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**COMPARATIVE STUDY ON THE RESPONSE OF RC FRAME STRUCTURE  
BRACED WITH DIFFERENT STEEL BRACING SYSTEMS**

**By**

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**Comparative study on the response of RC frame structure  
braced with different steel bracing systems**

A thesis submitted to the Graduate Studies of Addis Ababa University in Partial  
fulfillment of the requirements for the Degree of Master of Science (MSc) in  
Structural Engineering

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## **DECLARATION**

I, the undersigned, declare that this thesis is my own work, and that all sources of material used for the thesis have been duly acknowledged.

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## ABSTRACT

Multistory buildings are mostly affected by earthquake force in seismic prone area. The major concern in the design of multistory building is the structure mainly to have enough lateral stability to resist lateral forces and to control the lateral drift of the buildings. The steel bracing system in reinforced concrete frames is viable for resisting lateral forces, steel bracing is easy to erect occupies less space and has flexibility in design for meeting the required strength and stiffness.

In this study to know the importance of different bracing systems in the multistory RC frame structure a linear analysis of different story buildings are considered i.e., 4story (G+3), 6story (G+5), 9story (G+8), and 13story (G+12) of three bays by five bays RC building for seismic zone IV have been analyzed with four different types of steel bracing at the exterior of the frame in the same location and the same pattern. The bracings studied are diagonal brace, X-brace, V-brace and inverted V-brace (chevron brace) and performance of each frame has been carried out using linear static analysis.

Two types of arrangements has been used; 3x1 braced (3 bays out of 5 and 1 bay out of 3 are braced) and corner braced (corners are braced in all direction) and all frames are analyzed without bracing and with all types of bracing, a total of 40 different models are analyzed and compared using nonlinear finite element software ETABS 2015. Lateral story displacement, seismic base shear, and story drift of braced frames have been evaluated and compared with bare frame. The percentage reduction in lateral displacement and story drift is found out and also the increase in the seismic base shear and base moment capacity after the applications of all bracing.

Inverted V-bracing (chevron bracing) and X-bracing show the best performance in reduction of lateral displacement and story drift. X-bracing type is found to be the most in increasing the shear capacity of RC frame building which indicates that the X-brace type of steel bracing significantly contributes to the structural stiffness. Corner braced arrangement has shown better result in reduction of lateral displacement than 3x1 braced. From the analysis result one can

conclude that both X-bracing and Inverted-bracing system may be used to in new building or to retrofit existing damaged building. However, X-bracing system is more suitable to use.

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## CHAPTER ONE

### 1. INTRODUCTION

#### 1.1. Back ground of the study

The magnitude and intensities of earthquakes are varies from place to place causing low to severe destructive powers on engineered properties as well as giving rise to great economic losses and life threat. Steel bracing of RC frames has received some attention in recent years both as a retrofitting measure to increase the shear capacity of existing RC buildings and as a shear resisting element in the seismic design of new buildings. Earlier investigators focused on the retrofitting aspect of bracing and studied external bracing of buildings as well as internal indirect bracing of individual bays of the RC frames. Lately, the direct bracing of RC frames has attracted more attention since it is less costly and can be adopted not only for retrofitting purposes, but also as a viable alternative to RC shear walls at pre-construction design level. Experimental works, as well as analytical investigations have studied the capabilities of the direct bracing system of RC frames with encouraging results.

To resist lateral earthquake loads, shear walls are commonly used in RC framed buildings, whereas, steel bracing is the most often used in steel structures. In the past two decades, a number of reports have also indicated the effective use of steel bracing in RC frames. Steel bracing of RC buildings started as a retrofitting measure to strengthen earthquake-damaged buildings or to increase the load resisting capacity of existing buildings.

The bracing methods adopted in the past fall into two main categories, namely external bracing and internal bracing. In the external bracing system, existing buildings are retrofitted by attaching a local or global steel bracing system to the exterior frames. In the internal bracing method, the buildings are braced by incorporating a bracing system inside the individual bays of the RC frames. The bracing may be attached to the RC frame either indirectly or directly.

This thesis investigated the seismic performance of RC bare frame and steel braced RC frame, and the study has focused on four different steel bracings, such as diagonal brace, X-brace, V-

brace and inverted V-brace (chevron brace). Assessing the seismic capacity of RC frame by these steel bracing and have the better bracing systems has the main objective of this thesis.

## **1.2. Statement of the problem**

Most of the reinforced concrete structures were designed primarily for gravity loads. They were also designed for lateral forces that may be much smaller than that prescribed by the codes and this needs additional supporting element in the design of RC frame such as; bracing RC frame by internal steel bracing which act as shear resisting element for the design of new RC building. And for retrofitting of existing structure in the case of an inadequate seismic load capacity of existing RC structure. In such case some bracing systems are more effective or give better results than other bracing system in resisting all the horizontal loads, therefore among the main bracing systems which are most efficient in resisting seismic load?

## **1.3. Objective of the study**

### **1.3.1. General objective**

- 1) The main objective of this thesis is to compare and evaluate the effectiveness of steel braced reinforced concrete structure for different storied RC buildings by different types of bracing systems under earthquake lateral loads.
- 2) It is to identify the most efficient and suitable lateral loads resistant steel bracing types which give the minimum lateral displacements, minimum story drift and which increase shear capacity of RC frame from the selected groups of bracings types.
- 3) To suggest the better strengthened and retrofitting option for reinforced concrete structure frame design for seismic load resistance.

### **1.3.2. Specific objective**

- 1) Comparative study for different types of concentrically placed lateral load resisting systems (steel bracing system) at different locations.

- 2) To study the seismic behavior of RC building by performing linear static analysis with different bracing systems.
- 3) To compare various parametric results such as lateral story displacement, Story drift and Story forces for different types of bracing systems of braced RC frames in order that suitable types may be proposed for different seismic design and retrofit needs
- 4) To know the better seismic load resisting steel bracing system with respect to practical use.

#### **1.4. Scope of the study**

This study is limited to symmetric multistory building which is regular in plan and elevation of only 4story, 6story, 9story, and 13story RC frame buildings which has five bays by three bays. The frames are assumed to be firmly fixed at the bottom and the soil–structure interaction is neglected. The Concentric type of direct steel bracing systems has considered. The thesis result is based on analytical analysis by using finite element analysis software systems ETABS software.

#### **1.5. Thesis organization**

The thesis has organized in five chapters which are arranged as follows:

Chapter one deals with introductory parts which include background, objective, state of the problem, scope of the study and thesis organization.

Chapter two briefly reviews theoretical background of steel bracing systems for RC structure, classifications of bracing, and types of steel bracing.

Chapter three mainly focus on modeling input data's and describe about the analysis method and loading considered based on EBCS EN 1998-1-2014 and other EBCS EN procedures.

Chapter four presents the analysis result of all stored building and finally present comparison and discussion for lateral displacement, story drift, base shear and base overturning moment for each of the bracing type.

Finally, in chapter five conclusions drawn and recommendation is presented.

## CHAPTER TWO

### 2. LITERATURE REVIEW

#### 2.1. Lateral load resisting system

Buildings are subjected to two types of load (1) Vertical load due to gravity, and (2) Lateral load due to earthquake and wind. The structural system of the building has to cater for both the types of load. The structural system of a building may also be visualized as consisting of two components (i)Horizontal framing system, consisting of slabs and beams, which is primarily responsible for transfer of vertical load to the vertical framing system and (ii)Vertical framing system, consisting of beams and columns, which is primarily responsible for transfer of lateral load to foundation. However the two components work in conjunction with each other. <sup>[24]</sup>

The selection of the lateral load resisting system (LLRS) for a building structure is a key decision in seismic design. Not every structural component in a building is designed to resist seismic loads. Rather, some building elements are designed only for gravity or vertical loads. The LLRS includes the components selected by the designer to resist the lateral forces acting on the building. The selection of this system depends on many factors: the preference of the designer, design specifications, the construction materials, costs, and the height of the structure. Increasing the height of a building increases its sensitivity to both wind and earthquake forces. The taller the building, the more these forces will dominate the design of the entire structure, and the more attention should be given to the designing of them. <sup>[18]</sup>

The conventional arrangements of stabilizing elements used in low-rise buildings may be extended for use in buildings up to 20 to 25 stories in height (Allen and Iano 1995). The same considerations that apply to low-rise buildings apply to taller buildings as well. Stabilizing elements should be arranged so as to resist lateral forces along all major axes of the building. These elements should be arranged in a balanced manner either within the building or at the perimeter and such elements must be integrated with the building plan of elevation.

Shear walls and braced frames are the stabilizing elements most commonly used in buildings of medium height, due to their structural efficiency. They may be used either separately or in combinations. The use of rigid frames as the sole means of stabilizing structures of medium height is possible, although this may be less than desirable because of the large size of the beams and columns that are generally required. The most common forms of LLRS for low to medium rise buildings are structural shear walls, bracing system and rigid frame systems.

## **2.2. Shear Wall System**

Khajehpour<sup>[2]</sup> described Shear Walls as stiff structures with high ductility which keeps the deformations of non-ductile framing systems in the elastic range. He noted that the in-plane load resistance is the principal strength of shear walls and that the resistance against both gravity and lateral loads can be assigned to shear walls if they are appropriately located in a building. Shear walled frames can be used from 10story up to 50story, is more economical and efficient for building above 30story. <sup>[26]</sup>

## **2.3. Moment Resisting Frame System**

Moment Resisting Frame is suitable where the presence of Shear Wall is undesirable, especially in situations where architectural limitation is imposed. The results of Khajehpour's work <sup>[2]</sup> revealed that Moment Resisting Frame is economically up to 20stories while submitted that it is economical for buildings up to 25stories, above which their behaviour is costly to control. The rigid frame is economical up to approximately 30stories for steel buildings and up to 20stories for concrete buildings. <sup>[26]</sup>

### **2.3.1. Response of rigid RC frame structure for seismic load**

Earthquakes are the most destructive and life damaging phenomenon of all the times. Earthquakes are caused due to the large release of strain energy by the movement of faults, which causes shaking of the ground as the seismic waves travel in all the directions inside the earth layer. These seismic waves will carry different levels of energy, have different amplitudes and arrive at various instants of time to the surface. Earthquake can be classified based on its size and occurrence into minor, moderate and strong depending on the severity of

ground shaking during the earthquake event. Magnitude (M) is the parameter which is used to measure the size of the earthquake which is recorded on Seismograms. For the same magnitude, shaking of ground will have different intensity at different locations. This is measured in MMI scale (Modified Mercalli Intensity).

When an earthquake does occur, different buildings located on the same site will show variation in the level of performance experienced by them as shown in figure below. This variation in levels depend on many factors such as random differences in the material strength, the amount of mass and stiffness of structural and non-structural members, levels of workmanship, condition of each structure, intensity and distribution of live load at the time of earthquake and response of the soil beneath the buildings.

An ideal multi-story building which is designed to resist lateral loads due to earthquake would be symmetric in distribution of mass and stiffness in plan at every story and as well as along the height of the building. Such building would respond only laterally and is considered as torsional balanced building. Because of restrictions as architectural requirement and functional needs, it is very difficult to achieve such a condition in the building.

Now-a-days the buildings with irregular configurations in both plan and elevation are seen. These buildings with asymmetry will suffer severely during earthquake<sup>[3]</sup>.

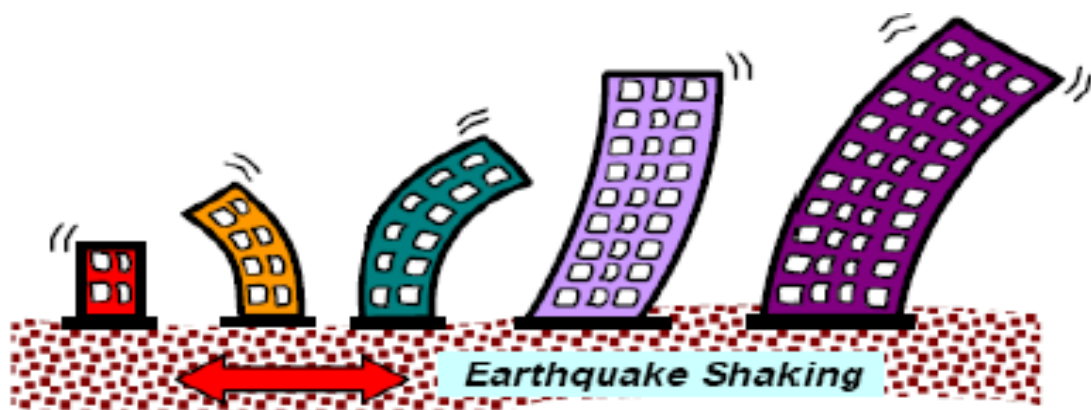


Figure 2.1: Seismic response of different buildings

## **2.4. Bracing system**

Bracing systems are used to resist horizontal forces (seismic action, wind load) and to transmit to the foundation. The bracing members are arranged in many forms, which carry solely tension, or alternatively tension and compression. Such systems reduce bending moment and shear force in the columns. Bracings hold the structure stable by transferring the loads sideways (not gravity, but earthquake or wind loads) down to the ground and are used to resist lateral loads, thereby preventing sway of the structure. Diagonal braces are efficient elements for developing stiffness and resistance to lateral load. There are different types of bracing systems in common use such as single diagonal bracing, X bracing, V bracing, K bracing, inverted V bracing. <sup>[4]</sup>

## **2.5. Steel bracing system**

Existing RC framed buildings designed without seismic criteria and ductile detailing can represent a considerable hazard during earthquake ground motions. The non-ductile behavior of these frames derives from the inadequate transverse reinforcement in columns, beams and joints, from bond slip of beam bottom reinforcement at the joint, from the poor confinement of the columns. In the presence of these deficiencies the upgrading of seismic performance may be realized with two different approaches,

1. The introduction of new structural members such as steel bracing systems or RC shear walls.
2. The local strengthening of some structural elements with the use of concrete, steel and fiber reinforced plastic.

