevation having a regular square shape of 12mX12m plan dimensions. Columns are spaced each other at 4m interval in both directions and its corresponding story height is a dimension of 4m. This implies the width and height is taken to be equal. It is assumed that a live load (LL) of 5KN/m^2 and imposed load from external walls as 12 KN/m acts at each story of the external frames of all building type due to similar purpose of the building is assumed. It is also assumed that to support the deck slab secondary steel beam is provided with a spacing of one meter at each floor level. The bracing are assumed to be a wide flange (I-section steel).

The modeling process is taken using the help of finite element structural Analysis and design software ETABS version 9.

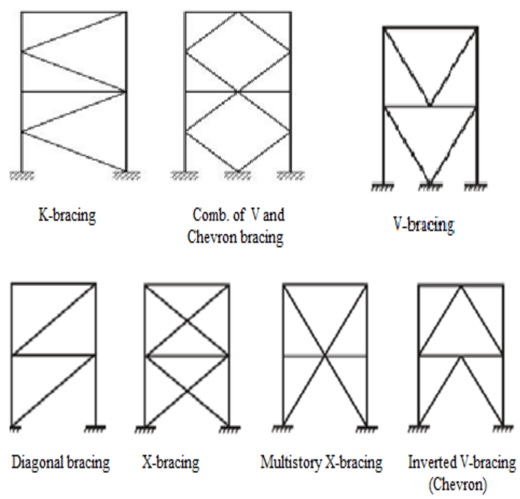


Figure 2.11: Different types of concentrically braced frames

Braced frames and moment frames are the most widely used framing systems for steel construction in seismic regions. Compared to a moment frame, a braced frame offers high-lateral stiffness for drift control. In a CBF, the members (beams, columns and braces) with the centerlines meeting at a joint form a vertical truss system. Members in a CBF are subjected primarily to axial loads in the elastic range. The diagonal bracing members are designed to deform inelastically during a moderate or severe earthquake.

Braces in a conventional CBF are expected to buckle and yield during a significant seismic event. On the basis of a significant amount of research in the past few decades, seismic design provisions have been developed. In the AISC Seismic Provisions (2002), a conventional CBF can be designed as a Special CBF (SCBF) or as an Ordinary CBF (OCBF), depending on the ductility detailing requirements that are implemented into the system.

V or inverted-V bracing is a popular configuration in the United States [16]. Because one brace in a story is expected to buckle and lose a significant amount of compressive strength while the other brace is expected to yield during tension, the AISC Seismic Provisions require that for SCBFs the beam be designed for an unbalanced vertical load at mid span. It has been suggested that the adverse effect of this unbalanced load be mitigated by using bracing configurations such as V and inverted-V braces in alternate stories to create an X-configuration over two-story modules.

The global design objective for energy dissipation in the case of Concentrically Braced Frames is to form dissipative zones in the diagonals under tension, and to avoid yielding or buckling of the beams or columns. Diagonals in compression are designed to buckle. The expected behavior for global mechanism in the case of a frame with chevron bracing (inverted V bracing) is shown in Figure 2.12.

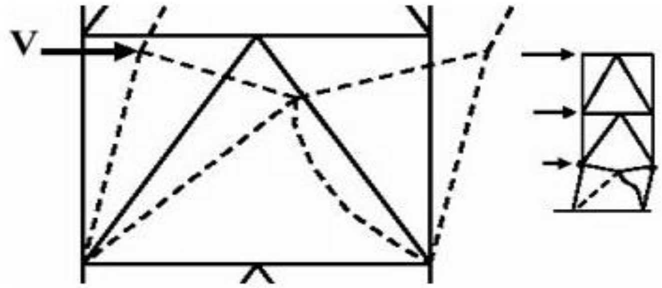


Figure 2.12: Chevron Brace Buckling

In this case, when the compression brace buckles, tension brace force doubles (before buckling has 50% of V in the tension brace and 50% of V in the compression brace). The vertical component of the tension brace axial force becomes a point load on the beam, pulling the beam down and possibly leading to hinging and buckling of the brace frame column.

When chevron bracing is used, the beam must be designed for an unbalanced load when the compression brace buckles. Often the resulting brace frame beam design weighing more. By comparison, when a two story X brace is used, when the compression brace buckles at the first floor, the braces at the second floor prevents the brace frame beam from buckling and designing the beam for an unbalanced loading is not necessary[16].

The standard analysis of bracing frame is made assuming that: under gravity loading, only the beams and columns are present in the model and under seismic loading, only the diagonals in tension are present in the model as shown in Figure 2.13 below. [17]

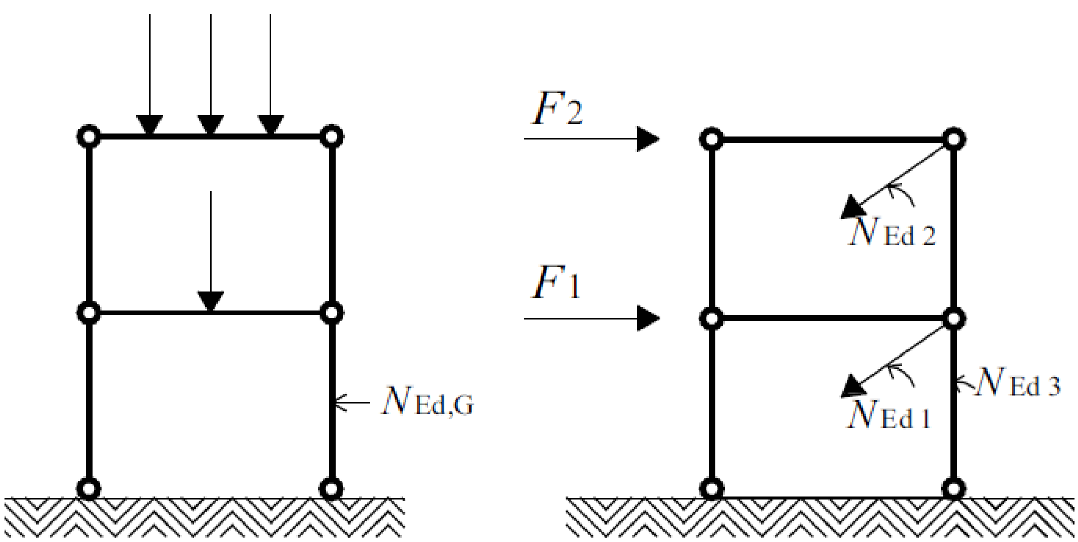


Figure 2.13: models used for analysis: - A, under gravity load   B, under lateral load

2.4.3 Performance of Concentrically Braced Frames

The design of a multi-story steel building under lateral loads is usually governed by system performance criteria (overall stiffness) rather than by component performance criteria (strength). An important task in the design of a tall steel building for structural designers is to select cost efficient lateral load resisting systems. Pure rigid frame systems alone are not efficient in resisting lateral loads for tall steel buildings due to associated high costs. Truss members such as diagonals are often used to brace steel frameworks to maintain lateral drifts within acceptable limits. In the absence of an efficient optimization technique, the selection of lateral bracing systems for multi-story steel frameworks is usually undertaken by the designer based on a trial and error process and previous experience. The optimal layout design of bracing systems is a challenging task for structural designers because it involves a large number of possibilities for the arrangement of bracing systems.

System performance is strongly influenced by aspects of brace behavior (Lehman et al. 2008). As Lehman proved in his experiment, Brace buckling places large inelastic demands on the brace at the middle of the brace, typically resulting in a plastic hinge at midspan as shown in Figure 2.14- (a). Brace buckling also places significant demands on gusset plate connections (Figure 2.14-(b) and adjacent framing members (Figure 2.14-(c). Limited cracking of the welds joining the gusset plate to the beams and columns generally is expected because of gusset plate deformation. These cracks normally initiate at story drifts in the range of 1.5 % to 2.0 %, but the cracks remain stable if the welds meet size and demand-critical weld requirements. [18].

(a) Brace buckling deformation (b) Deformation of gusset plate (c) Local yielding in beam and column

Figure 2.14: Various aspects of braced frame behavior

CBFs are strong, stiff and ductile, making them ideal for seismic framing systems. The inelastic behavior of the brace provides most of the ductility, but in order to fully utilize the frame, the connections and framing members must also be taken into account. Therefore, it is important to consider not only the performance of the brace when designing, but also the ability for the connections and the framing members to withstand the strength and deformation demands transferred from the brace during cyclic loading. Through these considerations, a maximum amount of energy can be dispersed before the system fails. [19]

Cyclic testing of conventional braced frames shows that these braces buckle in compression and yield in tension. Plastic hinges occur after the brace has buckled and the stiffness and resistance of the frame decreases, illustrated in Figure 2.15. In Zone 0-A, the frame retains its elasticity, but the brace buckles at A, causing a plastic hinge to form in Zone A-B. Load reversal in Zones B-C, C-D and D-E cause the brace to become unstable, decreasing the effectiveness of the frame. This unstable behavior is evident in the unsymmetrical response seen in Figure 2.15a. For this reason, Special Concentrically Braced Frames (SCBFs), with braces in opposing pairs, are used given the stable inelastic performance seen in Fig. 2.15c. [19]

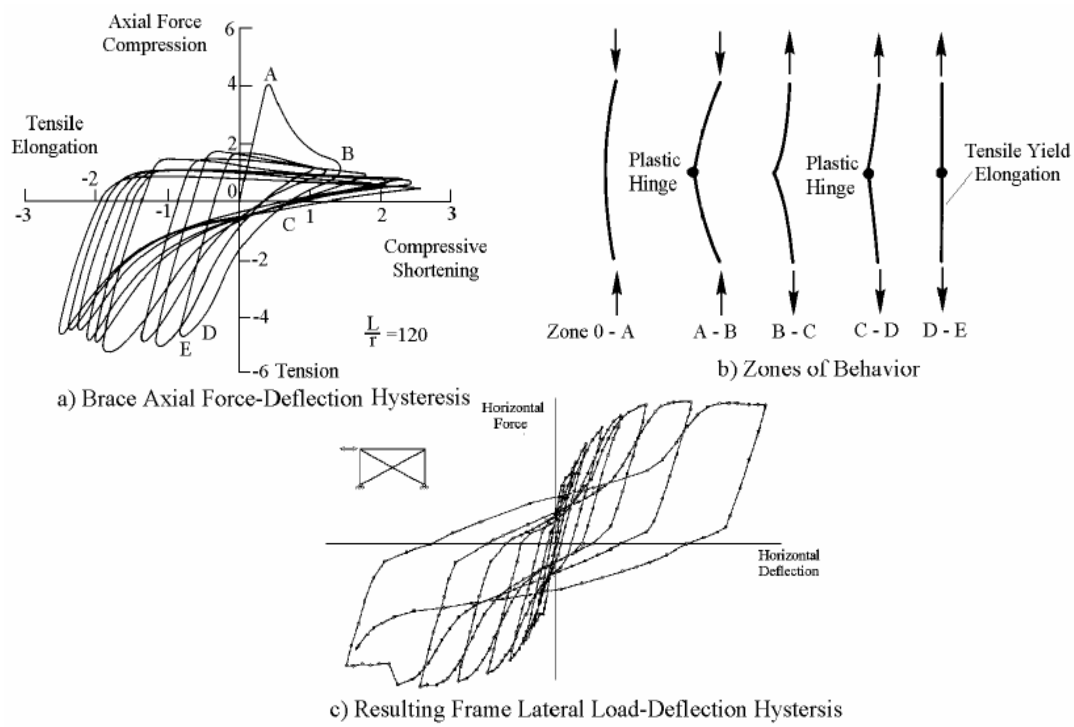


Figure 2.15: Behavior of Special Concentrically Braced Frames

As per the code provision what the engineer is expected to do is that, the brace should fail before the connection does. The goal of the Performance-Based approach is to create a more detailed hierarchy of failures. A collection of permissible yield mechanisms and failure modes for a system can be identified. The permissible yield mechanisms are brace buckling and yielding, local yielding of the gusset plate, bolt-hole elongation, and the permissible failure modes include fracture or tearing of the brace. Unacceptable failure modes are buckling of the gusset plate or fracture of connection components such as bold or weld which is shown in the Figure below [20].

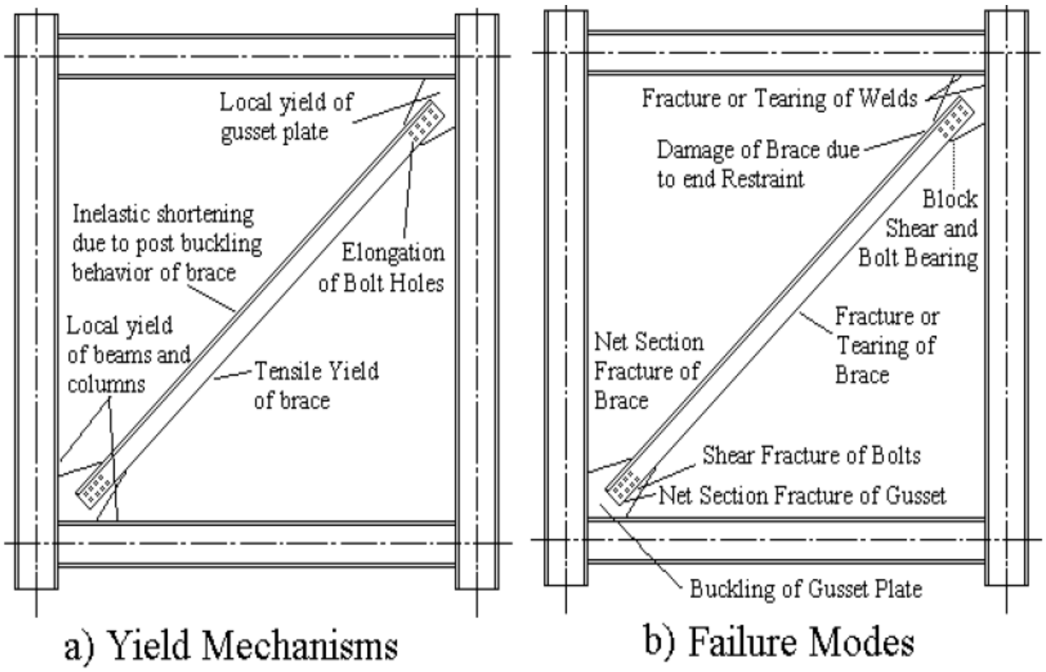


Figure 2.16: Yield Mechanisms and Failure Modes for SCBF Components

Performance-Based Methods match the performance of a structure and the damage that is expected with varying levels of seismic activity. Figure 2.17 shows these possible relationships. The three performance levels are Immediate Occupancy (IO), Life Safety (LS) and Collapse Prevention (CP). As is expected structural damage increases with seismic levels and the permissible damage is more restricted with CP than IO.