The first approach is realized with the introduction of steel braces in steel structures and of RC shear walls in RC structures. However, the use of steel bracing systems for RC buildings may have both practical and economic advantages. In particular, this system offers advantages such as the ability to accommodate openings and the minimal added weight of the structure. Furthermore, if it is realized with external steel systems (External Bracing) the minimum disruption to the full operationally of the building is obtained. <sup>[5]</sup>

The idea of steel bracing system application to reinforced concrete buildings was first suggested for seismic strengthening of concrete buildings. From the viewpoint of both research and application, this idea has been very prevalent during past two decades because of the simplicity of its implementation and its relatively lower cost compared to shear wall. For example, Sugano and Fujimura<sup>[7]</sup> performed a series of experiments on a model of one-story frame which had been strengthened through various methods. They examined the frame samples with X- and K-Shape bracing systems and compared them to the samples strengthened by concrete- and masonry-in filled walls. They aimed to determine the effect of each of these systems on enhancement of in-plane strength and ductility of the samples<sup>[7]</sup>

In 1999, the direct internal use of steel bracing system in concrete frame was studied in laboratory. Experiments were carried out on five one-span one-story frame samples with a scale of 1: 2.5. Two of them had no bracing system but the other three samples were strengthened by X-bracing systems with different component connectors including bolt and nut, cover of RC column, and plates placed in concrete. The prepared frames were exposed to constant gravity and lateral cyclic loadings. Results showed, depending upon various component connectors, the bracing system considerably increases the equivalent stiffness of the frame and notably changes its behavior. When the bracing connector is implanted inside concrete, the performance of frame gets even better and further energy is absorbed.

Generally, experiments demonstrated that bracing tolerates a major part of lateral load in reinforced concrete frame.<sup>[6]</sup> A ten-story reinforced concrete structure is studied in four cases. Results revealed that the existence of shear walls extensively enhances structure's stiffness and strength but decreases its ductility. In comparison with shear wall, the application of steel bracing system increases the structure's ductility, although it has negligible effect on its strength.<sup>[7]</sup>

Maheri and Akbari<sup>[8]</sup> first reviewed previous studies on strengthening by steel bracing systems and then investigated three models including a simple frame, a frame strengthened with X-bracing, and a frame strengthened with knee bracing system under lateral load until failure stage. They found that ductility of RC frame considerably increases when using knee bracing system.

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 as well as one of the major retrofit measures. 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. A number of researchers have investigated various techniques such as infilling walls, adding walls to existing columns, encasing columns, and adding steel bracing to improve the strength and/or ductility of existing buildings. 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. Steel braced frames are efficient structural systems for buildings subjected to seismic or wind lateral loadings. Therefore, the use of steel bracing systems as shear resisting system of new designed building and for retrofitting reinforced concrete frames with inadequate lateral resistance is attractive. <sup>[24]</sup>

Braced-frames virtually eliminate the columns and girder bending factors and thus improve the efficiency of the pure rigid frame actions. By the addition of truss members such as diagonals (between the floor systems) this can be achieved effectively. These diagonals carry the lateral loads and transfers the axial loads to the columns, which is an effective structural system. <sup>[23]</sup>

Steel bracing is generally used to increase the lateral load resistance of steel structures. In recent years, the concept of steel bracing has also been applied to the retrofitting of reinforced concrete frames. Increased architectural flexibility, reduced weight of the structure, ease and speed of construction and the ability to choose more ductile systems can be considered as the main advantages of steel bracing in comparison with RC shear walls. Two bracing systems are generally used, external bracing and internal bracing. <sup>[9]</sup>

Bracing members are widely used in steel frames due to their ability to reduce lateral displacements and dissipate energy during strong ground motions. There are two options available in the reinforced concrete frames: to use either reinforced concrete bracing members or steel bracing members. It appears that both options are in use, but such braced frames are not popular because their seismic response is not well understood. The 47-storey Place Victoria

concrete building in Montreal, Canada, has concrete bracing members to resist lateral forces. Recent studies show that different aspects of reinforced concrete braced frames are studied and concluded that their seismic response is much better as compared to that of moment resisting frames. In braced frames the lateral resistance of the structure is provided by diagonal members that together with the beams form the web of the vertical truss with the columns acting as chords. Because the horizontal shear on the building is resisted by the horizontal components of the axial tensile and compressive actions in the web members, bracing systems are highly efficient in resisting lateral loads.

Bracing is generally regarded as an exclusive steel system but now a day steel bracings are also used in reinforced concrete frames. The efficiency of bracing in 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 fully triangulated bracing is that the beams usually participate only minimally in the lateral bracing action. <sup>[24]</sup>

## **2.6. Method of bracing**

### **2.6.1. External bracing**

In external bracing, steel trusses or frames are attached either as a global external support to the building exterior or, more locally, to the face of individual building frames. A number of investigators have reported on the efficiency of external bracing in seismic retrofitting of existing RC buildings. Architectural concerns and difficulties in providing appropriate connections between the bracing system and RC frames are two of the shortcomings of this method. <sup>[9]</sup>

### **2.6.2. Internal bracing**

In internal bracing, steel bracing members are inserted in the empty space enclosed by columns and beams of RC frames. As a result, each unit frame is individually braced from within. The bracing may be attached to the RC frame either indirectly or directly. <sup>[9]</sup>

Internal steel bracing of RC frames with direct connections has received some attention in recent years, both as a retrofitting measure to increase the shear capacity of the existing RC buildings and as a shear resisting element in the seismic design of new buildings. Although its successful use to upgrade the lateral load capacity of existing reinforced concrete frames has been the subject of a number of studies, guidelines for its use in newly constructed RC frames need to be further developed. In internal bracing, steel trusses or bracing members are inserted in the empty space enclosed by columns and beams of RC frames.<sup>[10]</sup>

Recent works by Ghaffarzadeh and Maheri<sup>[10]</sup> (2006) have shown further that the directly-connected internal bracing systems can be used effectively in retrofitting of existing concrete frames as well as shear resisting elements for construction of new RC structures. In this study, the use of X-shaped concentric internal steel bracing for new construction was investigated experimentally. An important consideration in the design of steel braced RC frames is the level of interaction between the strength capacities of the RC frame and the bracing system. And on the experiment the level of interaction between bracing steel and RC frame is investigated.



Figure; 2.2 Setup for cyclic testing of internal brace specimens

## 2.7. Types of bracing

### 2.7.1. Horizontal bracing system

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.

Horizontal floor diaphragms Floor slabs, roof slabs, and cast-in-place toppings on precast concrete floors may all be used as diaphragms to transfer horizontal forces acting in the building to the lateral force- resisting system and, eventually, to the foundations.<sup>[25]</sup>

### 2.7.2. Vertical bracing system

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



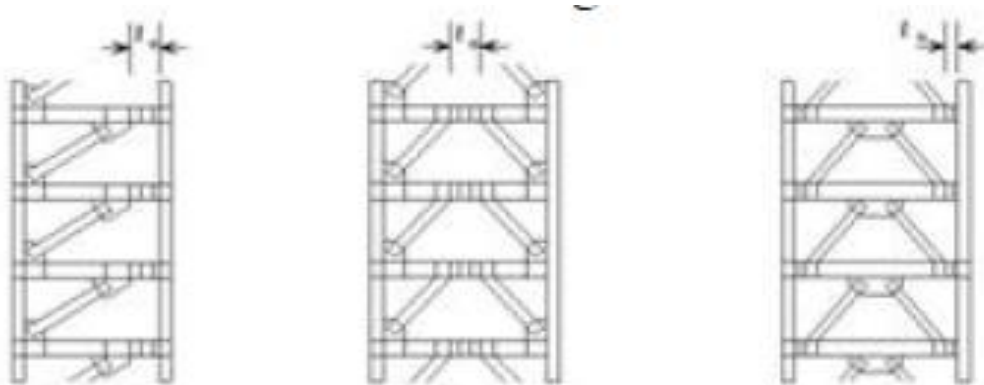
Figure; 2.3 Concentric and Eccentric bracing

### 2.7.3. Eccentric bracing system

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.<sup>[5]</sup>

Eccentrically braced frames (EBF) are a hybrid lateral load resisting systems. In fact it can be considered as a superposition of two different frame systems: conventional moment –resisting frames (MRFs) and concentrically braced frames (CBFs). EBFs can be combine the main advantages of the two systems (MRFs and CBFs), while minimizing their respective disadvantages. In general EBFs are possessing high elastic stiffness, stable inelastic response under cyclic lateral loading, and excellent ductility and energy dissipation capacity. EBFs in the building typical include the use of shear link, which are section of beam that yield and plastically deform in shear, to provide a stiff and ductile lateral load resisting system.

In an eccentrically braced frame bracing members connect to separate points on the beam/girder. The beam/girder segment or “link” between the bracing members absorbs energy from seismic activity through plastic deformation.<sup>[12]</sup>



Figure; 2.4 typical arrangement of eccentric bracing



*Figure 2.31. Two BRADs installed in the new building of the University of Ancona (Antonucci et al. 2006).*

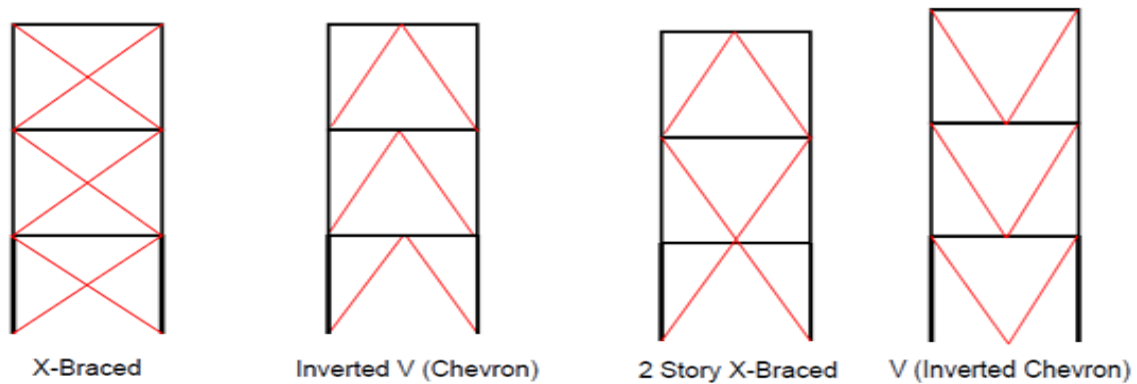
Figure; 2.5 eccentrically braced building

#### 2.7.4. Concentric bracing system

The Bracing is concentric when the center lines of the bracing members intersect. 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 on RC frame using concentric steel bracings. From Various types of concentric bracing some are:

- 1) *V brace*- Bracing where a pair of braces, located both above beam, terminates at a single point within the clear beam span.
- 2) *Inverted V brace*- bracing is that form of chevron bracing that intersects a beam from below.
- 3) *X braces* - Bracing where a pair of diagonal braces crosses near mid-length of the bracing members.

- 4) *K brace*- Bracing where a pair of braces located on one side of a column terminates at a single point within the clear column height.<sup>[12]</sup>



Figure; 2.6 configuration of different bracing

### 2.7.5. Indirect concentric bracing

In the indirect internal bracing, a braced steel frame is positioned inside the RC frame. As a result, the transfer of load between the steel bracing and the concrete frame is carried out indirectly through the steel frame. Successful retrofits of existing buildings by indirect internal bracing using different forms of X, V and K concentric and eccentric braces have been reported in different literatures. In some repair and retrofitting cases, provision of the steel frame may be necessary to reduce the strength demand on an already damaged and weakened RC frame; however, in other instances the steel frame acts only as a costly connecting mechanism with inhibiting technical difficulties in fixing the steel frame to the RC frame.<sup>[9]</sup>

### 2.7.6. Direct concentric bracing

To overcome the shortcomings of the indirect internal bracing, Maheri and Sahebi [9] first recommended using direct connections between the brace elements and RC frame without the need for an intermediary steel frame. In an experimental work, they showed the ability of this bracing system to enhance the strength capacity of RC frames. Later experimental work on directly braced model frames by Tasnimi and Masoomi<sup>[6]</sup> also showed the applicability of this method.

In continuation of their previous work, Mahmoud R. Maheri<sup>[9]</sup> conducted experimental investigations on pushover response of scaled RC frames; braced with both diagonal bracing and knee bracing systems. In this study the effectiveness of the two bracing systems in increasing some seismic performance parameters was shown. Also, in a theoretical study, Maheri and Akbari<sup>[8]</sup> presented the behaviour factor, R, for this class of dual systems. Appropriate design of direct connections between the bracing members and the RC frame is important to achieve the required lateral load capacity. Recent experimental works by Ghaffarzadeh and Maheri<sup>[11]</sup> have shown further that different directly-connected internal bracing systems can be used effectively in retrofitting of existing concrete frames as well as shear resisting elements for construction of new RC structures.



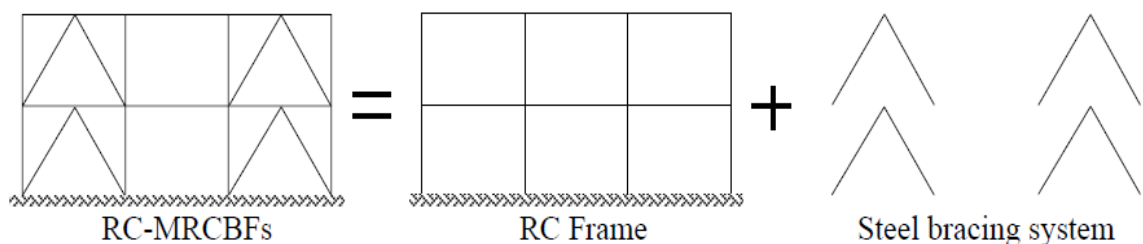
Figure; 2.7 direct concentric bracing

## 2.8. Braced frame system

### 2.8.1. Centrally Braced Frames

Centrally Braced Frames (CBFs) are a class of structures resisting lateral loads through a vertical concentric truss system, the axes of the members aligning concentrically at the joints. CBFs tend to be efficient in resisting lateral forces because they can provide high strength and stiffness. These characteristics can also result in less favorable seismic response, such as low drift capacity and higher accelerations.<sup>[13]</sup>

Due to the good behavior observed in the retrofitted RC structures by the addition of steel bracing in Mexico City, it is attractive to consider using this system for original design as well. In fact, the seismic guidelines of Mexico's Federal District Code (MFDC-04) allow for more than 30 years the design of a moment-resisting reinforced concrete concentric braced frames structures (RC-MRCBFs), considering even the possibility of a ductile behavior. However, in the design practice, this system has almost never been used for new construction. Moment frames at all the stories must resist, without the bracing system contribution, at least 50% of the seismic force<sup>[14]</sup>



Figure; 2.8 how seismic shear force is resisted

Viswanath K.G et al<sup>[15]</sup> (2010) studied on seismic analysis of steel braced reinforced concrete frames. He studied the seismic performance of reinforced concrete (RC) buildings rehabilitated using concentric steel bracing. For peripheral columns the bracings were provided. A 4story building was analyzed for seismic zone IV as per IS 1893: 2002 using STAAD Pro software in his paper. It was examined the effectiveness of various types of steel bracing in rehabilitating a 4story study. The seismic performance of the rehabilitated building was studied on the effect of the distribution of the steel bracing along the height of the RC frame. In terms of global and story drifts the performance of the building was evaluated. His result shows the reduction of percentage in the lateral displacement. Then he concluded that the X type of steel bracing significantly contributed to the structural stiffness and reduces the maximum inter story drift of the frames.

Madhusudan et al.<sup>[16]</sup> (2014) had studied on the effect of a provision of concentric bracings on the seismic performance of the steel frames and in the study they considered the two different types of concentric bracings (viz. X and inverted-V type bracing) for the different story levels.

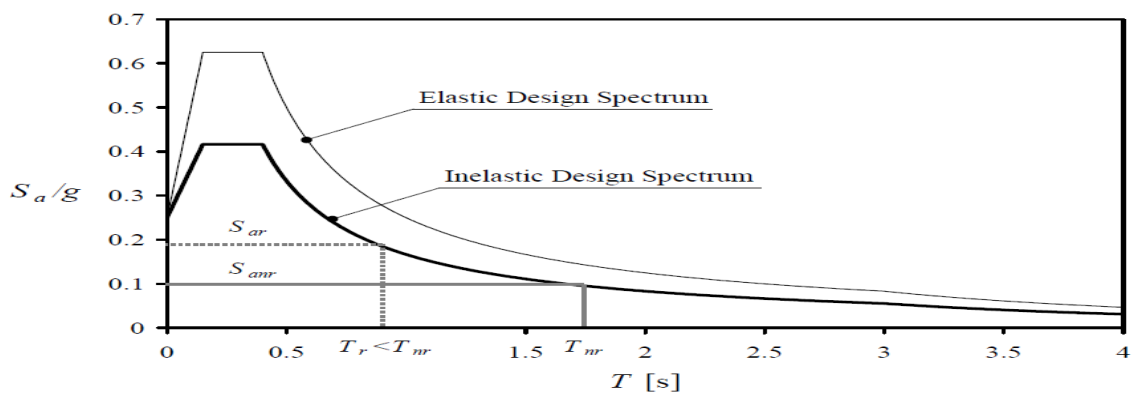
For this purpose, ETABS, Finite Element software has been used and the comparison between the performances of 1- bay X and inverted-V type and bare frames is made using pushover curves. To analyze the seismic performance of the frames they considered base shear, roof displacement and the number of hinges formed. Observation showed that steel bracing can be used as a retrofit for the existing structure. They found that inclusion of bracing increased the base shear capacity and decreased the roof displacement and also reduced the inter story drift. The lateral story displacements of the building are reduced by the use of inverted-V bracing in comparison to the X bracing system. Steel frames with insufficient lateral stiffness can be retrofitted with braces.

Nauman Mohammed et al <sup>[17]</sup> (2013) studied on Behavior of Multistory RCC Structure with Different Type of Bracing System (A Software Approach). They aims to evaluate the response of braced and unbraced structure subjected to seismic loads and to identify the suitable bracing system for resisting the seismic load efficiently. A G+14 floors building were analyzed using STAAD V8i software for special moment resisting frame situated in zone 4. The RCC G+14 structure was analyzed for both without bracings and with different types of bracings system. For all type of structural systems i.e. braced and unbraced structural system bending moments, shear forces, story shears, story drifts and axial forces was compared. They had been concluded that the displacement of the structure decreased after the application of the bracing system. After the application of cross bracing system the maximum reduction in the lateral displacement occurs. In the columns bracing system reduces bending moments and shear forces. The paper also states that the execution of cross bracing system was better than the other specified bracing systems. To retrofit the existing structure steel bracings were used. Significantly after the application of the bracings, total weight of the existing structure will not be changed. <sup>[17]</sup>

## **2.9. Retrofitting**

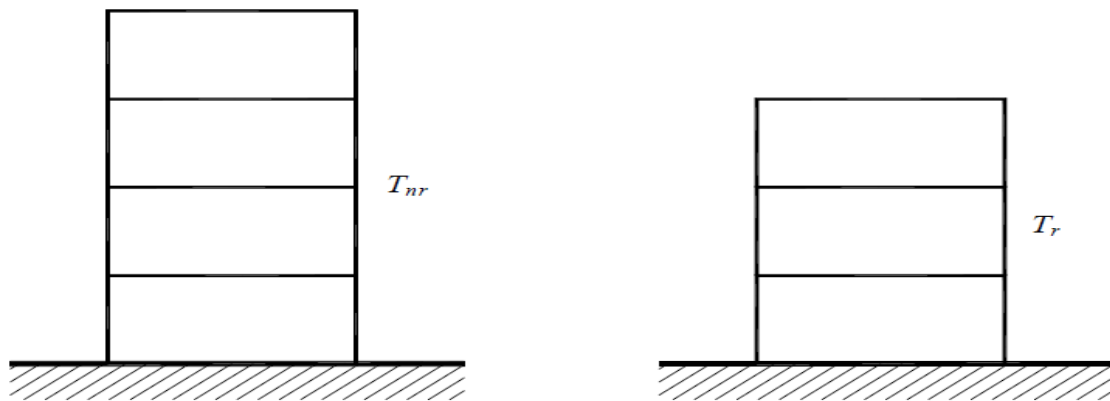
Traditional methods of seismic retrofitting fall essentially into two categories, one based on the classical principles of structural design which requires an increase of strength and stiffness, and the other based on mass reduction. Thus the first one tends to satisfy the design inequality by an increase of the capacity while the second one achieves the same result by a reduction of the

demand. Since seismic design is different from ordinary design, both techniques may turn out to be quite ineffective as is shown in the following. With reference to the first method, that is increase of strength and stiffness, the concept involved in its application can be understood using Figure below. Suppose that the fundamental period of the structure is  $T_{nr}$ , to which corresponds a demand  $S_{anr}$  in pseudo acceleration terms, which the structure cannot satisfy. On applying a strength and stiffness increment, the fundamental period will shorten from  $T_{nr}$  to  $T_r$ , to which corresponds a demand  $S_{ar}$  much larger than the original one. It is, therefore, possible that the structure will be less safe in the new condition than in the original one. Only after stiffness and strength have been increased up to a level where the fundamental period corresponds to the constant branch of the design spectrum, is it possible to achieve a condition where the design inequality is satisfied. It is, therefore, evident that an attempt to increase the seismic resistance capacity in this way only results in an increase of the seismic demand. When, in the end, the procedure converges, it is at the expense of a considerable expenditure of resources.



Figure; 2.9 increase of seismic demand following an increase of seismic resistance

A similar situation occurs with reference to mass reduction. This may be achieved, for instance, by removal of one or more storeys as shown in Figure below. In this case it is evident that the removal of the mass will lead to a decrease in the period, i.e.  $T_r < T_{nr}$ , which will lead to an increase in the required strength, i.e.  $S_{ar} > S_{anr}$ . Therefore the advantage acquired by the mass reduction is partially cancelled by the period shortening through the increase in the demand as shown in Figure above.



Figure; 2.10 seismic retrofitting by mass reduction

In conclusion, both of the traditional methods of seismic retrofitting, although effective, are rather expensive. It must however be pointed out that, as in the case of low buildings, the fundamental period may already fall within the constant branch of the design spectrum and a period shortening may not result in an increase of the seismic action. <sup>[20]</sup>

### 2.9.1. Stiffening and strengthening the whole structure

Interventions at the structure's level to increase global stiffness and reduce seismic deformation demands throughout the system may be more cost-effective than universal upgrading of the capacities of the existing components, if disruption of occupancy and demolition and replacement of partitions, architectural finishes and other interior non-structural components are considered.

#### 1) Addition of new concrete wall

Adding concrete walls is perhaps the most common technique for seismic retrofitting of buildings. It is very effective for the control of global lateral drifts and the reduction of damage in frames and non-structural elements. Full continuity of the wall across stories increases its strength. Good anchorage of the new reinforcement is essential for strength and deformation capacity. New walls should have proper foundation. As it typically has large cross-section, a new wall is expected to develop high seismic moments at the base. At any rate, the most serious problem and drawback of the technique is the difficulty to transfer the wall base

moment to the ground and the need of a major, costly and disruptive intervention to the foundation.

## 2) Addition of steel bracing

Adding diagonal bracings to selected bays in all stories of a frame structure, or to just one or few weak stories is effective for global strengthening and normally not as disruptive as adding walls. Normally the intervention to the foundation is minimal. The prime challenge in this technique is the connection of the bracings to the existing concrete elements. Bracings are normally made of structural steel. Steel bracing increases markedly the lateral force resistance of a concrete frame.

Steel bracing systems can be applied for retrofitting as concentric contributing to lateral-load resistance and stiffness and to energy dissipation through the axial forces in their inclined braces.<sup>[18]</sup>

## 2.10. Connection steel brace with RC

### 2.10.1. Indirect connection: load transfer through mortar joint

The first method investigated is to transfer the earthquake loads from the concrete to the steel frame through a mortar joint. In this case the steel frame is equipped with headed studs at its circumference and post-installed anchors with headed ends are fixed to the concrete structure. When the steel frame has been placed, spiral reinforcement is put into the space between frame and concrete and a non-shrink grout is injected into the space between the existing concrete member and the steel frame in order to connect headed studs and post-installed anchors.