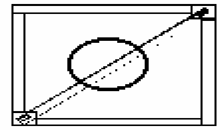
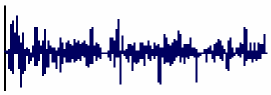
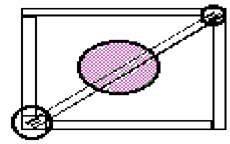
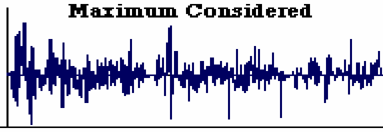
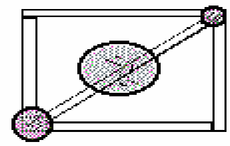
Seismic Hazard Level	Performance Level	Structural Damage
Seldom 	**Immediate Occupancy** ▪ Brace Buckling* ▪ Incipient Brace Yielding * SCBF only	
Rare 	**Life Safety** ▪ Brace Yielding ▪ Incipient Yielding of Gusset Plate ▪ Incipient Elongation of Bolt Holes	
Maximum Considered 	**Collapse Prevention** ▪ Full of Brace ▪ Yielding of Gusset Plate ▪ Elongation of Bolt Holes ▪ Incipient Brace Fracture	

Figure 2.17: Possible Performance Objectives for SCBFs

2.5 Principles for Design of Steel Special Concentrically Braced Frames

The Special concentrically braced frame (SCBF) system is generally an economical system to use for low and medium rise buildings in areas of high seismicity. It is preferred over Special Moment Frames because of the material efficiency of CBFs and the smaller required beam and column depths. SCBFs are only possible for buildings that can accommodate the braces in their architecture.

SCBFs economically develop the lateral strength and stiffness needed to assure serviceable structural performance during smaller, frequent earthquakes, but the inelastic deformation needed to ensure life safety through collapse prevention during extreme earthquakes is dominated by tensile yielding of the brace, brace buckling, and post buckling deformation of the brace. The ductility and inelastic deformations required by this second design goal vary in magnitude depending upon the seismic hazard level and the seismic design procedure. For areas of low seismicity, ASCE 7 allows steel framing systems to be designed with a Response Modification Factor, *R*, of 3.0 with no special detailing requirements to improve ductility. ASCE 7 also allows the use of Ordinary Concentrically Braced Frames (OCBFs). However, SCBFs are designed with relatively large *R* factors, and as a consequence are expected to experience relatively large inelastic deformation demands during extreme ground shaking. A story drift of approximately 2.5 % is commonly assumed as a target inelastic deformation to be achieved by SCBFs prior to brace fracture. As a result, ductile detailing and proportioning requirements are needed to ensure that SCBFs can achieve the required inelastic deformations. Corresponding inelastic flexural deformation in beams, columns, and connections will occur during these large inelastic excursions. The inelastic deformations in the beams and columns are not primary effects because they are not specific goals of the design process. Nevertheless, they influence the seismic performance of SCBFs and contribute to the cost of repair. Local slenderness limits for beams and columns are required by AISC 341 in recognition of these local inelastic deformations. [9]

The configuration of braces affects system performance. Multiple configurations of bracing can be used, and these configurations are identified in Figure 2.11. Braces buckle in compression and yield in tension. The initial compressive buckling capacity is smaller than the tensile yield force, and for subsequent buckling cycles, the buckling capacity is further reduced by the prior inelastic excursion. Therefore, bracing systems must be balanced so that the lateral resistance in tension and compression is similar in both directions. This means that diagonal bracing or chevron bracing must be used in matched tensile and compressive pairs. As a result, these bracing must be used in opposing pairs to achieve this required balance. Other bracing configurations, such as the X-brace, multistory X-brace and chevron brace directly achieve this balance. X-bracing is most commonly used with light bracing on shorter structures. Research shows that the buckling capacity of X-bracing is best estimated by using one half the brace length when the braces intersect and connect at mid section [23]. However, the inelastic deformation capacity of the X-braced system is somewhat reduced from that achievable with many other braced frame systems because the inelastic deformation is concentrated in one-half the brace length because the other half of the brace cannot fully develop its capacity as the more damaged half deteriorates. The compressive buckling resistance of most other brace configurations is best estimated by considering true end-to-end length of the brace with an effective length factor, K, of 1.0 (i.e., neglecting rotation stiffness of the brace-to-gusset connection.)

Inelastic deformation of the brace dominates the inelastic performance of SCBFs during moderate and large earthquakes, and fracture of the brace at mid-length is clearly the anticipated initial failure mode of the braced frame system. A number of brace design issues affect the inelastic deformation and ultimate fracture of the brace.

2.6 Design Approach for Bracing Systems

Braced-frame members are designed to resist the forces specified by the building code based on the type of structural system selected and the location of the building site relative to various faults and seismic source zones, as determined from seismic risk or zonation maps. Under the requirements of the AISC Seismic Provisions the brace members of an ordinary braced frame, except chevron configurations, are designed for the force corresponding to the application of the specified base shear force per the applicable building code. In the case of chevron or V braces, the design force is increased by 150% [17]. This is due to when compression brace buckles only tension brace will carry full loads. However, this requirement of 150% increase in the design force is not applicable if the chevron is designed as a special concentric brace frame (SCBF).

All bracing connections are required to be capable of resisting the maximum expected force that could be delivered to them by the bracing configuration. The design intent is that the strength of all of the brace frame components (beams, columns, connections) be larger than the expected maximum capacity of the brace member. By ensuring this, the failure of a braced-frame system is intended to be controlled by yielding and buckling of the braces only, not the other elements of the frame. As soon as braces yield in tension or buckle in compression, they start to plastify under increasing lateral loads. As full plastification occurs, the stiffness and load-carrying capacity of the brace are limited and, therefore, the load that may be attracted to the brace frame as a whole is limited. As a result only the brace member will be damaged and will require repairs after an earthquake, whereas all other components of the braced frame will be undamaged and require no repair. Note that as previously discussed; the design requirements for ordinary V- and chevron-braced frames are not adequate to accomplish this objective, as the beams at the apex of the V or chevron are vulnerable to damage.

The AISC Seismic Provisions require that brace connections be designed for the lesser of the following forces:

- The strength of the brace in axial tension.
- An over strength factor (Ω_o) times the design force in the brace including gravity loads.
- The maximum force that can be transferred to the brace by the system, considering other limiting factors, such as the capacity of diaphragms to transfer shear forces to the braced frames.

For the section to be safe, the yield resistance $N_{pl,Rd}$ of the bracing diagonals should be greater than the axial tension force N_{Ed} computed under the seismic action effect: $N_{pl,Rd} \geq N_{Ed}$

For each bracing diagonal, the ratio of the yield resistance provided $N_{pl,Rd}$ to the resistance required N_{Ed} is determined: $\Omega i = N_{pl,Rd,i} / N_{Ed},i$,[17].

These ratios Ωi represent the excess capacity of the sections with respect to the minimum requirement and are therefore called 'section over-strength'. In order to achieve a global plastic mechanism the values of Ωi should not vary too much over the full height of the structure, and a homogenization criterion is defined; the maximum Ωi should not differ from the minimum by more than 25%.

As the diagonals are effectively ductile 'fuses', the beam and column design forces are a combination of:

- The axial force $N_{Ed,G}$ due to gravity loading in the seismic design situation.
- The axial force $N_{Ed,E}$ due to seismic action amplified by the 'over-strength' of the diagonal, which is found by multiplying the section 'overstrength' factor Ω by the material 'overstrength' γ_{ov} (when applying so called capacity design).

The axial load design resistance $N_{pl,Rd}$ of the beam or the column, which takes into account interaction with the design bending moment M_{ED} in the seismic design situation, should satisfy:

$$N_{pl,Rd}(\mathrm{M_{ED}}) \geq N_{ED,G} + 1.1\gamma_{ov}\,\Omega.\,N_{ED,E}$$

2.7 Preference of Bracing Location

Bracing can be located at different location of the structure. It can be located at the center or any sides of the building but its resisting capacity and efficiencies for the stability and torsional capacity of the structure is completely different. As the free encyclopedia for UK steel construction information said it is preferable to locate bracing at or near the extremities of the structure [8], in order to resist any torsional effects.

Braced frames are most effective at the building perimeter, where they can control the building's torsional response. ASCE 7 allows buildings to be considered sufficiently redundant (and thus avoid a penalty factor) with two braced bays on each of the presumed four outer lines (assuming a rectangular layout). Such a layout is good for torsion control as well. In the same way; in mid-rise or high-rise buildings, braced frames are often used in the core of the structure, with a perimeter moment frame used to provide additional torsional resistance. Where possible, bracing members inclined at approximately 45° are recommended [8]. This provides an efficient system with relatively modest member forces compared to other arrangements, and means that the connection details where the bracing meets the beam/column junctions are compact. Narrow bracing systems with steeply inclined internal members will increase the sway sensitivity of the structure. Wide bracing systems will result in more stable structures. But the wider bracing affect the aesthetical values of the building and it may prevent door and widows openings. This obstruction can be minimized by providing V-bracing for windows opening and chevron bracing (inverted V-bracing) for door opening.   The table below gives an indication of how maximum deflection varies with bracing layout, for a constant size of bracing cross section.

Bracing efficiency			
Storey height	**Bracing width**	**Angle from horizontal**	**Ratio of maximum deflection (compared to bracing at 34°)**
h	2h	26°	0.9
h	1.5h	34°	1.0
h	h	45°	1.5
h	0.75h	53°	2.2
h	0.5h	63°	4.5

Table 2.1: Comparisons of Bracing efficiencies at different angle of bracing inclination [8].

3. MODELING AND LOADING OF STRUCTURAL SYSTEMS

3.1 Modeling Software, ETABS

ETABS (Extended Three-Dimensional Analysis of Building Systems) is special purpose analysis and design program developed specially for buildings.

Original development of TABS 30 years back led to the development of the today's ETABS. Early releases of ETABS provided input, output and numerical solution that took into consideration the characteristics unique to building type structures, providing a tool that offered significant savings in time and increased accuracy over general purpose programs.

As computers and computer interfaces evolved, ETABS added computationally complex analytical options such as dynamic nonlinear behavior, and powerful CAD-like drawing tools in a graphical and object-based interface.

ETABS offers the widest assortment of analysis and design tools available for the structural engineer working on building structures. The following list represents just a portion of the types of systems and analysis that ETABS can handle easily:

- Multi-story commercial, government and health care facilities
- Parking garages with circular and linear ramps
- Staggered truss buildings
- Buildings with steel, concrete, composite or joist floor framing
- Buildings based on multiple rectangular and/or cylindrical grid systems
- Flat and waffle slab concrete buildings
- Buildings subjected to any number of vertical and lateral load cases and combinations, including automated wind and seismic loads
- Multiple spectrum load cases, with built-in input curves
- Automated transfer of vertical loads on floors to beams and walls
- P-Delta analysis with static or dynamic analysis
- Explicit panel-zone deformations
- Construction sequence loading analysis
- Multiple linear and nonlinear time history load cases in any direction Foundation/support settlement

- Large displacement analysis
- Nonlinear static pushover
- Buildings with base isolators and dampers
- Floor modeling with rigid or semi-rigid diaphragms
- Automated vertical live load reductions

3.1.1 Physical Modeling Terminologies in ETABS

In ETABS objects, members, and elements are often referred. Objects represent the physical structural members in the model. Elements, on the other hand, refer to the finite elements used internally by the program to generate the stiffness matrices [21]. In many cases objects and physical members will have a one-to-one correspondence, and it is these objects that the user draws in the ETABS interface.

In ETABS, objects or physical members drawn by users, are typically subdivided into the greater number of finite elements needed for the analysis model, without user input.

3.1.2 Structural Objects

ETABS uses objects to represent physical structural members. The following objects are available in ETABS:

- Point objects
- Line objects and
- Area objects

Area objects are used to model walls, slabs, decks, planks, and other thin-walled members. Area objects will be meshed automatically into the elements needed for analysis if horizontal objects with the membrane definition are included in the model; otherwise, the user should specify the meshing option to be used [21].