Figure 1 - Frame prepared for indirect

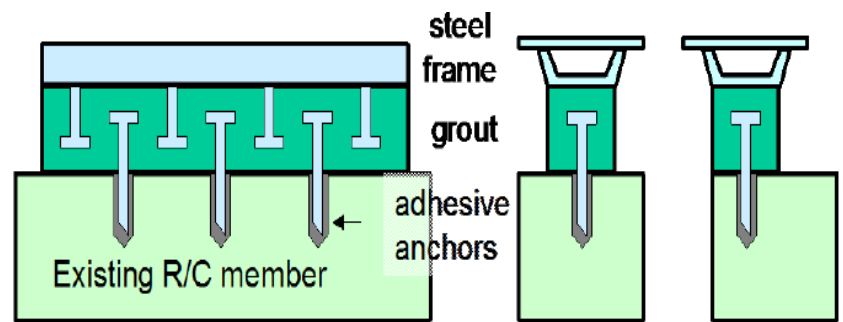


Figure 2 - Indirect connection

Figure; 2.11 indirect connections

### 2.10.2. Direct connection: fixing steel to concrete

Another possibility to fix braced steel frames to existing RC frames is the direct connection of the steel parts to the concrete. Adhesive or mechanical anchors usually perform the load transfer from the steel parts to the concrete frame. <sup>[21]</sup>



Figure; 2.12 diagonal brace to RC connection (direct connection)

## **2.11. Seismic design**

The seismic design of any lateral force resisting system begins with ductility. The fundamental assumption behind modern seismic resistant design is that rather than design a structure to resist a large earthquake force elastically, specific elements of the lateral force resisting system are designed to yield in a stable manner in order to dissipate the seismic energy that is imparted to the building.

Concrete is by far most useful material for building construction in the world and in last decade's reinforced concrete structures has played an important role in construction industry. Providing strength, stability and ductility are major purposes of seismic design. It is necessary to design a structure to perform well under seismic loads. <sup>[22]</sup>

## CHAPTER THREE

### 3. STRUCTURAL MODELLING AND ANALYSIS METHOD

#### 3.1. Modeling description

A rectangular building considered for analysis has symmetric in plan and elevation. Plan dimension of building to be modeled is 25mx15m. It consists of five bays of 5m with three bays of 5m in longer and shorter side respectively.

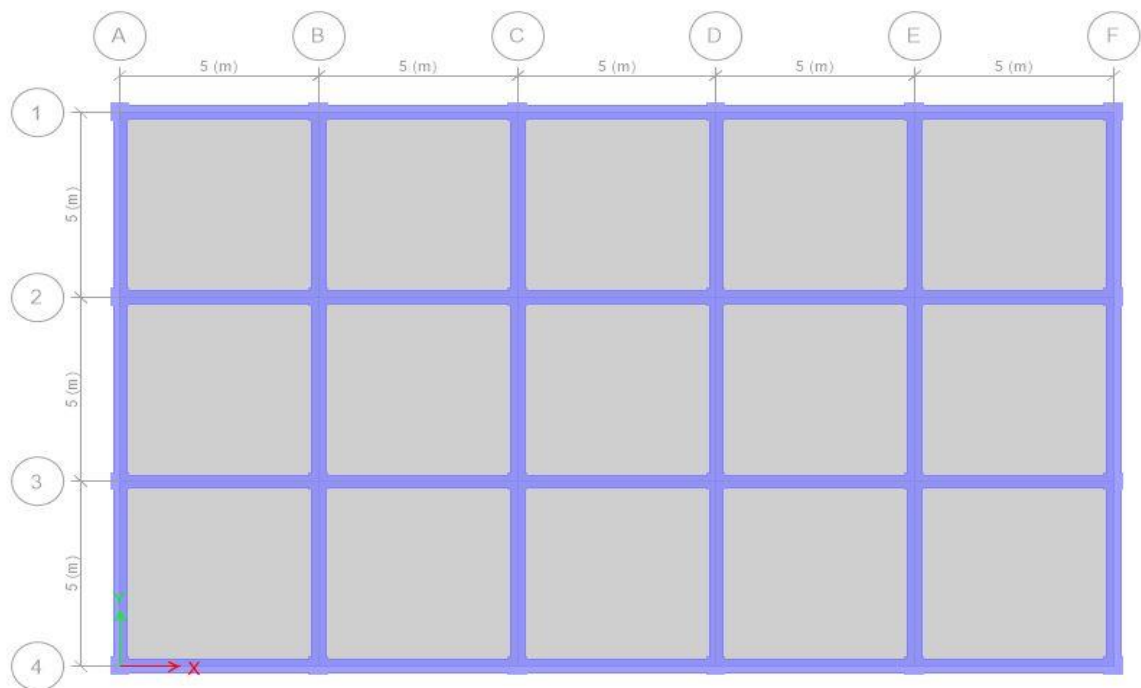
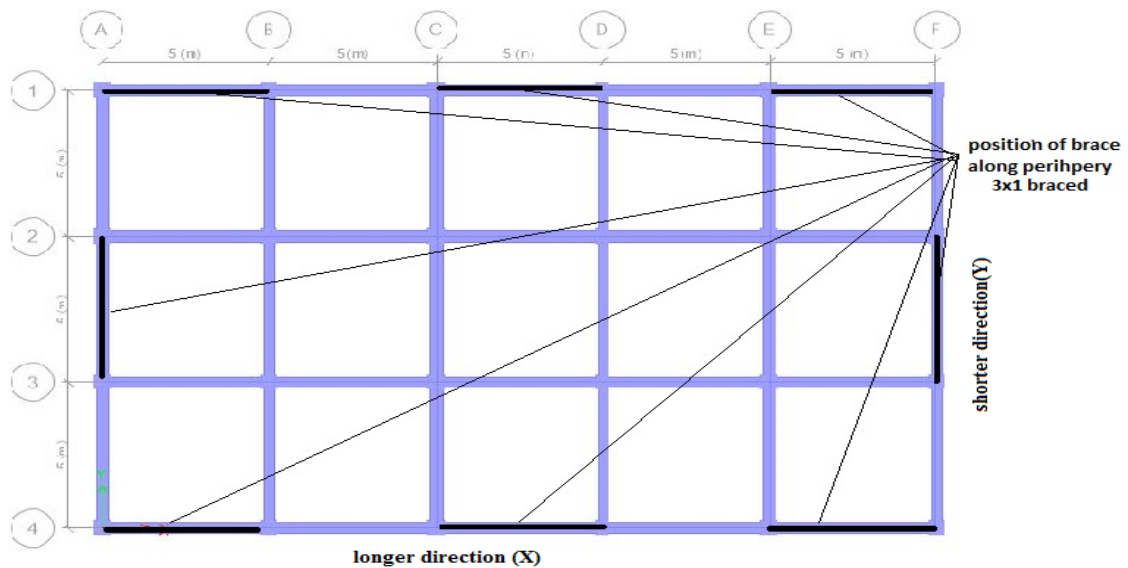
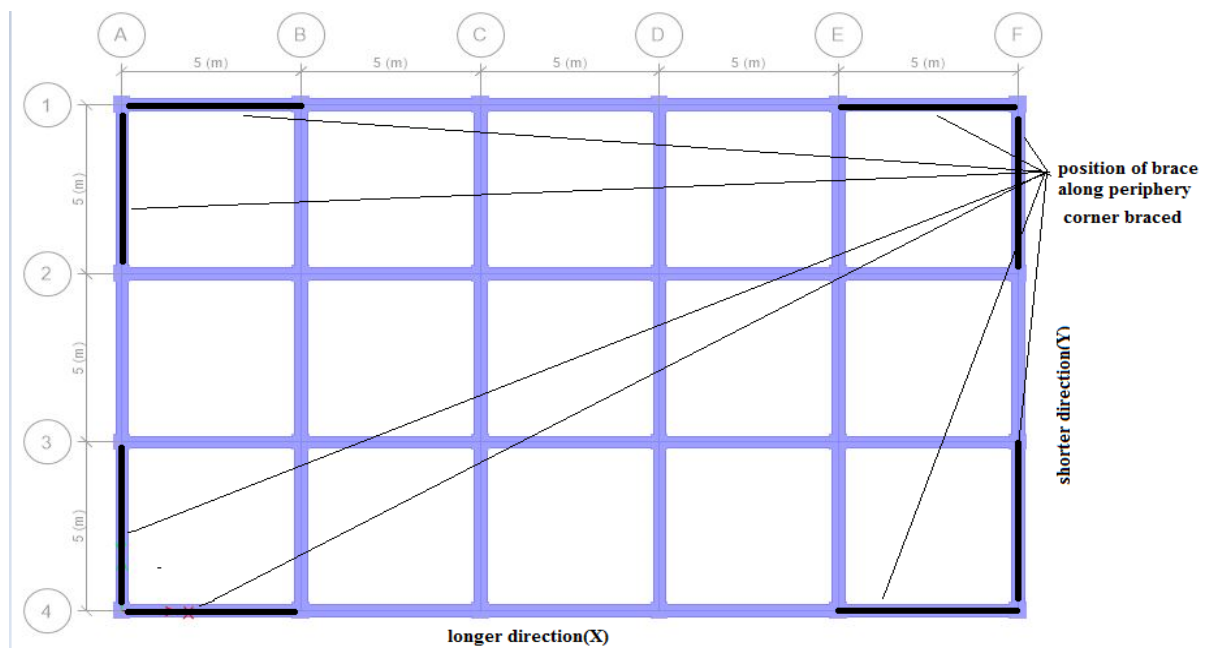


Figure 3.1 Floor plan



Figure; 3.2 plan representing the position of bracing where 3 of 5 & 1 of 3 bays braced (3x1 braced)



Figure; 3.3 plan representing the position of bracing (corner braced)

Different steel braced reinforced concrete frames are developed by placing the various types of bracing in the same location and the same configuration (arrangement) for all story types such as, 4story, 6story, 9story, and 13story buildings. Two cases are considered in this thesis, the

first case is **3x1 braced** (three bays braced in longer direction and one bay braced in shorter direction) as seen in figure 3.6 below and the second case is **corner braced** as seen in figure 3.7 below each case has 20 models total 40 frames are modeled and has analyzed with the same floor plan for all types of stories.

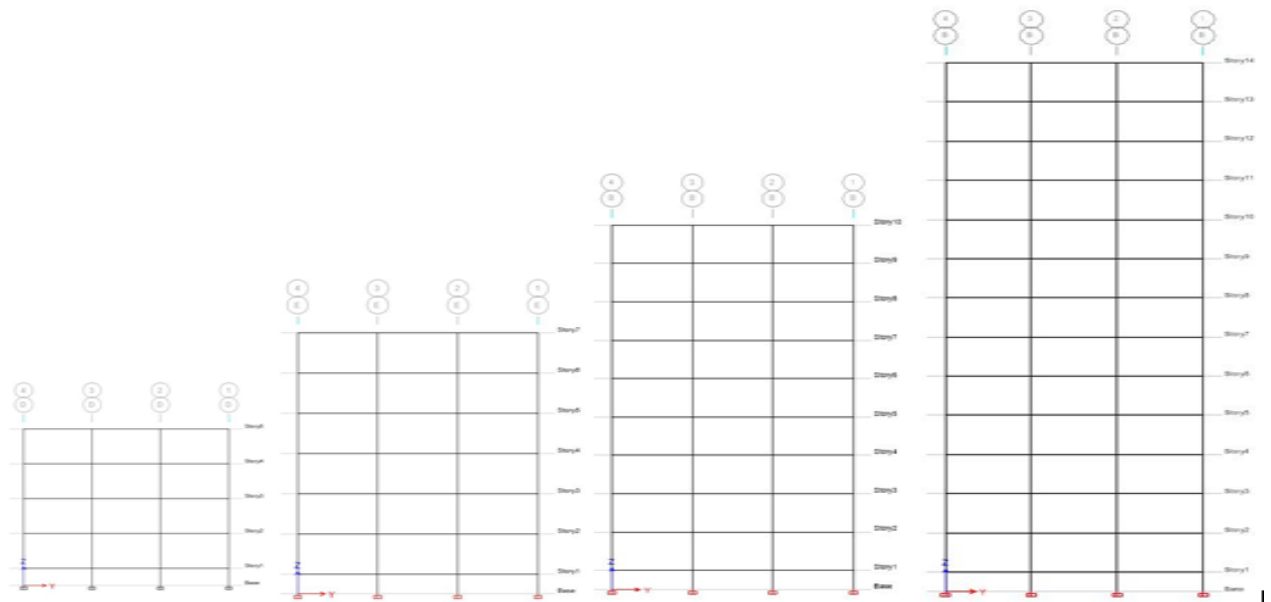
I decided to take those stories listed above according to the literatures which say steel braced reinforced concrete building is economical and efficient for building of any height. “The efficiency of bracing in being able to produce a laterally very stiff structure for a minimum of additional material makes it an economical structural form for any height of building, up to the very tallest. An additional advantage of fully triangulated bracing is that the beams usually participate only minimally in the lateral bracing action.”<sup>[24]</sup>

There are many different types of structural systems of tall buildings that have been developed and yet engineers are trying to come up with new efficient systems. According to Smith and Coull (1991), a rough estimation of how economically feasible each structural system is for different heights is listed as following

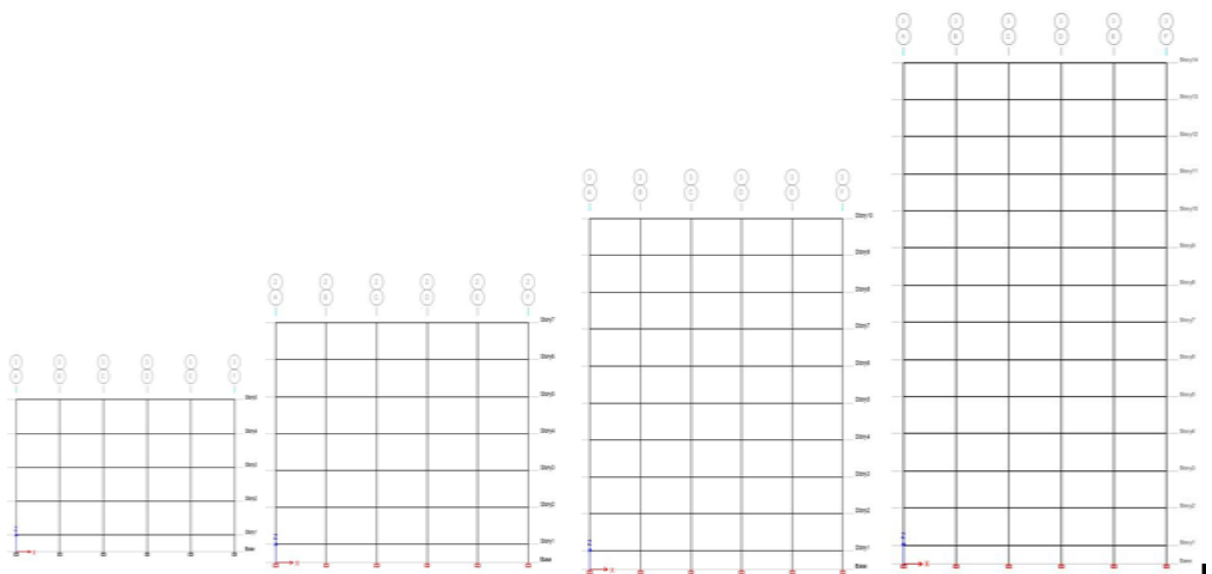
- 1) Non-braced frames with moment resisting connections: 1-25 stories
- 2) Truss-braced structures: 1- the very tallest
- 3) Shear wall-braced structures: 1-35 stories
- 4) Core structures with outriggers: 40- the very tallest
- 5) Tube structures: 40- the very tallest
- 6) Interacting structural systems: 1- the very tallest

The floor diaphragms of the building has assumed as being rigid in their planes, the masses and the moments of inertia of each floor may be lumped at the center of gravity.[] The four, six, nine and thirteen storied buildings are modeled with diagonal, X, V and inverted V (chevron) type of bracings for peripheral columns only. Section of bracing and dimension of column and beam are the same though out the building height and the orientation and size of column is kept same throughout the height of the structure. The section of bracing steel selected for this comparison purpose has tube section type. The location of all bracing type with in the bays of

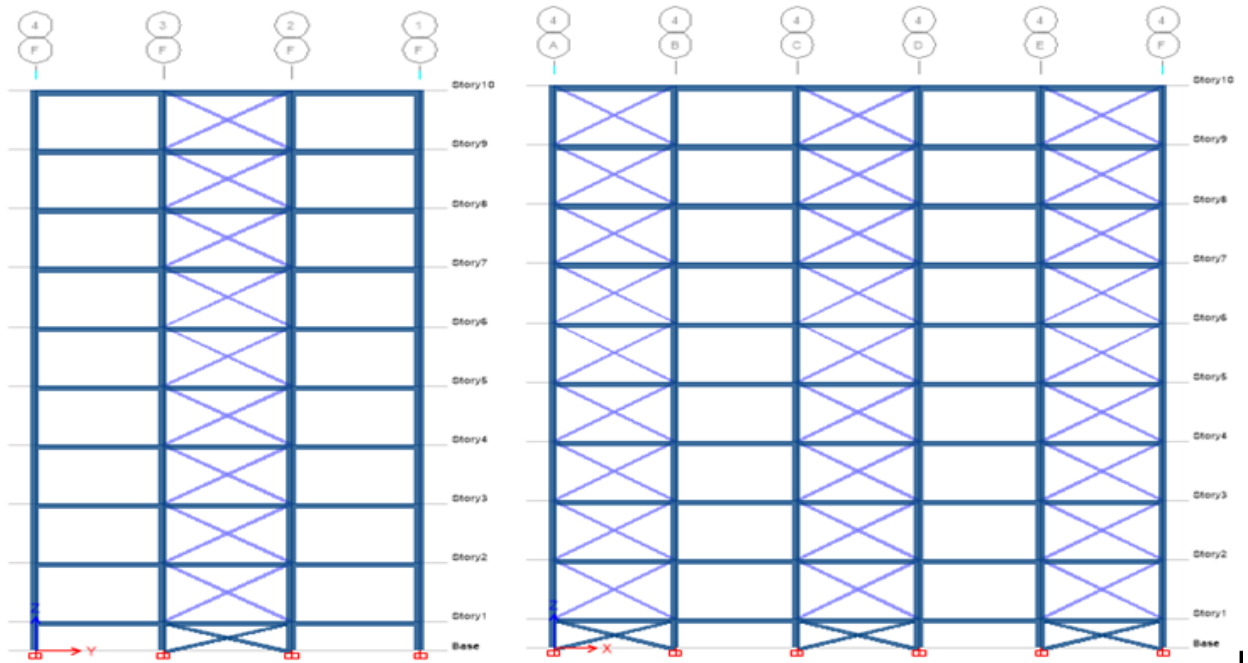
building is the same. The numbers of braced bays are 3 in the longer side and 1 in the shorter side for 3x1 braced case and 2 bays are braced in both direction for case of corner braced as seen in sample figure 3.6, figure3.7 and figure 3.8 below. Building has kept symmetric in both mutually perpendicular directions in plan to avoid torsional effects.



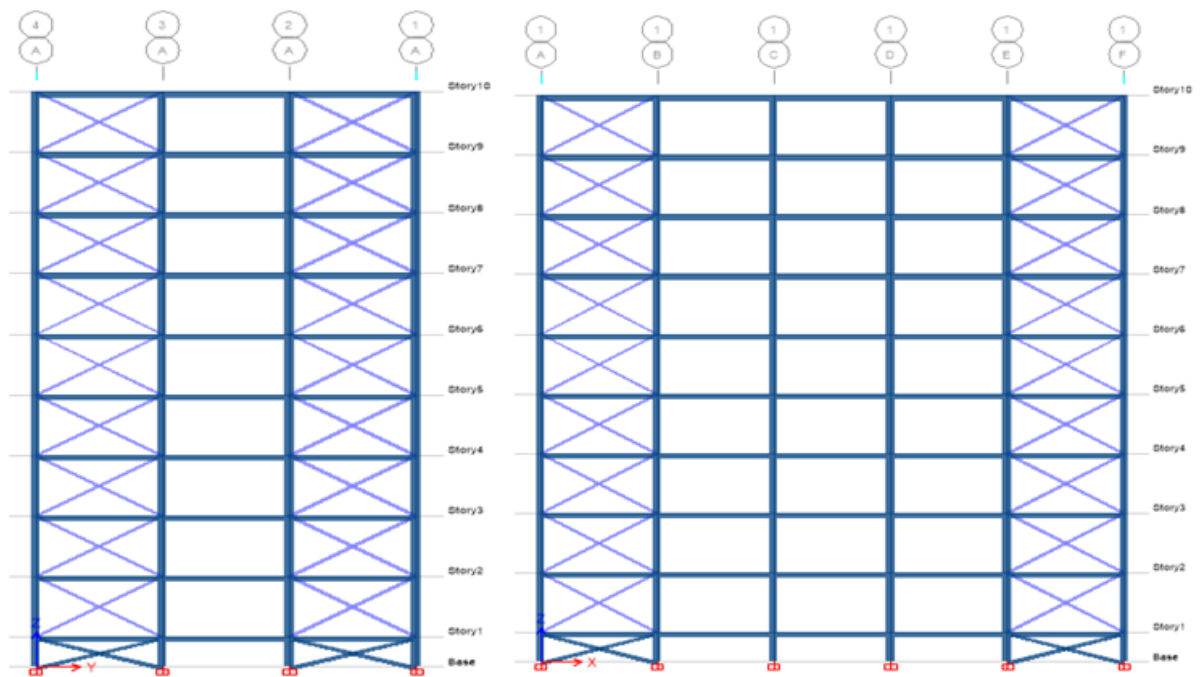
Figure; 3.4 elevation view of 4, 6, 9 and 13 story building in shorter direction(Y-dir.)



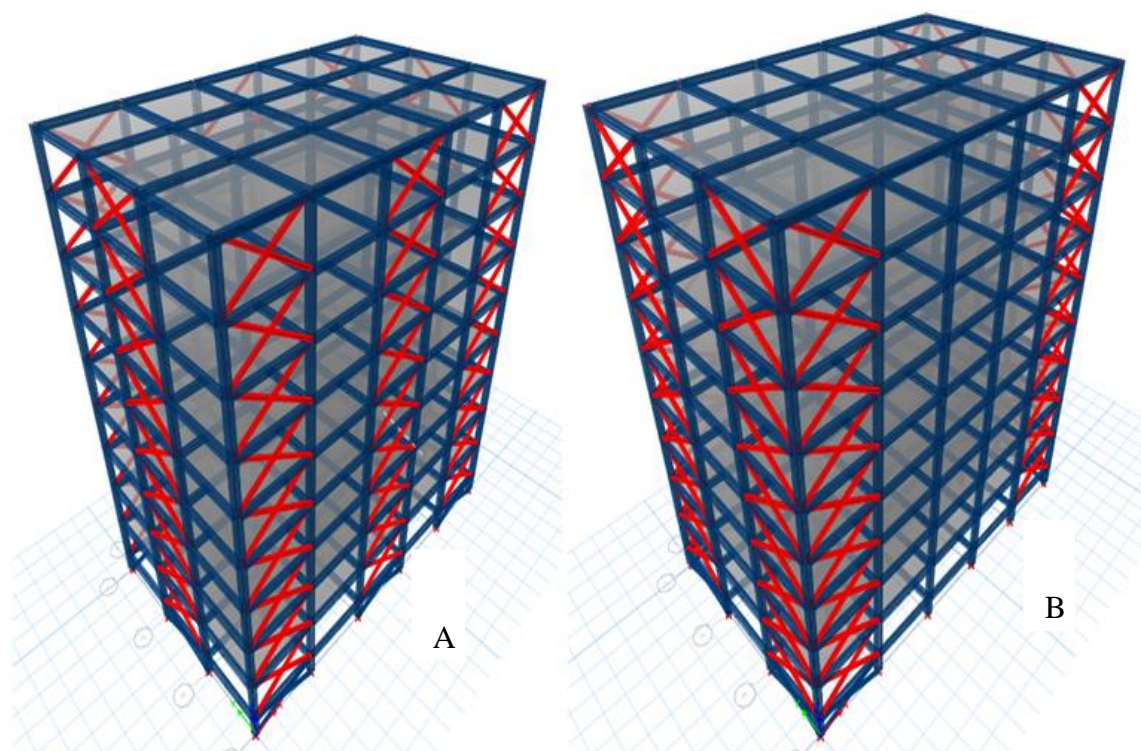
Figure; 3.5 elevation view of 4, 6, 9 and 13 story building in longer direction(X-dir.)



Figure; 3.6 bracing pattern for shorter(y) and longer(x) direction of 3x1 braced



Figure; 3.7 bracing pattern for shorter(y) and longer(x) direction of corner braced



Figure; 3.8 3D model of X-braced sample frame, A; 3x1 braced, B: corner braced

### 3.2. Modeling data of building

Table: 3.1 material properties

Material properties	Grade of concrete	C-25
	Density of reinforced concrete	25kN/m <sup>3</sup>
	Modulus of elasticity of concrete	31000N/mm <sup>2</sup>
	Grade of steel	S-400
		S-355 for bracing
	Density of reinforcing steel	7850 kg/m <sup>3</sup>
Modulus of elasticity of steel	200GPa	

Table; 3.2 member properties

Member properties	column	350mmx350mm	for story 4,6&9
		500mmx500mm	for story 13
	beam	300mmx300mm	for story 4,6&9
		350mmx350mm	for story 13
	slab thickness	150mm	for all storys
bracing steel	tube section 150x150x15mm	for all storys	

Some assumptions related to modeling according to EBCS EN

1. Material: Concrete is assumed to behave linearly elastic.

The self-weight of concrete assumed to be 25kN/m<sup>3</sup> and the specified compressive strength of concrete  $f'_c$  is assumed equal to 25Mpa, which is not lower, than C16/20 for primary seismic elements according to EBCS EN 1998-1-1:2014 and as used in practical applications of most buildings.