3.2 **Frame Geometry**

In order to evaluate different bracing systems, prior to going into any action for assessment, models with different bracing systems must be considered. In this regard the types of selected models, their shape and sizes are significant as they have influence on the behavior of the frame models. As a result, the frame geometry selected was identical both in x- and y- coordinate system having a plan dimension of 12m by 12m. Each bay has a 4m by 4m square shape element which consists of a secondary beam spaced in one meter is shown in Figure 3.1 below. The beams, columns and bracings are an I-section steel element with their corresponding size is tabulated as follow:

Types	**Wide flange Design section**
Beams	W12x30
columns	W12x96
Bracings	W5x16
Secondary beam	W8x10

Table 3.1: Design sections used in the model

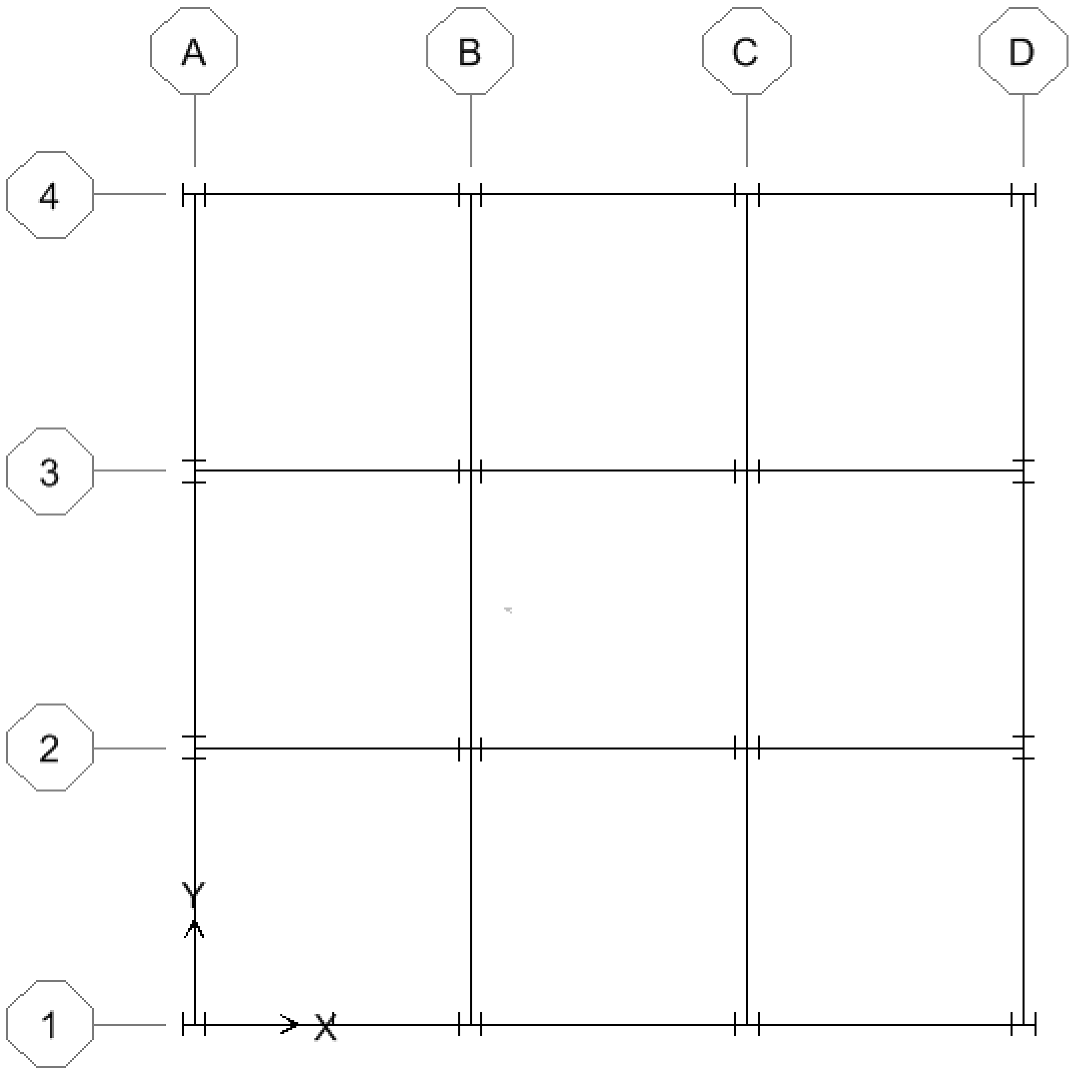
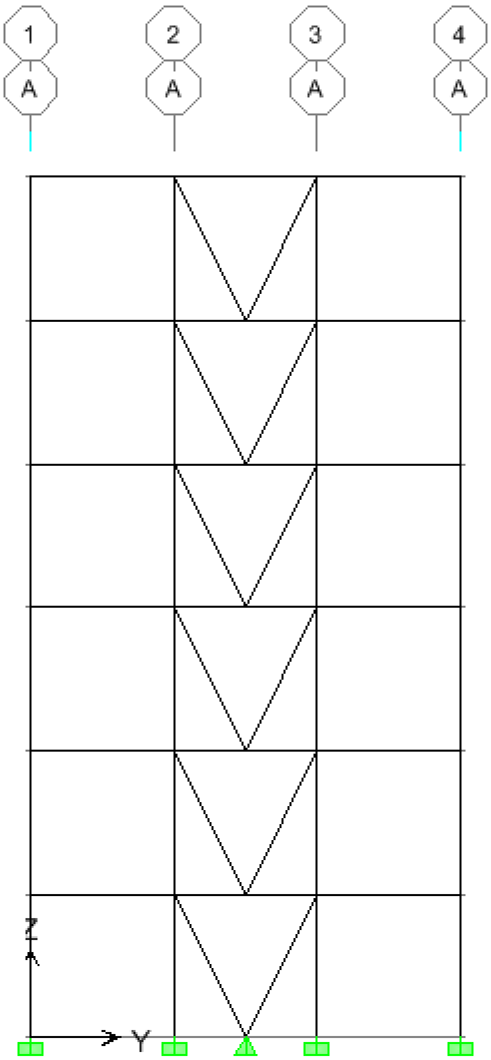


Figure 3.1: Floor Plan of the steel building

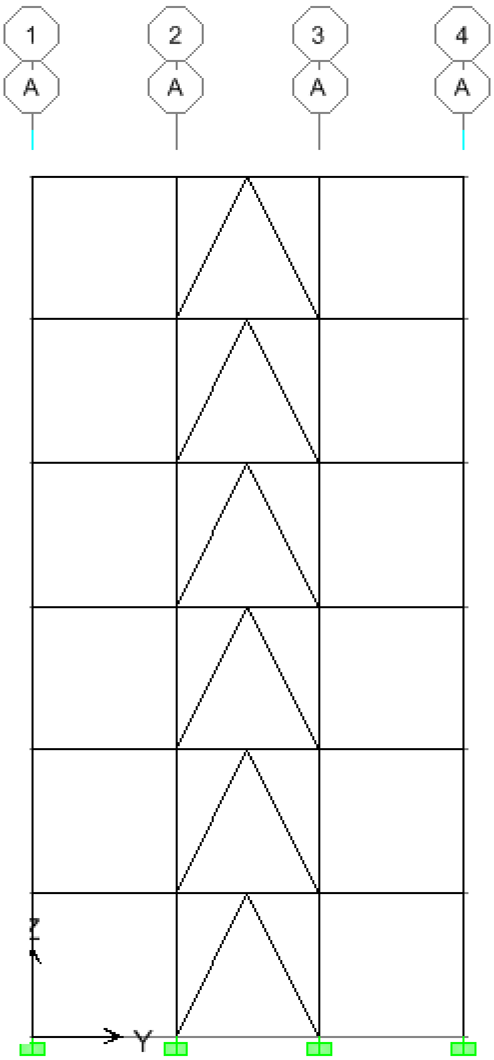
The details of frames geometry and location of bracings are as following**:**

Five different concentric types of braces have been implemented in three frames with various stories (4-, 6- and 10-storey frames) so that these frames represent low to medium rise structures. The use of three bays is good choice for the efficient placement of the selected bracing type which consists of V-bracing, inverted V- bracing, K-bracing, mirror of k-bracing and combination of V-and inverted V-bracing systems provided within the frame central bays of the exterior parts of the building as shown in Figure below. Each of these five bracing types is provided in 4- storey frames, 6- storey frames and 10-storey frames systems. This makes to differentiate the behavior of bracings in low rise and medium rise structures.

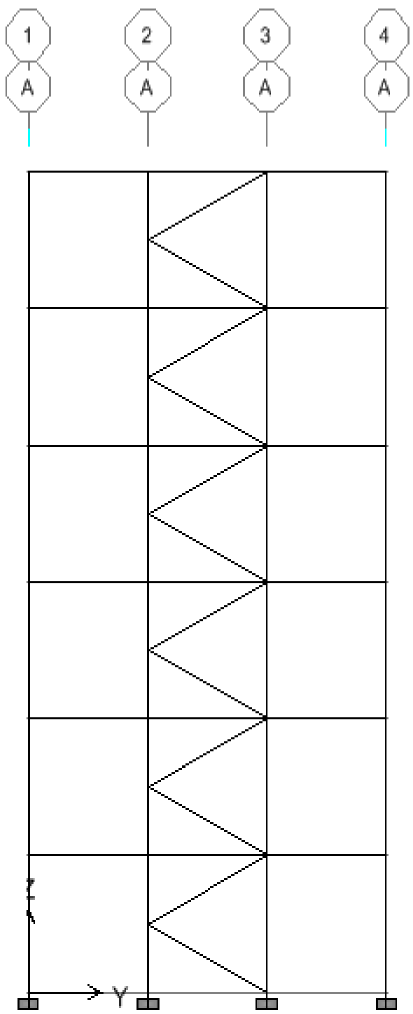
The height to width ratio of the buildings is taken to be one for equal treatments of all bracing type having equal weight for all comparison of the bracing systems.



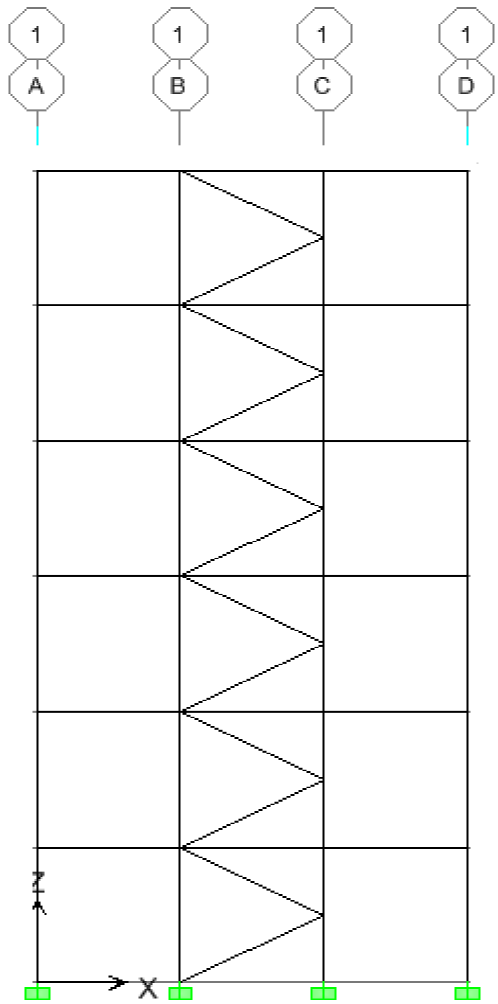
A) V-bracing,



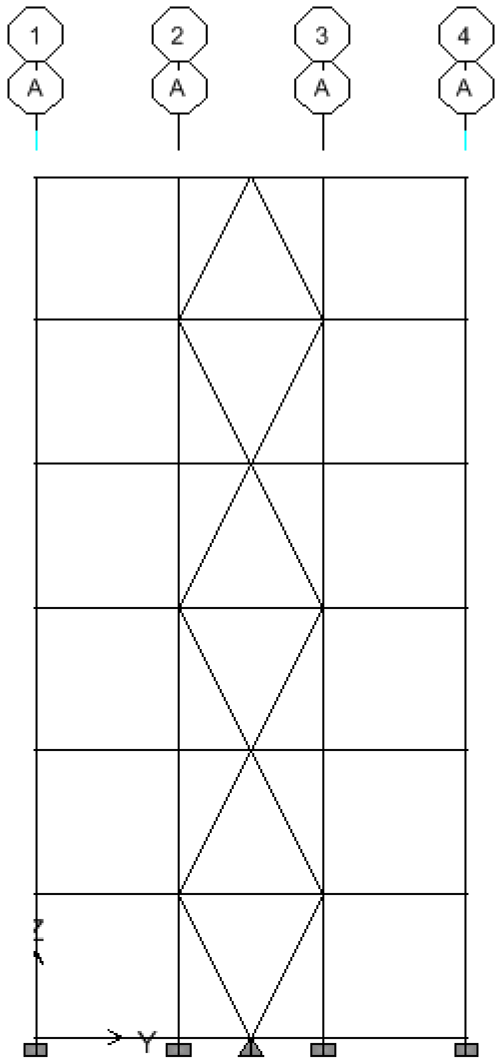
B) Inverted V-bracing,



C) K-bracing,



D) Mirror of K-bracing and



E) Combination of V and chevron bracing.

Figure 3.2: Typical geometry of bracing systems in 6 story frame

The type of steel grade with its corresponding nominal yield strength f_y and ultimate tensile strength f_u that used in the model is classified based on the number of stories as shown in Table 3.4 below.

Story	Nominal steel grade	f_y (MPa)	f_u(MPa)
4story	Fe430	275	430
6story	Fe430	275	430
10story	Fe510	355	510

Note: f_y and f_u values are for thickness of section $t \leq 40$mm.

Table 3.2: Steel Grade Considered for the given story

3.3 Loading consideration in ETABS software

For analysis of these steel building different loading is considered. These are self weight of the structure, external wall load at the periphery of the building which is called cladding load, superimposed load from fixed furniture, live load, and earthquake loads are considered. The magnitudes of these loads are shown in the Table below.

Live load	5KN/m^2
External wall load (cladding load)	15*4*0.2=12KN/m
Load from fixed furniture (super dead)	1.15KN/m^2
Earthquake load	As per the code provision

Table 3.3: Assumed Live and Dead Load Magnitudes acting on the Structure

These loads are taken by assuming the building is to produce similar service at each level of the floors systems. Because the main target of the study is to evaluate and identify the most effective types of bracing, from the given concentrical types of bracings.

4 ANALYSIS AND COMPARISON OF ANALYSIS RESULTS OF STRUCTURAL SYSTEM FOR SEISMIC LOADS

4.1 Earthquake analysis

The lateral load analysis of this study is based on Eurocode 8 design manuals. This is due to the new EBCS version of 2013 is not publically distributed. In addition lack of national annex for our country prohibits this study to work on EBCS codes.