2. Participating components: Only the primary structural components are assumed to participate in the overall behavior. The effects of secondary structural components and non-structural components are assumed to be negligible; these include staircases, partitions, cladding, and openings.
3. Floor slabs: are assumed to be rigid in plane, with thickness equal to 150 mm in all models. This assumption causes the vertical elements at any floor level undergo the same components of translational displacement and rotation in the horizontal plane.
4. Cracking: The effect of cracking in reinforced concrete members due to flexural tensile stresses is represented by reducing moment of inertia, according to EBCS EN 1998-1-1:2014

RC structures start cracking at a relatively low level of loading. Hence the elastic linear analysis based on the uncracked section serves little in predicting actual behavior. Linear analysis based on the cracked section is more meaningful<sup>[27]</sup>.

Unless a more accurate analysis of the cracked elements is performed, the elastic flexural and shear stiffness properties of concrete and masonry elements may be taken to be equal to one-

half of the corresponding stiffness of the uncracked elements. EBCS EN 1998-1-1:2014 (4.3.1(7))

Property modification factors are used to reduce moment and torsion stiffness due to crack section. Torsional stiffness of the cracked section should be set equal to 10% of the torsional stiffness of the un-cracked section.

Table; 3.3 Stiffness of cracked section of RC members

Column (Line element)	Beam (Line element)	Slab (Shell element)
$I_{22}=I_{33}=0.5$	$I_{22}=I_{33}=0.5$	$m_{11}=m_{12}=m_{22}=0.5$
$I_t=0.1$	$I_t=0.1$	$I_t=0.1$

### 3.3. Method of structural modeling and Analysis

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 behaviour of the structure. The reference method for determining the seismic effects shall be the modal response spectrum analysis, using a linear-elastic model of the structure and the design spectrum given below in 3.3.1.

Depending on the structural characteristics of the building one of the following two types of linear-elastic analysis may be used

- 1) The “lateral force method of analysis(elastic response spectrum)” for buildings meeting the conditions given in EBCS EN 1998-1-1:2014 section 4.3.3.2;
- 2) The “modal response spectrum analysis”, which is applicable to all types of buildings

#### 3.3.1. Lateral force method of analysis

This type of analysis may be applied to buildings whose response is not significantly affected by contributions from modes of vibration higher than the fundamental mode in each principal direction.

This approach defines a series of force acting on a building to represent the effect of earthquake ground motion, typically defined by seismic design response spectrum in lateral force method seismic load is applied is the following steps;

Step 1: the design lateral force shall be first computed for the building as a whole

Step 2: this design lateral force shall then be distributed to the various floor levels

Step 3: the overall design seismic force thus obtained at each floor level shall then be distributed to individual lateral load resisting elements depending on the floor diaphragm action.

And the method should the following two conditions;

- a) They have fundamental periods of vibration  $T_1$  in the two main directions which are smaller than the following values

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

Where  $T_c$  from table

- b) They meet the criteria for regularity in elevation given in 4.2.3.3.EBCS EN 1998-1 2014;  
In this study lateral force method (Equivalent Static Force Method) is used, the analysis of the structure is made for seismic loads using Equivalent Static Force Method because of symmetry of the structure, both in geometry and in mass. The analysis of the study is based on the provision of EBCS EN 1998-1:2014

Finite element analysis of Structural Systems ETABS software is used for the analysis of all structural systems by lateral force method (Equivalent Static Lateral Force Method) for Zone IV. Based on the method of analysis considered, lateral load calculations are made by the software itself and then applied to the structure to analyse. Hence the results are tabulated for the study of behaviour of structural systems

The four, six, nine and thirteen storied buildings are analyzed for zone IV and with diagonal, X, V and inverted V (chevron) type of bracings for peripheral columns only.

### 3.3.2. Design spectrum for elastic analysis

The capacity of structural systems to resist seismic actions in the non-linear range generally permits their design for resistance to seismic forces smaller than those corresponding to a linear elastic response.

To avoid explicit inelastic structural analysis in design, the capacity of the structure to dissipate energy, through mainly ductile behavior of its elements and/or other mechanisms, is taken into account by performing an elastic analysis based on a response spectrum reduced with respect to the elastic one, henceforth called a "design spectrum". This reduction is accomplished by introducing the behavior factor  $q$ . 3.2.2.5(2)

Modern seismic code provisions for buildings rely on energy dissipation through inelastic deformations during the design earthquake due to the necessity to compromise life safety requirements with economic considerations. To avoid demanding nonlinear analysis in the framework of everyday design purposes, an equivalent lateral load or modal response spectrum analysis is permitted, using spectral accelerations that result from a response spectrum, appropriately reduced by a, so called, behavior factor ( $q$  in Europe) or force reduction factor ( $R$  in the U.S.).

For the horizontal components of the seismic action the design spectrum,  $S_d(T)$ , shall be defined by the following expressions:

$$0 \leq T \leq T_B: \quad S_d(T) = a_g \cdot S \cdot \left[ \frac{2}{3} + \frac{T}{T_B} \cdot \left( \frac{2.5}{q} - \frac{2}{3} \right) \right] \dots$$

$$T_B \leq T \leq T_C: \quad S_d(T) = a_g \cdot S \cdot \eta \cdot \frac{2.5}{q}$$

$$T_C \leq T \leq T_D: \quad S_d(T) \quad \left\{ = a_g \cdot S \cdot \frac{2.5}{q} \cdot \left[ \frac{T_C}{T} \right] \geq \beta \cdot a_g \dots \right.$$

$$T_D \leq T: \quad S_d(T) \quad \left\{ = a_g \cdot S \cdot \frac{2.5}{q} \cdot \left[ \frac{T_C \cdot T_D}{T^2} \right] \geq \beta \cdot a_g \dots \right.$$

Where

$S_d(T)$  Design spectrum

$a_g$  Design ground acceleration

$S$	Soil factor
$T_C$	Upper limit of the period of the constant spectral acceleration branch
$T_D$	Value defining the beginning of the constant displacement response range of the spectrum
$q$	Behavior factor
$\beta$	The lower bound factor for the horizontal design spectrum The recommended value for $\beta$ is 0.2. (Found in National Annex.)

### 3.3.3. Behavior factor $q$

The behavior factor  $q$  is an approximation of the ratio of the seismic forces that the structure would experience if its response was completely elastic with 5% viscous damping, to the seismic forces that may be used in the design, with a conventional elastic analysis model, still ensuring a satisfactory response of the structure

- A behavior factor reflects the capacity of a structure to deform plastically

The values of the behavior factor  $q$ , which also account for the influence of the viscous damping being different from 5%, are given for various materials and structural systems according to the relevant ductility classes in the various Parts of EBCS EN 1998-1,2014 section 3.2.2.5(3)

- Behavior factors for horizontal seismic actions

The upper limit value of the behaviour factor  $q$ , to account for energy dissipation capacity, shall be derived for each design direction as follows according to the code:

$$q = q_0 K_w \geq 1.5$$

Where;

$q_0$  is the basic value of the behavior factor, dependent on the type of the structural system and on its regularity in elevation

$K_w$  Is the factor reflecting the prevailing failure mode in structural systems with walls.

For purposes of defining the value of behaviour coefficient is necessary classify the structural system and define their regularity in plan and height. If the structure is not defined by regular in height, the reference values of behaviour coefficient must be reduced in 20%. The reinforced concrete buildings are classified into several types of structural systems, as represented below.

- 1) Frame system; Structural system which resists by frames whose shear resistance at the building base exceeds 65% of the total shear resistance
- 2) Dual System (frame-equivalent); Dual system in which the shear resistance of the frame system at the building base is higher than 50% of the total shear resistance.
- 3) Dual System (wall equivalent); *Dual system in which the shear resistance of the walls at the building base is higher than 50% of the total seismic resistance.*
- 4) Wall system; Structural system which resists by vertical structural walls, either coupled or uncoupled, whose shear resistance at the building base exceeds 65% of the total shear resistance
- 5) Core system; Dual or wall system not having a minimum torsional rigidity
- 6) Inverted Pendulum System; *System in which 50% or more of the mass is in the upper third of the height of the structure.*

For buildings that are regular in elevation in accordance to the code, the basic values of  $q_0$  for the various structural types are given in Table 5.1 in EBCS EN 1998-1, 2014.

Table; 3.4 Basic value of the behavior factor,  $q_0$  for system regular in elevation

<i>Structural System</i>	<i>Low Ductility</i>	<i>Medium Ductility</i>	<i>High Ductility</i>
Frame system, dual system, coupled wall system	1,5	$3,0 \cdot \frac{\alpha_u}{\alpha_1}$	$4,5 \cdot \frac{\alpha_u}{\alpha_1}$
Wall system	1,5	3,0	4,0
Core system	1,5	2,0	3,0
Inverted pendulum system	1,5	1,5	2,0

Source: table 5.1 of EBCS EN 1998-1, 2014

$\alpha_1$  and  $\alpha_u$  are defined as follows:

$\alpha_1$  is the value by which the horizontal seismic design action is multiplied in order to first reach the flexural resistance in any member in the structure, while all other design actions remain constant;

$\alpha_u$  is the value by which the horizontal seismic design action is multiplied, in order to form plastic hinges in a number of sections sufficient for the development of overall structural instability, while all other design actions remain constant. The factor  $u$  may be obtained from a nonlinear static (pushover) global analysis.

When the multiplication factor  $\frac{\alpha u}{\alpha_1}$  has not been evaluated through an explicit calculation, for buildings which are regular in plan the following approximate values of  $\frac{\alpha u}{\alpha_1}$  may be used

a) Frames or frame-equivalent dual systems.

One-story buildings:  $\frac{\alpha u}{\alpha_1}=1.1$ ;

Multistory, one-bay frames:  $\frac{\alpha u}{\alpha_1}=1.2$ ;

Multistory, multi-bay frames or frame-equivalent dual structures:  $\frac{\alpha u}{\alpha_1}=1.3$ .

b) Wall-or wall-equivalent dual systems.

Wall systems with only two uncoupled walls per horizontal direction:  $\frac{\alpha u}{\alpha_1}=1.0$ ;

Other uncoupled wall systems:  $\frac{\alpha u}{\alpha_1}=1.1$ ;

Wall-equivalent dual or coupled wall systems:  $\frac{\alpha u}{\alpha_1}=1.2$ .

The factor  $K_w$  reflecting the prevailing failure mode in structural systems with walls shall be taken as follows:

$$k_w = \left\{ \begin{array}{l} 1.00, \text{ for frame and frame-equivalent dual systems} \\ (1 + \alpha_0) / 3 \leq 1, \text{ but not less than } 0.5, \text{ for wall-equivalent and torsionally} \\ \text{flexible systems} \end{array} \right\}$$

In this study the ductility class assumed is DCM, therefore the behavior factor is calculated according to provided parameters for this class.

Assume the system is frame-equivalent dual systems. From the above table frame-equivalent dual systems in which the shear resistance of the system at the building base is higher than 50% of the total seismic resistance.

$$q = q_o K_w \geq 1.5$$

Where  $q_o = 3 \frac{\alpha_u}{\alpha_1}$ .....From table 3.2 for DCM and the value of  $\frac{\alpha_u}{\alpha_1}$  for the selected system is  $\frac{\alpha_u}{\alpha_1} = 1.3$

$$\therefore q_o = 3 * 1.3 = \underline{3.9}$$

$K_w = 1$  for the selected system

$$\diamond q = \underline{3.9}$$

#### 3.3.4. Base shear force

The seismic base shear force  $F_b$ , for each horizontal direction in which the building is analysed, shall be determined using the following expression:

$$F_b = S_d(T_1) * m * \lambda$$

Where

$S_d(T_1)$  Is the ordinate of the design spectrum at period  $T_1$

$T_1$  is the fundamental period of vibration of the building for lateral motion in the direction considered;

$m$  is the total mass of the building, above the foundation or above the top of a rigid basement, computed in accordance with 3.2.4(2);

The inertial effects of the design seismic action shall be evaluated by taking into account the presence of the masses associated with all gravity loads appearing in the following combination of actions. EBCS EN 1998 3.2.4(2)

$$\sum G_{k,j} \psi_{E,i} \cdot Q_{k,i} \text{ (Eq.3.17)}$$

$\psi_{E,i}$  Is the combination coefficient for variable action i

$$\psi_{E,i} = \phi \cdot \psi_{2,i} \text{ (Eq. 4.2) } \dots \dots \dots \text{ 4.2.4(2)}$$

$\psi_{2,i}$  = Occupancy type coefficient,  $\phi$  = Load type coefficient

Table; 3.5 value of  $\phi$  for calculating  $\psi_{Ei}$  and values of factors  $\psi_2$  for buildings

Story	$\phi$	occupancy type	$\psi_2$
roof	1	Residential, office	0.3
stories with correlated occupancies	0.8	Public, commercial(shops), parking	0.6
interdependently occupied story's	0.5	Roof with snow	0.2
Archives	1	Archives, libraries, staircases	0.8

Source: table 4.2 on EBCS EN 1998-1, 2014 and table A1.1 of EBCS EN1991-2013

$$\psi_{E,i} = 0.8 \cdot 0.3 = 0.25 \dots \dots \dots \text{from table 4.2 of EBCS EN 1998-1, 2014 and table A1.1 of EBCS EN 1991-2013}$$

$$: - m = \sum DL + 0.25 \sum LL \text{ (Seismic load/mass)}$$

$\lambda$  is the correction factor, the value of which is equal to:  $\lambda = 0.85$  if  $T_1 < 2 T_C$  and the building has more than two storeys, or  $\lambda = 1.0$  otherwise

For buildings with heights of up to 40 m the value of  $T_1$  (in sec.) may be approximated by the following expression:

$$T_1 = C_t \cdot H^{3/4}$$

Where

$C_t = 0.085$  for moment resistant space steel frames,

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

$0.050$  For all other structures;

H is the height of the building, in m, from the foundation or from the top of a rigid basement.