As per Euro code 8, the horizontal design forces are defined from maximum acceleration of the structure, under the expected earthquake, that is represented with the acceleration spectrum of the structure. The starting point is an elastic response spectrum, which is reduced with factors that take into consideration the ability of structure to absorb seismic energy through rigid deformation. In the horizontal plane, the seismic action acts simultaneously and independently in two orthogonal directions that have the same response.

Euro code suggests two different design spectrums. Type 1 for the more seismically active regions (southern Europe), and type 2 for less seismically active regions (central and northern Europe).

In this study, Type1 design spectrum is selected in order to notice the effect of earthquake on each bracing systems which may give maximum lateral displacement. In addition, there are also different parameters that are considered as an input for analysis. Of which behavior factor (q) is one factor that affect the analysis result. Depending on the dissipation behavior of the structure, behavior factor values varies from bracing to bracings [22]. K-bracing types are less dissipative behavior compared to other concentrically bracing (V and Chevron types of bracings). Behavior factor values which satisfy regularity in elevation are shown in Table 4.1.

STRUCTURAL TYPE	Ductility class	
	DCM	DCH
a) Moment resisting frames	4	$5\alpha_u/\alpha_1$
b) Frames with concentric bracings Diagonals bracings	4	4
c) Frames with eccentric bracings	4	$5\alpha_u/\alpha_1$
d) Inverted pendulum	2	$2\alpha_u/\alpha_1$
e) Structures with concrete cores or concrete walls	see section 5	
f) Moment resisting frame with concentric bracing	4	$4\alpha_u/\alpha_1$
g) Moment resisting frames with infills Unconnected concrete or masonry infills, in contact with the frame	2	2
Connected reinforced concrete infills	see section 7	
Infills isolated from moment frame(see moment frames)	4	$5\alpha_u/\alpha_1$

Table 4.1: Upper limit of reference values of behavior factors for systems regular in elevation

As per Euro code 8 ground types is classified under five types by considering deep geological event and the influence of local ground conditions on the seismic action shown in Table 4.2.

Ground Type	Description of stratigraphic profile	Parameters		
		V_s,30(m/s)	N_{SPT} below 30cm	c_u (Kpa)
A	Rock or other rock-like geological formation, including at most 5m of weaker material at the surface	>800	_	_
B	Deposit of very dense sand, gravel or very stiff clay, at least several tens of meters in thickness, characterized by a gradual increase of mechanical properties with depth.	360-800	>50	>250
C	Deep deposit of dense or medium- dense sand, gravel of stiff clay with thickness from several tens to many hundreds of meters.	180-360	15-50	70-250
D	Deposit of loose-to-medium cohesionlesss soil (with or without some soft cohesive layers), or of predominantly soft-to-firm cohesive soil.	<180	<15	<70
E	A soil profile consisting of a surface alluvium layer with vs values of type C or D and thickness varying between about 5m and 20m, underlain by stiffer material with vs> 800m/s			

Table 4.2: Ground type classification as per Euro code 8.

For this study the following parameters are considered for earthquake analysis as per Euro code 8-design of structures for earthquake resistance using ETABS version 9.7.0.

S.No	Parameters	Values
1	Design Ground Acceleration, ag	0.2g
2	Design spectrum type	1
3	Ground type	B
4	Behavior factor	2 for K-bracing
		4 for other bracing
5	Accidental eccentricity	0.05

Table 4.3: General Parameters considered during Analysis

Structural Regularity

Structural simplicity, characterized by the existence of clear and direct paths for the transmission of the seismic forces, is an important objective to be followed, since the modeling, analysis, dimensioning, detailing and construction of simple structures are subject to much less uncertainty and thus the prediction of its seismic behavior is much more reliable. For the purpose of seismic design, building structures are categorized into being regular or non-regular. The distinction as regular or non-regular in plan or in elevation or in both has implications on the Structural modeling, method of analysis and values of behavior factor being as:-

- Structural model to be used - either planar or a spatial modeling
- Method of analysis to be used can be static or dynamic
- The value of behavior factor in equivalent static analysis can be basic or increased which is shown in Table 4.4.

REGULARITY		SIMPLIFICATION		BEHAVIOR FACTOR
PLAN	ELEVATION	MODAL	ANALYSIS	
Yes	Yes	Planar	Static*	Basic
Yes	No	Planar	Static*	Increased
No	Yes	Spatial	Static*	Basic
No	No	Spatial	Dynamic	Increased

*Fundamental period < 2 second

Table 4.4: Effect of Regularity on structures

It is very important to know the regularity of structure before analysis is started. In this study the building is regular in plan that satisfies the following criteria of Euro code 8 provisions:-

- Simplified and symmetrical arrangement of building plan in two orthogonal directions;
- Compact plan configuration (no H, I, X shapes) total dimensions of re-entrant corners or recess in one direction < 25 % of overall dimension in that direction.
- In plane stiffness of the floors is assumed to be greater than lateral stiffness of the vertical elements so that the deformation of floors has small effect on the distribution of the forces among the vertical elements due to rigid floor diaphragm assumption.

It is also regular in elevation which can satisfy the following criteria of Euro code 8:-

- Both lateral stiffness and mass is reduced gradually without abrupt changes from base to top;
- Storey resistance capacities shall not vary disproportionately with their adjacent story;
- The given building elevation doesn't have any set back effect.

Within the seismic effects and the effects of the other actions included in the seismic design situation may be determined on the basis of the linear-elastic behavior of the structure. Depending on the structural characteristics of the building one of the following two types of linear-elastic analysis may be used:

1. Lateral force method of analysis
2. Modal response spectrum analysis.

Lateral force method of analysis is applied to buildings whose response is not significantly affected by contributions from modes of vibration higher than the fundamental mode in each principal direction.

The above requirement is deemed to be satisfied in buildings which fulfill both of the following two conditions.

a) They have fundamental periods of vibration T1 in the two main directions which are smaller than the following values.

$$T_1 = \begin{cases} 4T_c \\ 2.0S \end{cases}$$

Where S and T_C is given in Table 4.5 for type 1 elastic response spectra

b) They meet the criteria for regularity in elevation.

For buildings with heights of up to 40 m the value of T1 (in s) may be approximated by the following expression:

$$T_1 = C_t . H^{3/4}$$

Where

Ct = 0.085 for moment resistant space steel frames,

= 0.075 for moment resistant space concrete frames and for eccentrically braced steel frames

=0.050 for all other structures;

H is the height of the building, in m from basement.

Ground type	**S**	**T_B(S)**	**T_c(S)**	**T_D(S)**
A	1	0.15	0.4	2
B	1.2	0.15	0.5	2
C	1.15	0.2	0.6	2
D	1.35	0.2	0.8	2
E	1.4	0.15	0.5	2

Table 4.5: Values of Parameters describing the recommended Type 1 elastic response spectra

The previously models of five different concentrically bracings shown in Figure 3.2 are subjected for seismic excitation in both orthogonal directions on the 4-story, 6-story and 10-story steel buildings, and analyzed using the finite element analysis software of ETABS nonlinear version 9.7.0 with an extended 3D analysis of building systems.

From the ETABS analysis result, due to lateral earthquake load effect, the frame produces different lateral displacement for each type of bracing model with bracings provided on the middle bays of the elevation view. Each of the analysis result is subsequently presented as follow for each of the bracing systems.

4.2 Analysis results

Those steel building frames subjected to the previously specified loads and the corresponding lateral displacement at each floor story level for the particular bracing type (V-bracing, chevron bracing, alternative V and inverted bracings at each story level (called combined bracing), K-bracing and mirror of K bracing) when the earthquake is acting in the y-direction is found and presented in subsequent section shown below. The lateral displacement values are taken on the axis at which bracings are located (periphery of the building).

4.2.1 Four Story Building Structure Analysis Result

Story	**Disp $\wedge$ in (mm)**
Story 1	1.858
Story 2	4.16
Story 3	6.17
Story 4	7.558

Table 4.6: Lateral displacement result for Inverted V or (chevron) bracing

Story	Disp V
Story 1	2.246
Story 2	4.98
Story 3	7.397
Story 4	9.162

Table 4.7: Lateral displacement result for V bracing structural system

Story	Disp comb
Story 1	2.234
Story 2	4.713
Story 3	6.982
Story 4	8.587

Table 4.8: Lateral displacement results for combination of V and inverted V bracing system

Story	Disp K
Story 1	4.068
Story 2	9.063
Story 3	13.484
Story 4	16.654

Table 4.9: Lateral displacement result for K bracing structural system

Story	Disp K mirror
Story 1	4.068
Story 2	9.063
Story 3	13.483
Story 4	16.653

Table 4.10: Lateral displacement results for K mirror bracing structural system

4.2.2 Six Story Building Structure Analysis Result

Story	Disp chevron
Story 1	2.408
Story 2	5.73
Story 3	9.186
Story 4	12.472
Story 5	15.309
Story 6	17.433

Table 4.11: Lateral displacement result for Inverted V or (chevron) bracing structural system

Story	Disp V
story 1	2.769
story 2	6.413
story 3	10.115
story 4	13.625
story 5	16.706
story 6	19.11

Table 4.12: Lateral displacement result for V bracing structural system

Story	**Disp comb**
Story 1	2.849
Story 2	6.195
Story 3	9.859
Story 4	13.212
Story 5	16.169
Story 6	18.444

Table 4.13: Lateral displacement result for combination of v and inverted v bracing systems

Story	**Disp K**
Story 1	5.168
Story 2	12.117
Story 3	19.288
Story 4	26.1
Story 5	32.043
Story 6	36.618

Table 4.14: Lateral displacement of K bracing structural system

Story	**Disp K mirror**
Story 1	5.168
Story 2	12.117
Story 3	19.287
Story 4	26.098
Story 5	32.041
Story 6	36.616

Table 4.15: Lateral displacement result for mirror of K bracing structural system

4.2.3 Ten Story Building Structure Analysis Result

Story	Disp Chevron
Story1	2.184
Story2	5.479
Story3	9.283
Story4	13.388
Story5	17.597
Story6	21.737
Story7	25.664
Story8	29.261
Story9	32.444
Story10	35.077

Table 4.16: Lateral displacement result for Inverted V or (chevron) bracing structural system

Story	Disp V
Story1	2.687
Story2	6.434
Story3	10.553
Story4	14.882
Story5	19.251
Story6	23.516
Story7	27.556
Story8	31.275
Story9	34.607
Story10	37.419

Table 4.17: Lateral displacement result for V bracing structural system

Story	Disp comb
Story1	2.757
Story2	6.111
Story3	10.228
Story4	14.349
Story5	18.682
Story6	22.847
Story7	26.811
Story8	30.467
Story9	33.687
Story10	36.413

Table 4.18: Lateral displacement result for combination of v and inverted v bracing system

Story	Disp K
Story1	4.883
Story2	11.928
Story3	19.873
Story4	28.324
Story5	36.915
Story6	45.332
Story7	53.313
Story8	60.644
Story9	67.179
Story10	72.663

Table 4.19: Lateral displacement result for K bracing structural system

Story	Disp K mirror
Story1	4.883
Story2	11.928
Story3	19.872
Story4	28.323
Story5	36.913
Story6	45.331
Story7	53.311
Story8	60.643
Story9	67.178
Story10	72.661

Table 4.20: Lateral displacement result for mirror of K bracing structural system

5 COMPARISONS AND DISCUSSION OF LATERAL DISPLACEMENTS

5.1 Result

The previously analyzed structures of each bracing structures of the same story is plotted using excel spreadsheet for comparison purpose and the corresponding result is tabulated as shown below for the above analysis values.

Story	Chevron	V-brace	Comb. brace	K-brace	K mirror
Story 1	1.858	2.246	2.234	4.068	4.068
Story 2	4.16	4.98	4.713	9.063	9.063
Story 3	6.17	7.397	6.982	13.484	13.483
Story 4	7.558	9.162	8.587	16.654	16.653

Table 5.1: Lateral displacement values of five bracings for four story building

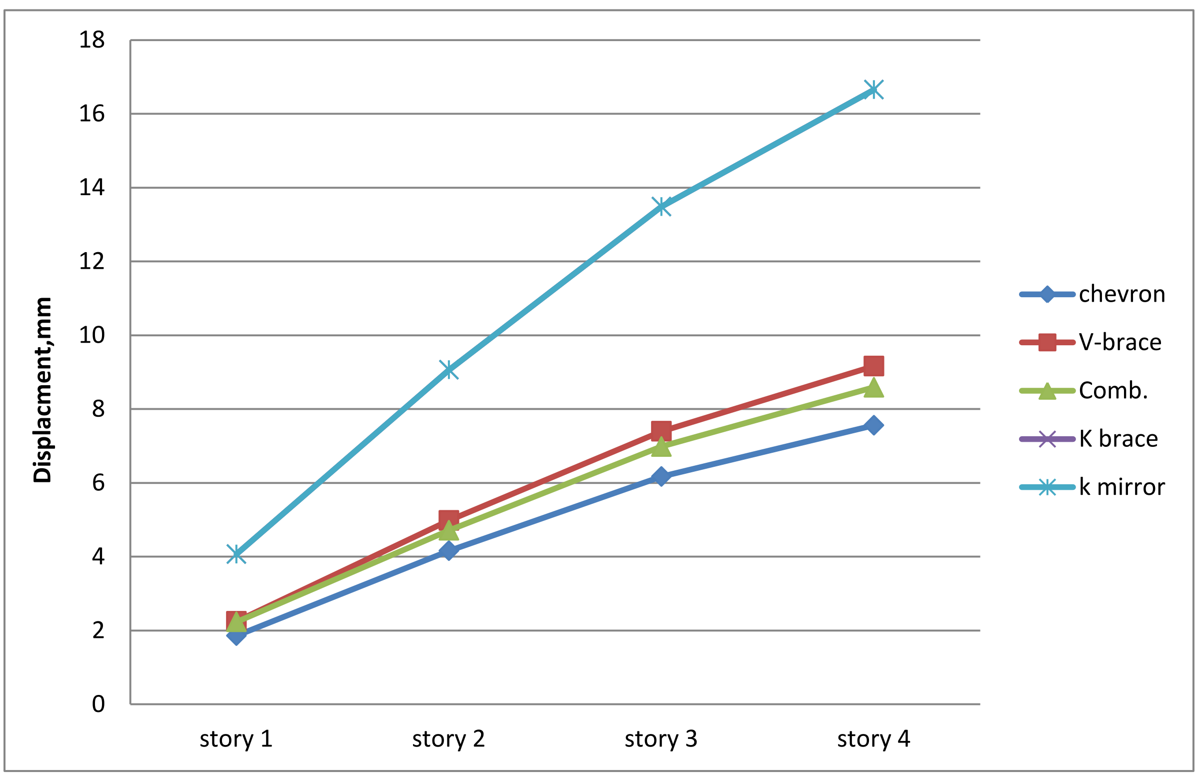


Figure 5.1: Plot of Lateral displacement values of each bracing types for four story Building