### 3.3.5. Distribution of the horizontal seismic forces

When the fundamental mode shape is approximated by horizontal displacements increasing linearly along the height, the horizontal forces  $F_i$  should be taken as being given by:

The horizontal forces  $F_i$  determined in accordance with this clause shall be distributed to the lateral load resisting system assuming the floors are rigid in their plane.

$$F_i = F_b \frac{Z_i * m_i}{\sum Z_j * m_j}$$

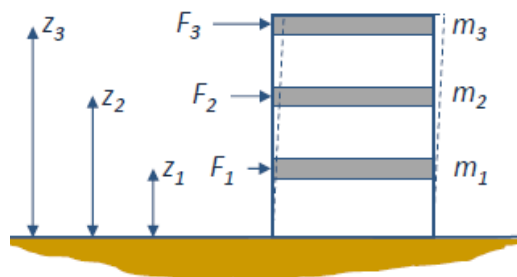
Where

$Z_i, Z_j$  are the heights of the masses  $m_i, m_j$  above the level of application of the seismic action (foundation or top of a rigid basement).

$m_i, m_j$  stories masses

$F_i$  horizontal seismic force of stories

$F_b$  base shear



Figure; 3.9 distribution of story force

### 3.3.6. Type of Structure

Type of structure considered for the analysis is ordinary moment resisting reinforced concrete frame which is regular in plan and elevation

### 3.3.7. Importance category (class) of Structure and importance factor ( $\gamma_1$ )

Table 4.3 of EBCS EN 1998-1, 2014 present four importance classes to classification the buildings based on the following parameters: loss of human lives; the importance of building to public safety and civil protection immediately after the earthquake; social and economic consequences associated with its collapse.

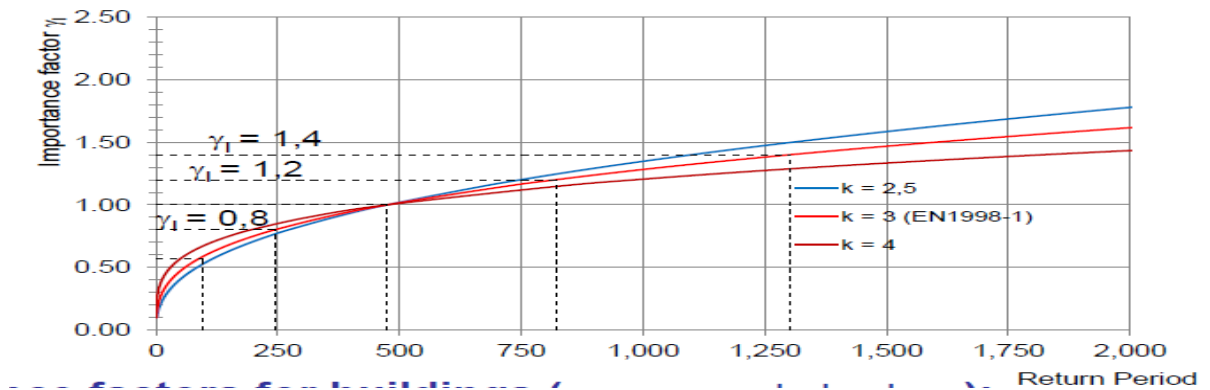
Table; 3.6 importance classes for building

Importance class	Building	Importance
I	Building of minor importance for public safety, e.g. agricultural	0.8
II	Ordinary buildings, not belonging in the other categories.	1.0
III	Buildings whose seismic resistance is of importance in view of the consequences associated with a collapse, e.g. schools, assembly halls, cultural institutions etc.	1.2
IV	Buildings whose integrity during earthquakes is of vital importance for civil	1.4

Source: table 4.3 of EBCS EN 1998-1, 2014

The structure in this thesis used as office building, the importance class of structure is represented class II category from the above Table 4.3 of EBCS EN 1998-1,2014....4.2.5(4)

The corresponding Importance factor ( $\gamma_1$ ) =1.0 of EBCS EN 1998-1, 2014



Figure; 3.10 Graph of importance factor

### 3.3.8. Soil Type (ground type)

The average response acceleration coefficient ( $S_a/g$ ) depends on the type of soil where the structure is located and the fundamental natural time period ( $T_a$ ) of buildings. Hence knowing the soil type becomes important for the calculation of lateral load.

The values of the period  $T_B$ ,  $T_C$  and  $T_D$  and of the soil factor  $S$  describing the shape of the elastic response spectrum depend upon the ground type. The recommended choice is the use of two types of spectra: Type 1 and Type 2

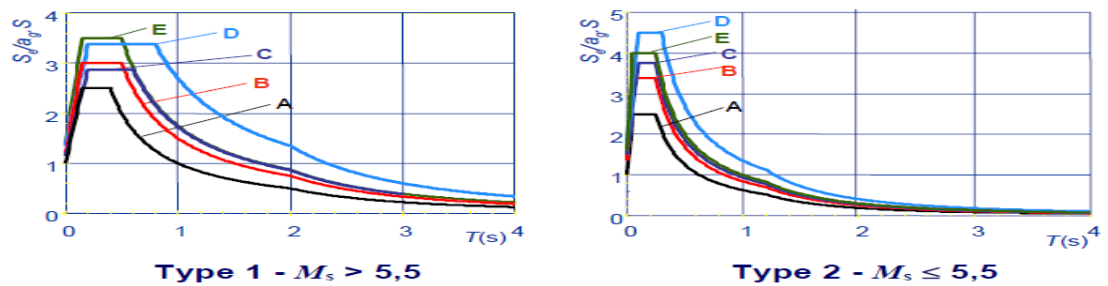
If the earthquakes that contribute most to the seismic hazard defined for the site for the purpose of probabilistic hazard assessment have a surface-wave magnitude,  $M_s$ , not greater than 5.5 ( $M_s < 5.5$ ), it is recommended that the Type 2 spectrum is adopted. Otherwise type 1 spectrum has used

Depending on the characteristics of the most significant earthquake contributing to the local hazard:

Type 1 - High and moderate seismicity regions ( $M_s > 5.5$ )

Type 2 - Low seismicity regions ( $M_s \leq 5.5$ ); near field earthquakes

### Recommended elastic response spectra



Figure; 3.11 graphs of types of elastic response spectra

In the present study the suitable type of spectra for seismic zone IV is type 1 spectral to consider the maximum effect of earth quake on the building, therefore the corresponding parameters related to type 1, such as; periods and site coefficient of soil are listed in table 3.7 below from EBCS EN 1998-1, 2014

Table; 3.7 Values of the parameters describing the recommended Type 1 elastic response spectra

Ground type	$S$	$T_B$ (s)	$T_C$ (s)	$T_D$ (s)
A	1,0	0,15	0,4	2,0
B	1,2	0,15	0,5	2,0
C	1,15	0,20	0,6	2,0
D	1,35	0,20	0,8	2,0
E	1,4	0,15	0,5	2,0

Source: EBCS EN 1998-1, 2014 table 3.2.

#### 3.3.9. Seismic Zones

In the present study, the response of the structural systems is studied for seismic zones IV of Ethiopia as per EBCS EN 1998 new-draft code. According to Ethiopian building code EBCS EN 1998-1, 2014 the seismic hazard map is divided into 5 zone, where the ratio of the design bedrock acceleration to the acceleration of gravity  $g = \alpha_0$  for the respective zone is described in table below.

Table; 3.8 Bedrock Acceleration Ratio  $\alpha_0$

Zone	5	4	3	2	1	0
$\alpha_0 = a_g/g$	0.20	0.15	0.10	0.07	0.04	0

Source: Table D1 of EBCS EN 1998-1, 2014

### 3.4. Loading

The building weight and its content is considered in the dead load (gravity load) and calculated based on material densities by the program, While live load is taken as 5kN/m<sup>2</sup>. All the structural systems are subjected to three types of primary load cases as per the provisions of EBCS EN 1998-1; 2014 and other EBCS EN Code. They are:

1. Dead Load; only (self-weight) load from the frame of the buildings has considered for this thesis
2. Live Load (EBCS 1-1995 old) for office building from the code category of building area D (D1), the imposed floor area load is 5kN/m<sup>2</sup>.
3. Seismic Load (EBCS EN 1998-1; 2014) is calculated from the mass of the building by considering imposed loads (live load) as mentioned in section 3.3.4 of this chapter

## CHAPTER FOUR

### 4. ANALYSIS RESULT COMPARISON AND DISCUSSION

The building frames have been analysed using the simplified valued response spectrum analysis method (lateral force method).the analysis has done by nonlinear finite element software ETABS 2015 to assess the seismic behaviour of selected story building. From the analytical analysis of the models the results of various parameters like lateral story displacement, story drift, story shear and story overturning moment has been compared in the next section of this paper for 4story(G+3), 6story(G+5), 9story(G+8), 13story(G+12) respectively. The comparative study of all parameters has done at story level for seismic loading only.

The lateral displacements, base shear, base overturning moment and story drifts of RC bare frame(without bracing) building for the cases of seismic load has analyzed in both X and Y directions are presented in the preceding section of the chapter for both cases 3x1 braced and corner braced. The results of braced RC buildings are compared each other with various types of steel bracings and compared with bare frame. It is observed that the maximum lateral displacements are reduced due to the presence of bracings.

The graphs of lateral displacement, story drift versus no of story are plotted in X direction and Y direction and also graph of base shear and base overturning moment versus types of bracing (types of structure) are plotted in both directions and compared for 4story, 6story, 9story and 13story respectively of all types of buildings without and with different steel bracing systems. From the tables and figures, it is observed that the lateral displacement and story drift are reduced to largest extent for all bracing systems, while the displacement is maximum for the system without bracing. Lateral displacement in cases of 3 bays out of 5 and 1 bay out of 3 bays are braced (3x1 braced) and corner braced are show some variation while base shear and base overturning moment is the same for both cases as presented below. In general base shear and overturning moment are increased for braced frames

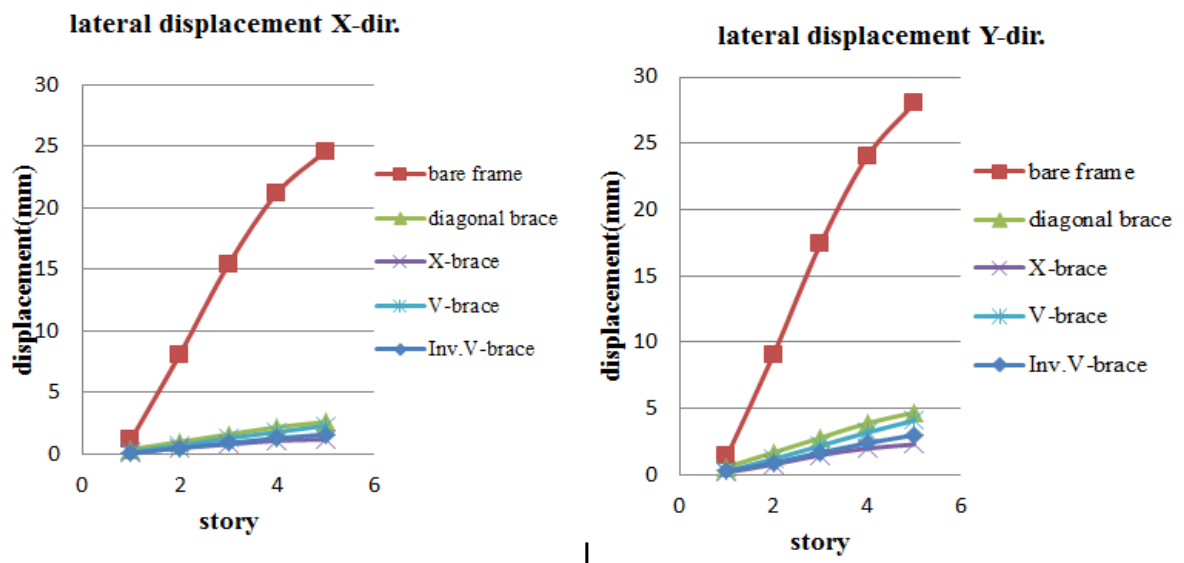
## 4.1. Story Four

### 4.1.1. Lateral displacement comparison of 4story

#### 4.1.1.1. Lateral displacement of 3x1 braced

Table; 4.1 lateral displacement of 4 story building where 3 of the 5 bays and 1 of the 3 bays are braced (3x1 braced)

story	elevation	bare frame		diagonal brace		X brace		V brace		Inv.V-brace	
		X	Y	X	Y	X	Y	X	Y	X	Y
	m	mm	mm	mm	mm	mm	mm	mm	mm	mm	mm
4	10.5	24.6	28	2.6	4.7	1.8	3.4	2.3	4.1	1.6	3
3	7.5	21.2	24	2.2	3.9	1.4	2.6	1.8	3.2	1.3	2.4
2	4.5	15.4	17.4	1.6	2.8	1	1.8	1.3	2.2	0.9	1.7
1	1.5	8.1	9.1	1	1.7	0.5	1	0.7	1.2	0.5	0.9
ground	0	1.2	1.4	0.4	0.6	0.2	0.3	0.2	0.3	0.1	0.3