Story	Chevron	V-brace	Comb. brace	K brace	K mirror
Story 1	2.408	2.769	2.849	5.168	5.168
Story 2	5.73	6.413	6.195	12.117	12.117
Story 3	9.186	10.115	9.859	19.288	19.287
Story 4	12.472	13.625	13.212	26.1	26.098
Story 5	15.309	16.706	16.169	32.043	32.041
Story 6	17.433	19.11	18.444	36.618	36.616

Table 5.2: Lateral displacement values of five bracings for six story building

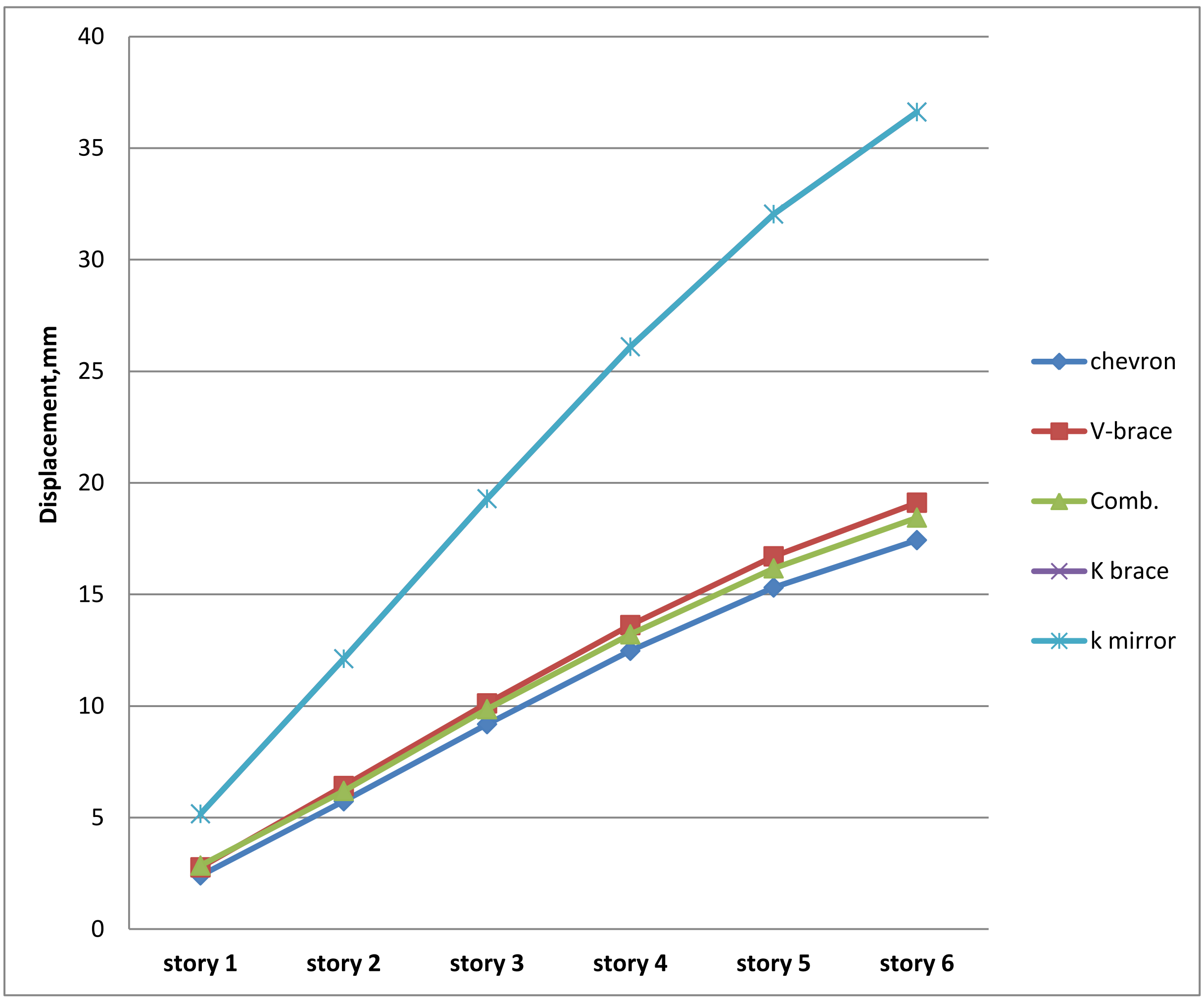


Figure 5.2: Plot of lateral displacement values of each bracing types for six story Building

Story	Chevron	V-brace	Comb. brace	K brace	K mirror
Story1	2.184	2.687	2.757	4.883	4.883
Story2	5.479	6.434	6.111	11.928	11.928
Story3	9.283	10.553	10.228	19.873	19.872
Story4	13.388	14.882	14.349	28.324	28.323
Story5	17.597	19.251	18.682	36.915	36.913
Story6	21.737	23.516	22.847	45.332	45.331
Story7	25.664	27.556	26.811	53.313	53.311
Story8	29.261	31.275	30.467	60.644	60.643
Story9	32.444	34.607	33.687	67.179	67.178
Story10	35.077	37.419	36.413	72.663	72.661

Table 5.3: Lateral displacement values of five bracings for ten story building

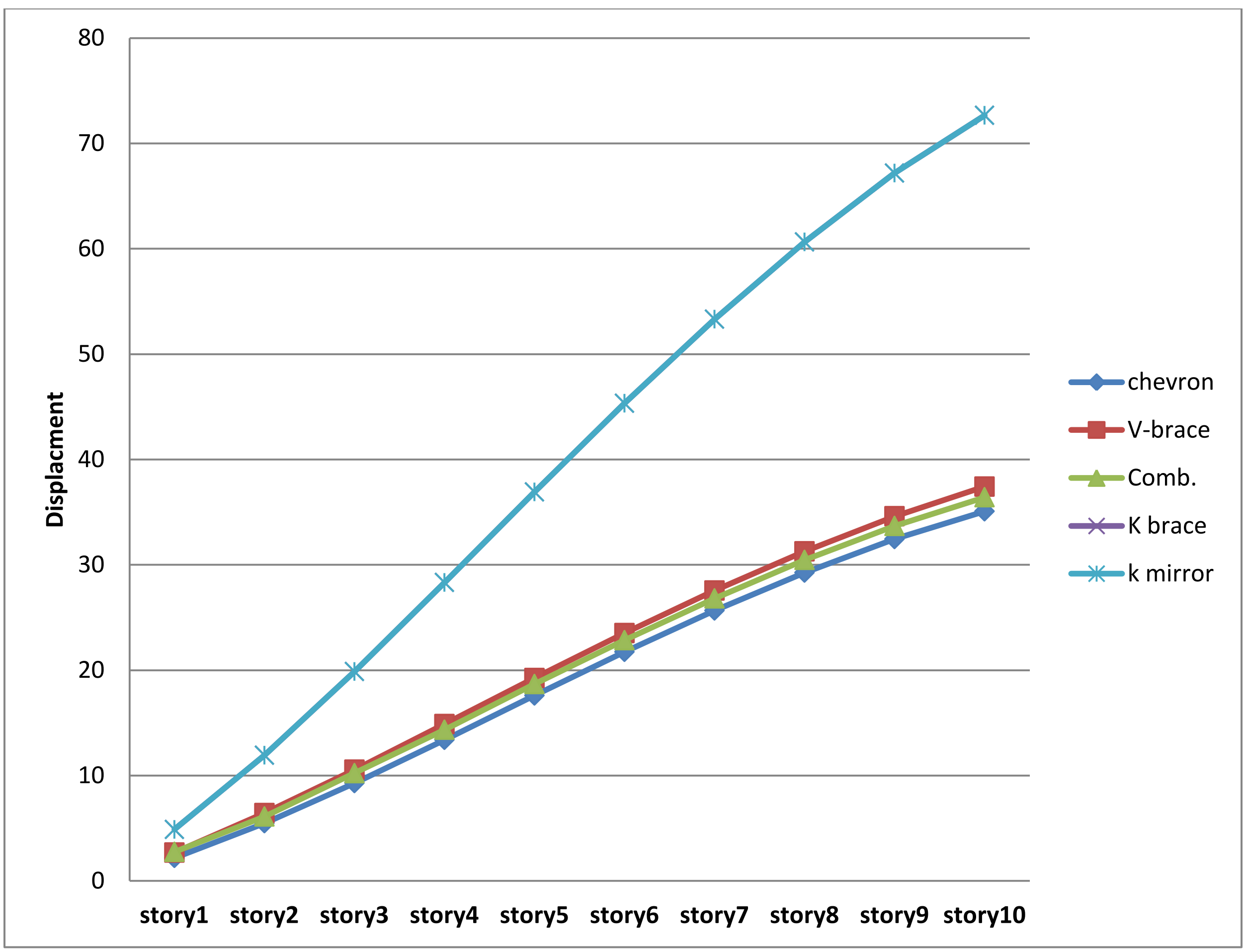


Figure 5.3: Plot of lateral displacement values of each bracing types for ten story Building

In addition to lateral displacements, it is also very important to check the story drift values. Story drift is the difference in horizontal deflection at the top and bottom of any story and corresponding result is tabulated as follow for each of stories.

Story	Chevron	V-brace	Comb. brace	K-brace	K mirror
Story 1	1.858	2.246	2.234	4.068	4.068
Story 2	2.302	2.734	2.479	4.995	4.995
Story 3	2.01	2.417	2.269	4.421	4.42
Story 4	1.388	1.765	1.605	3.17	3.17

Table 5.4: Story drift values of five bracings for four story building

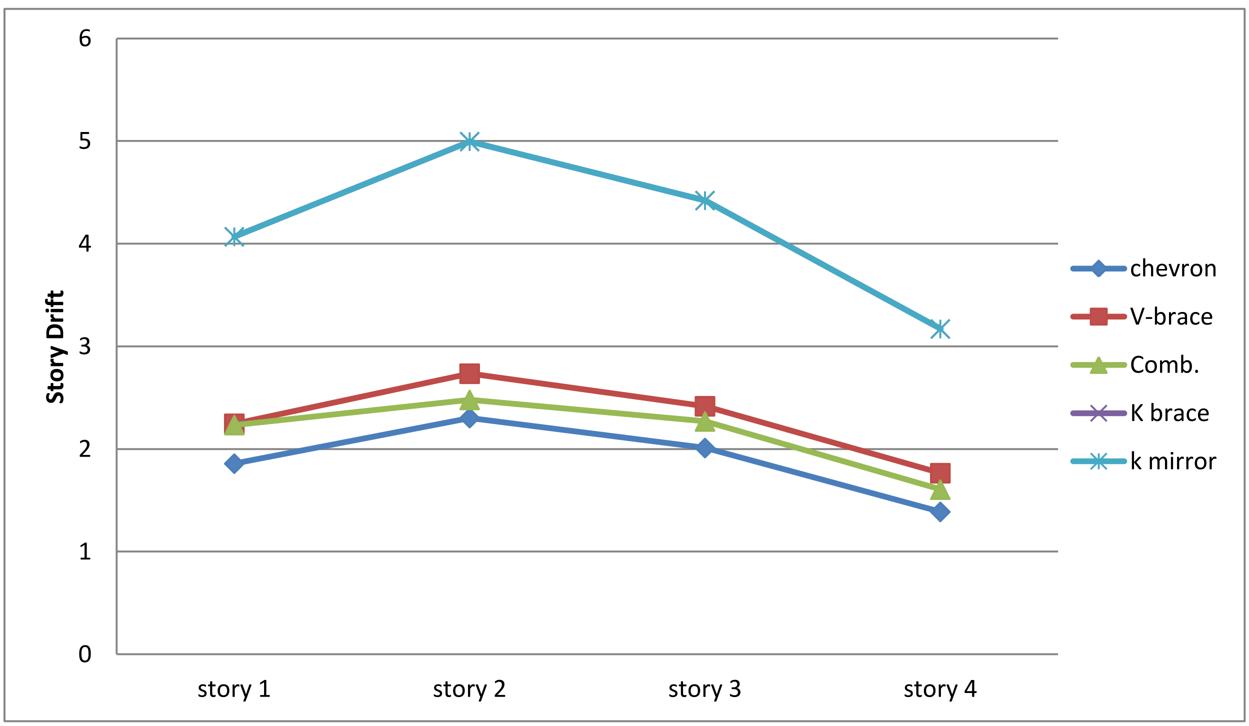


Figure 5.4: Plot of lateral displacement values of each bracing types for four story Building

Story	Chevron	V-brace	Comb. brace	K brace	K mirror
Story 1	2.408	2.769	2.849	5.168	5.168
Story 2	3.322	3.644	3.346	6.949	6.949
Story 3	3.456	3.702	3.664	7.171	7.17
Story 4	3.286	3.51	3.353	6.812	6.811
Story 5	2.837	3.081	2.957	5.943	5.943
Story 6	2.124	2.404	2.275	4.575	4.575

Table 5.5: Story drift values of five bracings for six story building

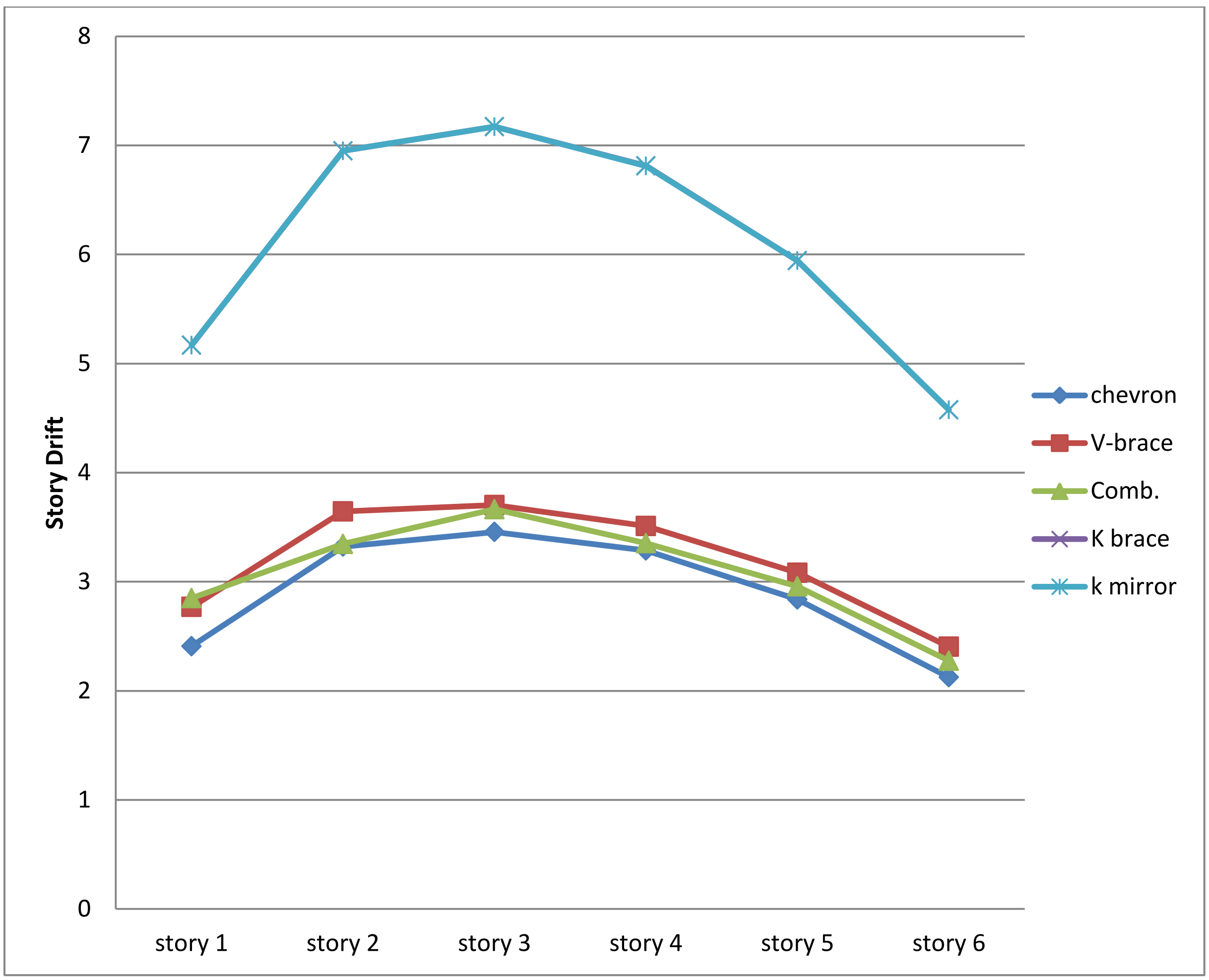


Figure 5.5: Plot of lateral displacement values of each bracing types for six story Building

Story	Chevron	V-brace	Comb. brace	K brace	K mirror
Story1	2.184	2.687	2.757	4.883	4.883
Story2	3.295	3.747	3.354	7.045	7.045
Story3	3.804	4.119	4.117	7.945	7.944
Story4	4.105	4.329	4.121	8.451	8.451
Story5	4.209	4.369	4.333	8.591	8.59
Story6	4.14	4.265	4.165	8.417	8.418
Story7	3.927	4.04	3.964	7.981	7.98
Story8	3.597	3.719	3.656	7.331	7.332
Story9	3.183	3.332	3.22	6.535	6.535
Story10	2.633	2.812	2.726	5.484	5.483

Table 5.6: Story drift values of five bracings for ten story building

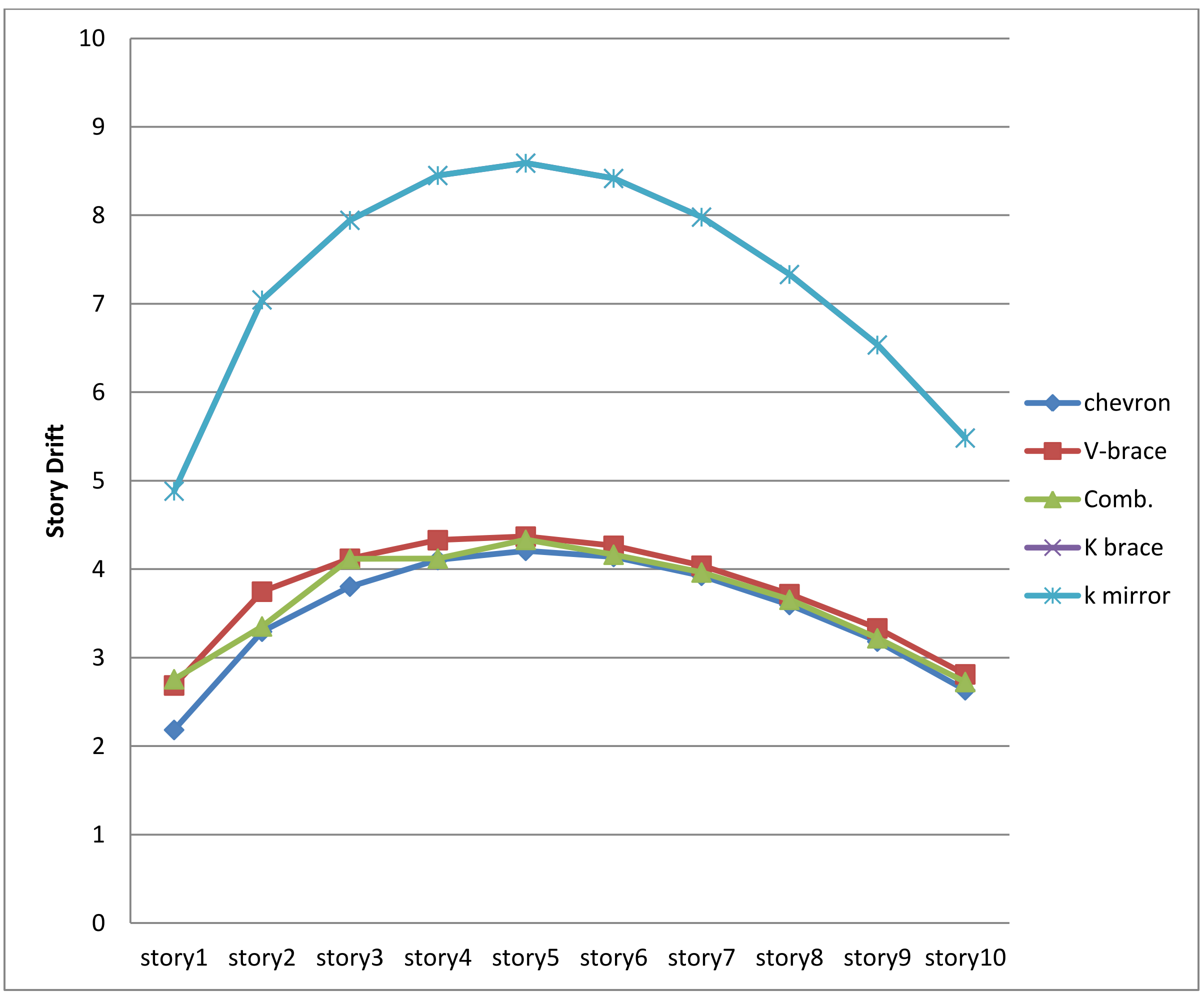


Figure 5.6: Plot of lateral displacement values of each bracing types for ten story Building

5.2 Discussion

In the selection of frame layout, bay width and story height of 4m is assumed. This is due to high ductility and strength to weight ratio of steel structures. In addition the presence of bracing gives an advantage to increase the spacing between columns.

From the Plot of lateral displacement values of each bracing types, it is discussed as follow:

- The lateral displacement values of K and mirror of K-bracing is exactly similar. This is observed from Plot of Figure 5.1 to 5.3 that they are overlapping with each other. It also gives higher values compared to other bracing types. This is due to weak point formation at the mid height of columns which is subjected for an unbalanced force coming from bracing member. This shows that lateral stiffness capacity of K and mirror of K-bracings are lesser compared to others bracing type.
- From comparison of chevron and K-bracing, lateral displacement of K-bracing is increased by an average of 52.5%. This implies that from structural stability point of view of this study, it is better to avoid k-bracings which may result in unbalanced force on columns. This leads to increase in columns dimension.
- From comparison of chevron and V-bracing, lateral displacement of V-bracing is increased by an average of 9.75%. This shows that there is slight increment of lateral displacement in V-bracing. This is happened due to the unbalanced force acting on the bottom beam which comes from two inclined bracing members.
- In Chevron bracing the lateral force is mainly resisted by the axial compression and tension effect of inclined bracing members which is shown in appendix Figure A1. From this Figure we can see that the advantage of bracing is to minimize stress on the neighborhood beams and columns.
- The output of lateral displacement values for combined V and inverted V bracing system are in between the two separate bracing results which verifies the expectations.
- While chevron bracing is used, the beam must be designed for an unbalanced load when the compression brace buckles. Often the resulting brace frame beam design weighing more.

- For each bracing systems, the story drift values will increase from one story to the next story up to then it reduces gradually for higher story levels.
- The above story drift values is checked and it is in the allowable range (as per Euro code 8 where cladding elements are rigidly attached to the structure, the SLS story drift is limited to 0.5% of storey height but this rise to 0.75% for rigidly attached ductile cladding). Compared to other bracing system, K and mirror of K- bracing has higher values of story drift.
- Maximum story drift is found at the middle story level of building height for all bracing systems.
- From the group of concentrically bracing systems, chevron bracing has smaller story drift which is seen in lateral displacement values.

6 CONCLUSIONS AND RECOMMENDATIONS

6.1 Conclusions

The following conclusions are drawn from the study;

- The lateral displacements of the structural system for the corresponding bracing type are determined from ETABS software analysis result. These lateral displacement values for the corresponding bracing are organized for steel buildings with four, six and ten stories.
- Comparisons is made initially by considering displacement values of each bracings found in a given story buildings separately. The plot of lateral displacement at each story level is done for each of three steel building structures. The main aim of considering three types of building with different number of stories is to know the behavior of bracing as the building height varies.
- It is found that chevron bracing performs better than the other four types of bracing .it gives higher resistance mechanism for the overall building structures which have lesser value of horizontal displacement of the structure compared to other bracings.
- As it is seen from comparison graphs, the behavior of each bracing doesn't change as the number stories increase from four to six and from six to ten stories. i.e for the three types of building considered, chevron bracing performs better. The next preference is the combination of chevron and v bracing followed by v bracing. K and mirror of K-bracing has lesser efficiency to resist lateral displacement of the bracing that creates a connection joint at the middle of column height. The graphs proofs all bracing behaves similarly even if the number of stories changes from four to ten stories.
- It is noticed that from comparison plots of each graph, the structural system that contain K and mirror of K-bracing are almost equal values of horizontal displacement. The percentage difference is less than 1%. The horizontal displacement that is found from these bracing is higher compared to the values found in chevron bracing. It has more than two times increment compared to chevron bracing.

6.2 Recommendations

This thesis work is an inch towards the complex phenomena to select the performance of various bracing types. The main purpose of this study is that when we compare different things first criteria of comparisons should be set to treats the given conditions equally.

Under this study the sample of the bracing type that I considered is classified under concentrically bracing. Among the possibilities for future study, the following are the main points that deserve attention.

- In this study it is only considered concentrical type of bracing. A study for eccentrical bracing type under similar criteria of comparison is left for future investigation.
- The analysis takes place by selecting a wide flange steel section for bracing cross section. The next researcher is expected to check the structural behavior under another cross-section like angle section, tubular section, etc.
- It is also considered that the structure here considered fulfils plan and elevation regularity, the behaviors for irregular structures under those bracing type can be considered for future study.

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APPENDICES

These are sample output for point displacement, Brace axial force and axial force diagram on chevron bracing.

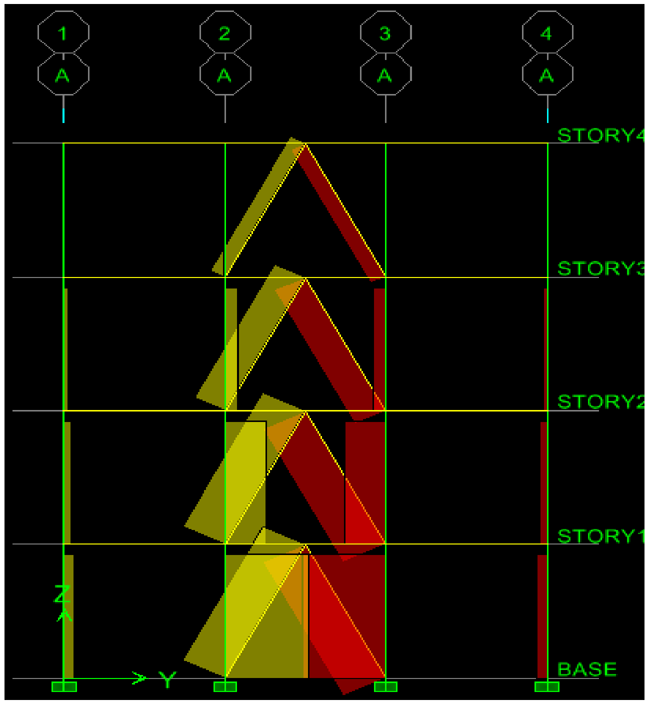

Figure A1: Axial force diagram on chevron bracing for EQy on axis A of elevation view. Yellow indicate in tension and red in compression. Its value is shown in table below.

Table A1: Brace axial force output for four stories building on chevron bracing due to EQy on axis A of elevation view

Story	Brace	Load	Loc(m)	P (KN)
STORY4	D1	EQY	0	2.61
STORY4	D1	EQY	2.236	2.61
STORY4	D1	EQY	4.472	2.61
STORY3	D1	EQY	0	5.17
STORY3	D1	EQY	2.236	5.17
STORY3	D1	EQY	4.472	5.17
STORY2	D1	EQY	0	7.1
STORY2	D1	EQY	2.236	7.1
STORY2	D1	EQY	4.472	7.1
STORY1	D1	EQY	0	7.08
STORY1	D1	EQY	2.236	7.08
STORY1	D1	EQY	4.472	7.08
STORY4	D2	EQY	0	-2.56
STORY4	D2	EQY	2.236	-2.56

STORY4	D2	EQY	4.472	-2.56
STORY3	D2	EQY	0	-4.94
STORY3	D2	EQY	2.236	-4.94
STORY3	D2	EQY	4.472	-4.94
STORY2	D2	EQY	0	-6.66
STORY2	D2	EQY	2.236	-6.66
STORY2	D2	EQY	4.472	-6.66
STORY1	D2	EQY	0	-6.43
STORY1	D2	EQY	2.236	-6.43
STORY1	D2	EQY	4.472	-6.43
STORY4	D3	EQY	0	-2.61
STORY4	D3	EQY	2.236	-2.61
STORY4	D3	EQY	4.472	-2.61
STORY3	D3	EQY	0	-5.17
STORY3	D3	EQY	2.236	-5.17
STORY3	D3	EQY	4.472	-5.17
STORY2	D3	EQY	0	-7.1
STORY2	D3	EQY	2.236	-7.1
STORY2	D3	EQY	4.472	-7.1
STORY1	D3	EQY	0	-7.08
STORY1	D3	EQY	2.236	-7.08
STORY1	D3	EQY	4.472	-7.08
STORY4	D4	EQY	0	2.56
STORY4	D4	EQY	2.236	2.56
STORY4	D4	EQY	4.472	2.56
STORY3	D4	EQY	0	4.94
STORY3	D4	EQY	2.236	4.94
STORY3	D4	EQY	4.472	4.94
STORY2	D4	EQY	0	6.66
STORY2	D4	EQY	2.236	6.66
STORY2	D4	EQY	4.472	6.66
STORY1	D4	EQY	0	6.43
STORY1	D4	EQY	2.236	6.43
STORY1	D4	EQY	4.472	6.43
STORY4	D5	EQY	0	42.68
STORY4	D5	EQY	2.236	42.68
STORY4	D5	EQY	4.472	42.68
STORY3	D5	EQY	0	86.52
STORY3	D5	EQY	2.236	86.52
STORY3	D5	EQY	4.472	86.52
STORY2	D5	EQY	0	119.04

STORY2	D5	EQY	2.236	119.04
STORY2	D5	EQY	4.472	119.04
STORY1	D5	EQY	0	118.55
STORY1	D5	EQY	2.236	118.55
STORY1	D5	EQY	4.472	118.55
STORY4	D6	EQY	0	-42.68
STORY4	D6	EQY	2.236	-42.68
STORY4	D6	EQY	4.472	-42.68
STORY3	D6	EQY	0	-86.52
STORY3	D6	EQY	2.236	-86.52
STORY3	D6	EQY	4.472	-86.52
STORY2	D6	EQY	0	-119.04
STORY2	D6	EQY	2.236	-119.04
STORY2	D6	EQY	4.472	-119.04
STORY1	D6	EQY	0	-118.55
STORY1	D6	EQY	2.236	-118.55
STORY1	D6	EQY	4.472	-118.55
STORY4	D7	EQY	0	47.81
STORY4	D7	EQY	2.236	47.81
STORY4	D7	EQY	4.472	47.81
STORY3	D7	EQY	0	96.6
STORY3	D7	EQY	2.236	96.6
STORY3	D7	EQY	4.472	96.6
STORY2	D7	EQY	0	132.79
STORY2	D7	EQY	2.236	132.79
STORY2	D7	EQY	4.472	132.79
STORY1	D7	EQY	0	132.06
STORY1	D7	EQY	2.236	132.06
STORY1	D7	EQY	4.472	132.06
STORY4	D8	EQY	0	-47.81
STORY4	D8	EQY	2.236	-47.81
STORY4	D8	EQY	4.472	-47.81
STORY3	D8	EQY	0	-96.6
STORY3	D8	EQY	2.236	-96.6
STORY3	D8	EQY	4.472	-96.6
STORY2	D8	EQY	0	-132.79
STORY2	D8	EQY	2.236	-132.79
STORY2	D8	EQY	4.472	-132.79
STORY1	D8	EQY	0	-132.06
STORY1	D8	EQY	2.236	-132.06
STORY1	D8	EQY	4.472	-132.06

Table A2: Point displacement for four stories on axis A at each story level

Story	Point	Load	UX	UY	UZ	RX	RY	RZ
STORY4	1	EQY	0.4349	8.4274	0.0553	-0.00012	0.00001	0.00007
STORY4	2	EQY	0.145	8.4274	0.3955	-0.00024	-0.00005	0.00007
STORY4	3	EQY	-0.4349	8.4274	-0.0553	-0.00012	-0.00001	0.00007
STORY4	4	EQY	-0.145	8.4274	-0.3955	-0.00024	0.00005	0.00007
STORY4	5	EQY	0.4349	7.5576	0.056	-0.00011	0.00001	0.00007
STORY4	6	EQY	0.145	7.5576	0.3555	-0.00022	0.00005	0.00007
STORY4	7	EQY	-0.145	7.5576	-0.3555	-0.00022	-0.00005	0.00007
STORY4	8	EQY	-0.4349	7.5576	-0.056	-0.00011	-0.00001	0.00007
STORY4	9	EQY	0.4349	7.8475	0.0576	-0.00019	0.00001	0.00007
STORY4	10	EQY	0.145	7.8475	0.0039	-0.00012	0.00002	0.00007
STORY4	11	EQY	-0.145	7.8475	-0.0039	-0.00012	-0.00002	0.00007
STORY4	12	EQY	-0.4349	7.8475	-0.0576	-0.00019	-0.00001	0.00007
STORY4	13	EQY	0.4349	8.1374	0.018	-0.00019	0.00001	0.00007
STORY4	14	EQY	0.145	8.1374	0.0035	-0.00012	-0.00002	0.00007
STORY4	15	EQY	-0.145	8.1374	-0.0035	-0.00012	0.00002	0.00007
STORY4	16	EQY	-0.4349	8.1374	-0.018	-0.00019	-0.00001	0.00007
STORY4	17	EQY	0.4349	8.2099	0.0204	-0.00017	-0.00001	0.00007
STORY4	18	EQY	0.145	8.2099	0.067	-0.00015	-0.00009	0.00007
STORY4	19	EQY	0.4349	8.2824	0.0367	-0.00016	-0.00002	0.00007
STORY4	20	EQY	0.145	8.2824	0.1828	-0.00018	-0.00012	0.00007
STORY4	21	EQY	0.4349	8.3549	0.0529	-0.00014	-0.00001	0.00007
STORY4	22	EQY	0.145	8.3549	0.3069	-0.00021	-0.00011	0.00007
STORY4	23	EQY	-0.145	8.2099	-0.067	-0.00015	0.00009	0.00007
STORY4	24	EQY	-0.145	8.2824	-0.1828	-0.00018	0.00012	0.00007
STORY4	25	EQY	-0.145	8.3549	-0.3069	-0.00021	0.00011	0.00007
STORY4	26	EQY	-0.4349	8.2099	-0.0204	-0.00017	0.00001	0.00007
STORY4	27	EQY	-0.4349	8.2824	-0.0367	-0.00016	0.00002	0.00007
STORY4	28	EQY	-0.4349	8.3549	-0.0529	-0.00014	0.00001	0.00007

STORY4	29	EQY	0.4349	7.63	0.0516	-0.00013	0	0.00007
STORY4	30	EQY	0.145	7.63	0.2744	-0.00019	0.0001	0.00007
STORY4	31	EQY	0.4349	7.7025	0.0566	-0.00015	-0.00001	0.00007
STORY4	32	EQY	0.145	7.7025	0.1655	-0.00017	0.00011	0.00007
STORY4	33	EQY	0.4349	7.775	0.0618	-0.00017	0	0.00007
STORY4	34	EQY	0.145	7.775	0.0637	-0.00014	0.00008	0.00007
STORY4	35	EQY	-0.145	7.63	-0.2744	-0.00019	-0.0001	0.00007
STORY4	36	EQY	-0.145	7.7025	-0.1655	-0.00017	-0.00011	0.00007
STORY4	37	EQY	-0.145	7.775	-0.0637	-0.00014	-0.00008	0.00007
STORY4	38	EQY	-0.4349	7.63	-0.0516	-0.00013	0	0.00007
STORY4	39	EQY	-0.4349	7.7025	-0.0566	-0.00015	0.00001	0.00007
STORY4	40	EQY	-0.4349	7.775	-0.0618	-0.00017	0	0.00007
STORY4	41	EQY	0.4349	7.92	0.0456	-0.00019	0.00001	0.00007
STORY4	42	EQY	0.145	7.92	-0.0121	-0.00012	0.00001	0.00007
STORY4	43	EQY	0.4349	7.9925	0.0357	-0.00019	0.00001	0.00007
STORY4	44	EQY	0.145	7.9925	-0.0163	-0.00012	0	0.00007
STORY4	45	EQY	0.4349	8.0649	0.0273	-0.00019	0.00001	0.00007
STORY4	46	EQY	0.145	8.0649	-0.0104	-0.00012	-0.00001	0.00007
STORY4	47	EQY	-0.145	7.92	0.0121	-0.00012	-0.00001	0.00007
STORY4	48	EQY	-0.145	7.9925	0.0163	-0.00012	0	0.00007
STORY4	49	EQY	-0.145	8.0649	0.0104	-0.00012	0.00001	0.00007
STORY4	50	EQY	-0.4349	7.92	-0.0456	-0.00019	-0.00001	0.00007
STORY4	51	EQY	-0.4349	7.9925	-0.0357	-0.00019	-0.00001	0.00007
STORY4	52	EQY	-0.4349	8.0649	-0.0273	-0.00019	-0.00001	0.00007
STORY4	53	EQY	0	7.5576	0	-0.00016	0	0.00007
STORY4	54	EQY	0	8.4274	0	-0.00018	0	0.00007
STORY3	1	EQY	0.3542	6.8786	0.0516	-0.00032	0.00002	0.00006
STORY3	2	EQY	0.1181	6.8786	0.3998	-0.00038	-0.00002	0.00006
STORY3	3	EQY	-0.3542	6.8786	-0.0516	-0.00032	-0.00002	0.00006
STORY3	4	EQY	-0.1181	6.8786	-0.3998	-0.00038	0.00002	0.00006
STORY3	5	EQY	0.3542	6.1702	0.0523	-0.00029	0.00002	0.00006
STORY3	6	EQY	0.1181	6.1702	0.3593	-0.00035	0.00003	0.00006
STORY3	7	EQY	-0.1181	6.1702	-0.3593	-0.00035	-0.00003	0.00006

STORY3	8	EQY	-0.3542	6.1702	-0.0523	-0.00029	-0.00002	0.00006
STORY3	9	EQY	0.3542	6.4064	0.0557	-0.00033	0.00002	0.00006
STORY3	10	EQY	0.1181	6.4064	0.0034	-0.00025	0.00002	0.00006
STORY3	11	EQY	-0.1181	6.4064	-0.0034	-0.00025	-0.00002	0.00006
STORY3	12	EQY	-0.3542	6.4064	-0.0557	-0.00033	-0.00002	0.00006
STORY3	13	EQY	0.3542	6.6425	0.0154	-0.00034	0.00002	0.00006
STORY3	14	EQY	0.1181	6.6425	0.0031	-0.00025	0	0.00006
STORY3	15	EQY	-0.1181	6.6425	-0.0031	-0.00025	0	0.00006
STORY3	16	EQY	-0.3542	6.6425	-0.0154	-0.00034	-0.00002	0.00006
STORY3	17	EQY	0.3542	6.7015	0.0153	-0.00034	-0.00001	0.00006
STORY3	18	EQY	0.1181	6.7015	0.066	-0.00029	-0.0001	0.00006
STORY3	19	EQY	0.3542	6.7605	0.0345	-0.00033	-0.00002	0.00006
STORY3	20	EQY	0.1181	6.7605	0.1928	-0.00032	-0.00013	0.00006
STORY3	21	EQY	0.3542	6.8196	0.0532	-0.00032	-0.00001	0.00006
STORY3	22	EQY	0.1181	6.8196	0.3238	-0.00035	-0.00011	0.00006
STORY3	23	EQY	-0.1181	6.7015	-0.066	-0.00029	0.0001	0.00006
STORY3	24	EQY	-0.1181	6.7605	-0.1928	-0.00032	0.00013	0.00006
STORY3	25	EQY	-0.1181	6.8196	-0.3238	-0.00035	0.00011	0.00006
STORY3	26	EQY	-0.3542	6.7015	-0.0153	-0.00034	0.00001	0.00006
STORY3	27	EQY	-0.3542	6.7605	-0.0345	-0.00033	0.00002	0.00006
STORY3	28	EQY	-0.3542	6.8196	-0.0532	-0.00032	0.00001	0.00006
STORY3	29	EQY	0.3542	6.2293	0.0452	-0.0003	0	0.00006
STORY3	30	EQY	0.1181	6.2293	0.2875	-0.00032	0.00009	0.00006
STORY3	31	EQY	0.3542	6.2883	0.0535	-0.00031	-0.00001	0.00006
STORY3	32	EQY	0.1181	6.2883	0.1741	-0.0003	0.00011	0.00006
STORY3	33	EQY	0.3542	6.3473	0.062	-0.00032	0	0.00006
STORY3	34	EQY	0.1181	6.3473	0.0643	-0.00027	0.00009	0.00006
STORY3	35	EQY	-0.1181	6.2293	-0.2875	-0.00032	-0.00009	0.00006
STORY3	36	EQY	-0.1181	6.2883	-0.1741	-0.0003	-0.00011	0.00006
STORY3	37	EQY	-0.1181	6.3473	-0.0643	-0.00027	-0.00009	0.00006
STORY3	38	EQY	-0.3542	6.2293	-0.0452	-0.0003	0	0.00006

STORY3	39	EQY	-0.3542	6.2883	-0.0535	-0.00031	0.00001	0.00006
STORY3	40	EQY	-0.3542	6.3473	-0.062	-0.00032	0	0.00006
STORY3	41	EQY	0.3542	6.4654	0.0393	-0.00033	0.00001	0.00006
STORY3	42	EQY	0.1181	6.4654	-0.0059	-0.00025	0	0.00006
STORY3	43	EQY	0.3542	6.5244	0.0301	-0.00034	0.00001	0.00006
STORY3	44	EQY	0.1181	6.5244	-0.0064	-0.00025	0	0.00006
STORY3	45	EQY	0.3542	6.5835	0.0259	-0.00034	0.00001	0.00006
STORY3	46	EQY	0.1181	6.5835	-0.0021	-0.00025	0	0.00006
STORY3	47	EQY	-0.1181	6.4654	0.0059	-0.00025	0	0.00006
STORY3	48	EQY	-0.1181	6.5244	0.0064	-0.00025	0	0.00006
STORY3	49	EQY	-0.1181	6.5835	0.0021	-0.00025	0	0.00006
STORY3	50	EQY	-0.3542	6.4654	-0.0393	-0.00033	-0.00001	0.00006
STORY3	51	EQY	-0.3542	6.5244	-0.0301	-0.00034	-0.00001	0.00006
STORY3	52	EQY	-0.3542	6.5835	-0.0259	-0.00034	-0.00001	0.00006
STORY3	53	EQY	0	6.1702	0	-0.00012	0	0.00006
STORY3	54	EQY	0	6.8786	0	-0.00013	0	0.00006
STORY2	1	EQY	0.2383	4.6362	0.0418	-0.0004	0.00003	0.00004
STORY2	2	EQY	0.0794	4.6362	0.3643	-0.00048	-0.00002	0.00004
STORY2	3	EQY	-0.2383	4.6362	-0.0418	-0.0004	-0.00003	0.00004
STORY2	4	EQY	-0.0794	4.6362	-0.3643	-0.00048	0.00002	0.00004
STORY2	5	EQY	0.2383	4.1596	0.0423	-0.00036	0.00003	0.00004
STORY2	6	EQY	0.0794	4.1596	0.3274	-0.00043	0.00003	0.00004
STORY2	7	EQY	-0.0794	4.1596	-0.3274	-0.00043	-0.00003	0.00004
STORY2	8	EQY	-0.2383	4.1596	-0.0423	-0.00036	-0.00003	0.00004
STORY2	9	EQY	0.2383	4.3185	0.0475	-0.00042	0.00003	0.00004
STORY2	10	EQY	0.0794	4.3185	0.0024	-0.00031	0.00002	0.00004
STORY2	11	EQY	-0.0794	4.3185	-0.0024	-0.00031	-0.00002	0.00004
STORY2	12	EQY	-0.2383	4.3185	-0.0475	-0.00042	-0.00003	0.00004
STORY2	13	EQY	0.2383	4.4773	0.0108	-0.00043	0.00002	0.00004
STORY2	14	EQY	0.0794	4.4773	0.0021	-0.00032	0	0.00004
STORY2	15	EQY	-0.0794	4.4773	-0.0021	-0.00032	0	0.00004
STORY2	16	EQY	-0.2383	4.4773	-0.0108	-0.00043	-0.00002	0.00004
STORY2	17	EQY	0.2383	4.517	0.0084	-0.00042	-0.00001	0.00004

STORY2	18	EQY	0.0794	4.517	0.0588	-0.00036	-0.00009	0.00004
STORY2	19	EQY	0.2383	4.5567	0.0277	-0.00041	-0.00002	0.00004
STORY2	20	EQY	0.0794	4.5567	0.1737	-0.0004	-0.00012	0.00004
STORY2	21	EQY	0.2383	4.5965	0.0463	-0.00041	-0.00001	0.00004
STORY2	22	EQY	0.0794	4.5965	0.2934	-0.00044	-0.0001	0.00004
STORY2	23	EQY	-0.0794	4.517	-0.0588	-0.00036	0.00009	0.00004
STORY2	24	EQY	-0.0794	4.5567	-0.1737	-0.0004	0.00012	0.00004
STORY2	25	EQY	-0.0794	4.5965	-0.2934	-0.00044	0.0001	0.00004
STORY2	26	EQY	-0.2383	4.517	-0.0084	-0.00042	0.00001	0.00004
STORY2	27	EQY	-0.2383	4.5567	-0.0277	-0.00041	0.00002	0.00004
STORY2	28	EQY	-0.2383	4.5965	-0.0463	-0.00041	0.00001	0.00004
STORY2	29	EQY	0.2383	4.1994	0.0336	-0.00037	0	0.00004
STORY2	30	EQY	0.0794	4.1994	0.2592	-0.0004	0.00009	0.00004
STORY2	31	EQY	0.2383	4.2391	0.0443	-0.00039	-0.00001	0.00004
STORY2	32	EQY	0.0794	4.2391	0.1571	-0.00037	0.0001	0.00004
STORY2	33	EQY	0.2383	4.2788	0.0553	-0.0004	0	0.00004
STORY2	34	EQY	0.0794	4.2788	0.0588	-0.00034	0.00008	0.00004
STORY2	35	EQY	-0.0794	4.1994	-0.2592	-0.0004	-0.00009	0.00004
STORY2	36	EQY	-0.0794	4.2391	-0.1571	-0.00037	-0.0001	0.00004
STORY2	37	EQY	-0.0794	4.2788	-0.0588	-0.00034	-0.00008	0.00004
STORY2	38	EQY	-0.2383	4.1994	-0.0336	-0.00037	0	0.00004
STORY2	39	EQY	-0.2383	4.2391	-0.0443	-0.00039	0.00001	0.00004
STORY2	40	EQY	-0.2383	4.2788	-0.0553	-0.0004	0	0.00004
STORY2	41	EQY	0.2383	4.3582	0.0277	-0.00042	0.00001	0.00004
STORY2	42	EQY	0.0794	4.3582	-0.0082	-0.00031	0.00001	0.00004
STORY2	43	EQY	0.2383	4.3979	0.0192	-0.00042	0	0.00004
STORY2	44	EQY	0.0794	4.3979	-0.0083	-0.00031	0	0.00004
STORY2	45	EQY	0.2383	4.4376	0.02	-0.00042	0	0.00004
STORY2	46	EQY	0.0794	4.4376	-0.0032	-0.00031	-0.00001	0.00004
STORY2	47	EQY	-0.0794	4.3582	0.0082	-0.00031	-0.00001	0.00004
STORY2	48	EQY	-0.0794	4.3979	0.0083	-0.00031	0	0.00004
STORY2	49	EQY	-0.0794	4.4376	0.0032	-0.00031	0.00001	0.00004
STORY2	50	EQY	-0.2383	4.3582	-0.0277	-0.00042	-0.00001	0.00004

STORY2	51	EQY	-0.2383	4.3979	-0.0192	-0.00042	0	0.00004
STORY2	52	EQY	-0.2383	4.4376	-0.02	-0.00042	0	0.00004
STORY2	53	EQY	0	4.1596	0	-0.00006	0	0.00004
STORY2	54	EQY	0	4.6362	0	-0.00007	0	0.00004
STORY1	1	EQY	0.1059	2.0695	0.0247	-0.00045	0.00003	0.00002
STORY1	2	EQY	0.0353	2.0695	0.2429	-0.00052	-0.00001	0.00002
STORY1	3	EQY	-0.1059	2.0695	-0.0247	-0.00045	-0.00003	0.00002
STORY1	4	EQY	-0.0353	2.0695	-0.2429	-0.00052	0.00001	0.00002
STORY1	5	EQY	0.1059	1.8577	0.025	-0.0004	0.00003	0.00002
STORY1	6	EQY	0.0353	1.8577	0.2183	-0.00047	0.00003	0.00002
STORY1	7	EQY	-0.0353	1.8577	-0.2183	-0.00047	-0.00003	0.00002
STORY1	8	EQY	-0.1059	1.8577	-0.025	-0.0004	-0.00003	0.00002
STORY1	9	EQY	0.1059	1.9283	0.0297	-0.00046	0.00003	0.00002
STORY1	10	EQY	0.0353	1.9283	0.001	-0.00032	0.00002	0.00002
STORY1	11	EQY	-0.0353	1.9283	-0.001	-0.00032	-0.00002	0.00002
STORY1	12	EQY	-0.1059	1.9283	-0.0297	-0.00046	-0.00003	0.00002
STORY1	13	EQY	0.1059	1.9989	0.0052	-0.00047	0.00002	0.00002
STORY1	14	EQY	0.0353	1.9989	0.0009	-0.00033	0	0.00002
STORY1	15	EQY	-0.0353	1.9989	-0.0009	-0.00033	0	0.00002
STORY1	16	EQY	-0.1059	1.9989	-0.0052	-0.00047	-0.00002	0.00002
STORY1	17	EQY	0.1059	2.0165	0.0011	-0.00046	-0.00001	0.00002
STORY1	18	EQY	0.0353	2.0165	0.0375	-0.00038	-0.00006	0.00002
STORY1	19	EQY	0.1059	2.0342	0.0176	-0.00046	-0.00002	0.00002
STORY1	20	EQY	0.0353	2.0342	0.1153	-0.00043	-0.00008	0.00002
STORY1	21	EQY	0.1059	2.0518	0.0328	-0.00045	0	0.00002
STORY1	22	EQY	0.0353	2.0518	0.1963	-0.00047	-0.00007	0.00002
STORY1	23	EQY	-0.0353	2.0165	-0.0375	-0.00038	0.00006	0.00002
STORY1	24	EQY	-0.0353	2.0342	-0.1153	-0.00043	0.00008	0.00002
STORY1	25	EQY	-0.0353	2.0518	-0.1963	-0.00047	0.00007	0.00002
STORY1	26	EQY	-0.1059	2.0165	-0.0011	-0.00046	0.00001	0.00002
STORY1	27	EQY	-0.1059	2.0342	-0.0176	-0.00046	0.00002	0.00002
STORY1	28	EQY	-0.1059	2.0518	-0.0328	-0.00045	0	0.00002

STORY1	29	EQY	0.1059	1.8754	0.015	-0.00041	0	0.00002
STORY1	30	EQY	0.0353	1.8754	0.1716	-0.00043	0.00006	0.00002
STORY1	31	EQY	0.1059	1.893	0.0264	-0.00043	-0.00001	0.00002
STORY1	32	EQY	0.0353	1.893	0.1045	-0.00039	0.00007	0.00002
STORY1	33	EQY	0.1059	1.9106	0.0382	-0.00044	0	0.00002
STORY1	34	EQY	0.0353	1.9106	0.04	-0.00036	0.00005	0.00002
STORY1	35	EQY	-0.0353	1.8754	-0.1716	-0.00043	-0.00006	0.00002
STORY1	36	EQY	-0.0353	1.893	-0.1045	-0.00039	-0.00007	0.00002
STORY1	37	EQY	-0.0353	1.9106	-0.04	-0.00036	-0.00005	0.00002
STORY1	38	EQY	-0.1059	1.8754	-0.015	-0.00041	0	0.00002
STORY1	39	EQY	-0.1059	1.893	-0.0264	-0.00043	0.00001	0.00002
STORY1	40	EQY	-0.1059	1.9106	-0.0382	-0.00044	0	0.00002
STORY1	41	EQY	0.1059	1.9459	0.0087	-0.00046	0.00001	0.00002
STORY1	42	EQY	0.0353	1.9459	-0.0073	-0.00032	0	0.00002
STORY1	43	EQY	0.1059	1.9636	0.0026	-0.00046	0	0.00002
STORY1	44	EQY	0.0353	1.9636	-0.0062	-0.00032	0	0.00002
STORY1	45	EQY	0.1059	1.9812	0.0104	-0.00047	0	0.00002
STORY1	46	EQY	0.0353	1.9812	-0.0016	-0.00033	0	0.00002
STORY1	47	EQY	-0.0353	1.9459	0.0073	-0.00032	0	0.00002
STORY1	48	EQY	-0.0353	1.9636	0.0062	-0.00032	0	0.00002
STORY1	49	EQY	-0.0353	1.9812	0.0016	-0.00033	0	0.00002
STORY1	50	EQY	-0.1059	1.9459	-0.0087	-0.00046	-0.00001	0.00002
STORY1	51	EQY	-0.1059	1.9636	-0.0026	-0.00046	0	0.00002
STORY1	52	EQY	-0.1059	1.9812	-0.0104	-0.00047	0	0.00002
STORY1	53	EQY	0	1.8577	0	0.00002	0	0.00002
STORY1	54	EQY	0	2.0695	0	0.00003	0	0.00002