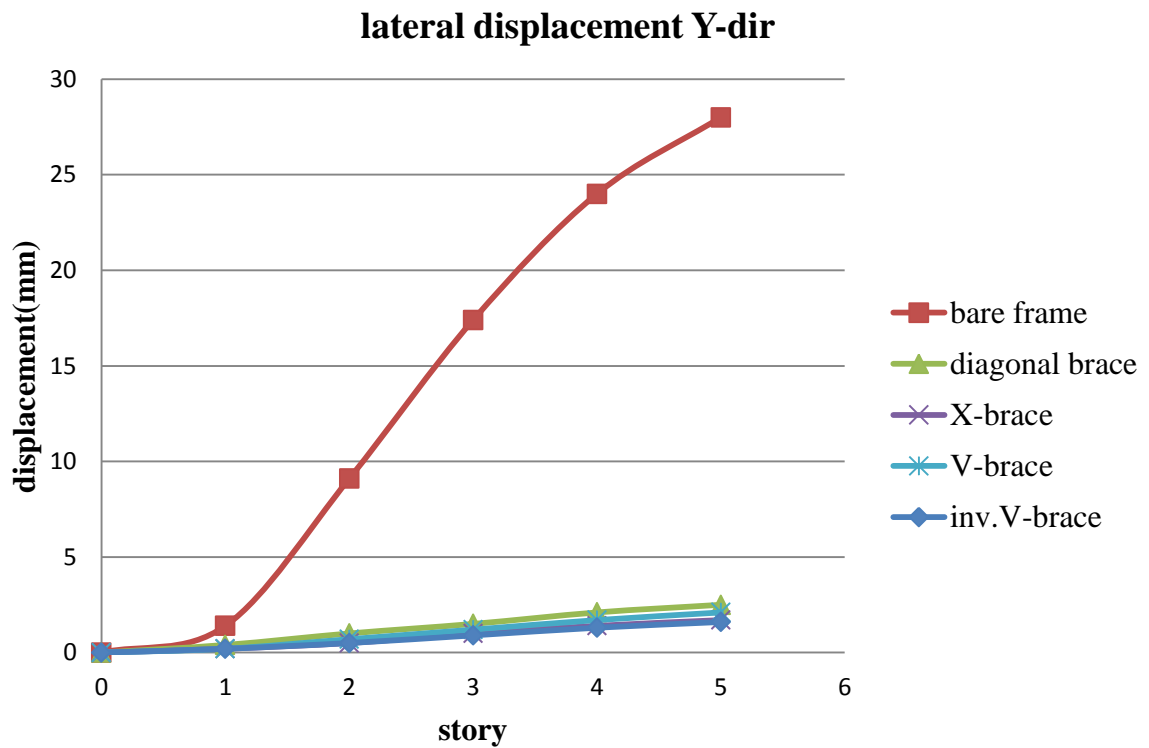


Figure; 4.1 max. Lateral displacement of story4 3x1 braced

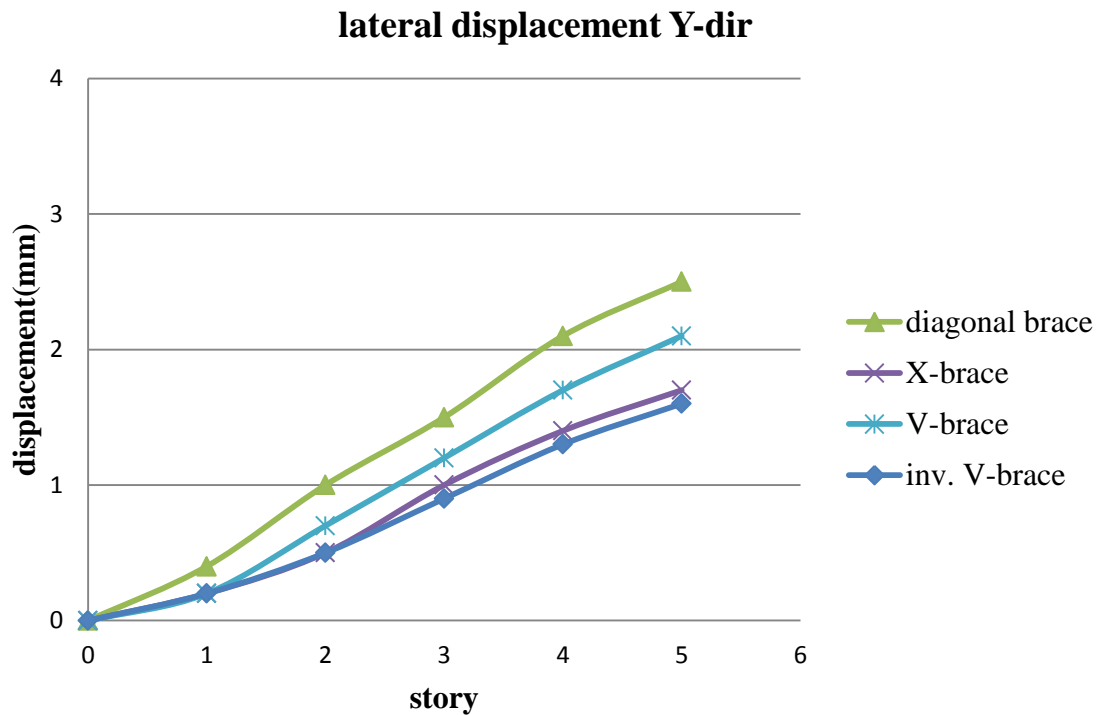
4.1.1.2. Lateral displacement of 4story corner braced

Table; 4.2 maximum lateral displacement of 4story corner braced

story	elevation	bare frame		diagonal brace		X brace		V brace		Inv.V-brace	
		X	Y	X	Y	X	Y	X	Y	X	Y
	m	mm	mm	mm	mm	mm	mm	mm	mm	mm	mm
4	13.5	24.6	28	2.4	2.5	1.7	1.7	2.1	2.1	1.6	1.6
3	10.5	21.2	24	2	2.1	1.4	1.4	1.7	1.7	1.3	1.3
2	7.5	15.4	17.4	1.5	1.5	0.9	1	1.2	1.2	0.9	0.9
1	4.5	8.1	9.1	0.9	1	0.5	0.5	0.7	0.7	0.5	0.5
ground	1.5	1.2	1.4	0.4	0.4	0.2	0.2	0.2	0.2	0.1	0.2



Figure; 4.2 max. Lateral displacement of story4 corner braced



Figure; 4.3 max. Lateral displacement of only braced frame of 4story corner braced

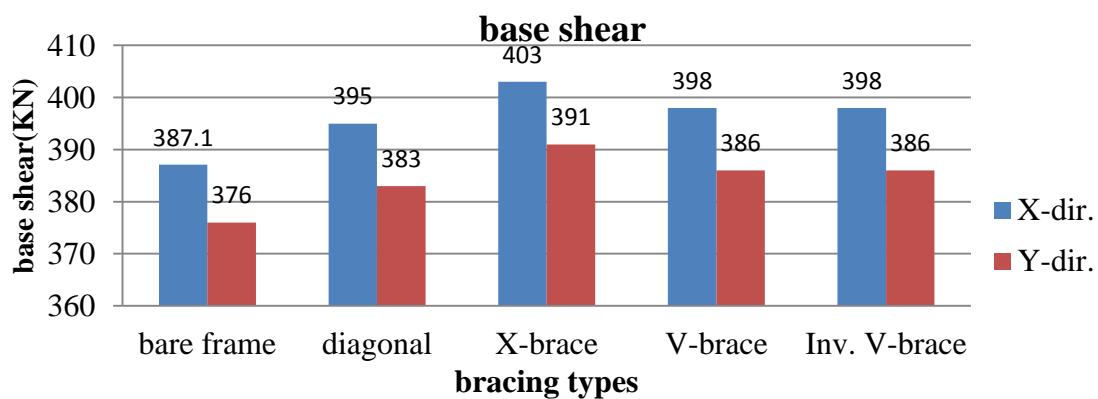
#### 4.1.2. Story force and base shear Comparison of 4story

Table; 4.3 story force and base shear of 4story

story	Elevatio m	bare frame		Diag. brace		X brace		V brace		Inv.V-brace	
		X	Y	X	Y	X	Y	X	Y	X	Y
	kN										
4	13.5	133	128.7	134.5	130.5	136.3	132.3	135.1	131.1	135.1	131.1
3	10.5	115	111.4	117.4	113.9	120.7	116.5	118.3	114.8	118.3	114.8
2	7.5	82	79.6	83.9	81.4	85.7	83.2	84.5	82	84.5	82
1	4.5	49.2	47.8	50.3	48.8	51.5	50	50.7	49.2	50.7	49.2
ground	1.5	8.5	8.3	8.8	8.6	9.2	8.9	9	8.8	9	8.8
<b>Base shear</b>		<b>387</b>	<b>376</b>	<b>395</b>	<b>383</b>	<b>403</b>	<b>391</b>	<b>398</b>	<b>386</b>	<b>398</b>	<b>386</b>

Table; 4.4 base shear of 4story

type of structures	base shear	
	X	Y
	kN	kN
bare frame	387	376
Diagonal	395	383
X-brace	403	391
V-brace	398	386
Inv. V-brace	398	386

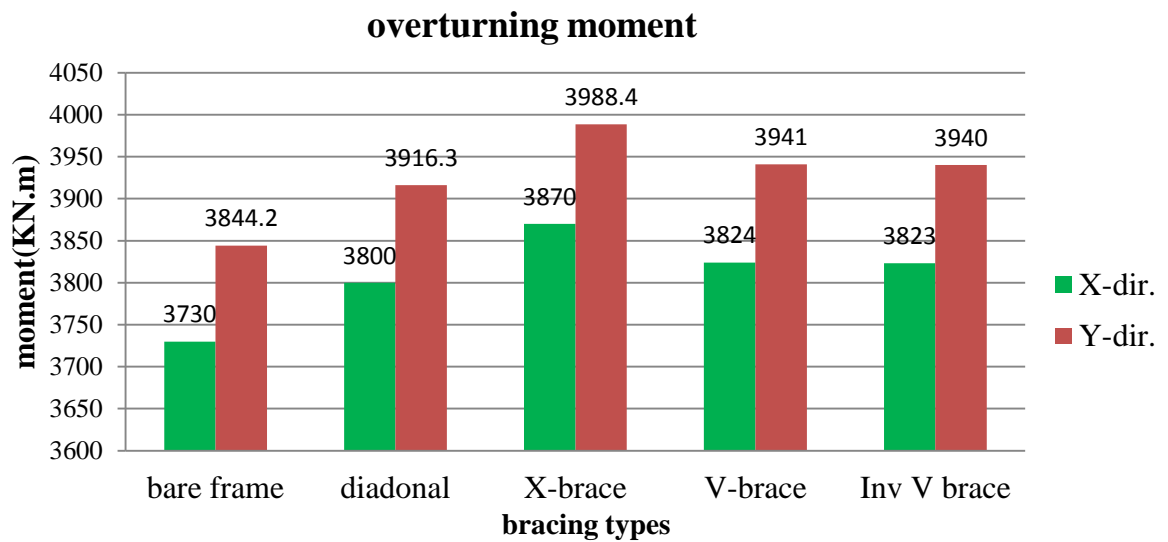


Figure; 4.4 base shear graphs of 4story

#### 4.1.3. Base overturning moment Comparison of 4story

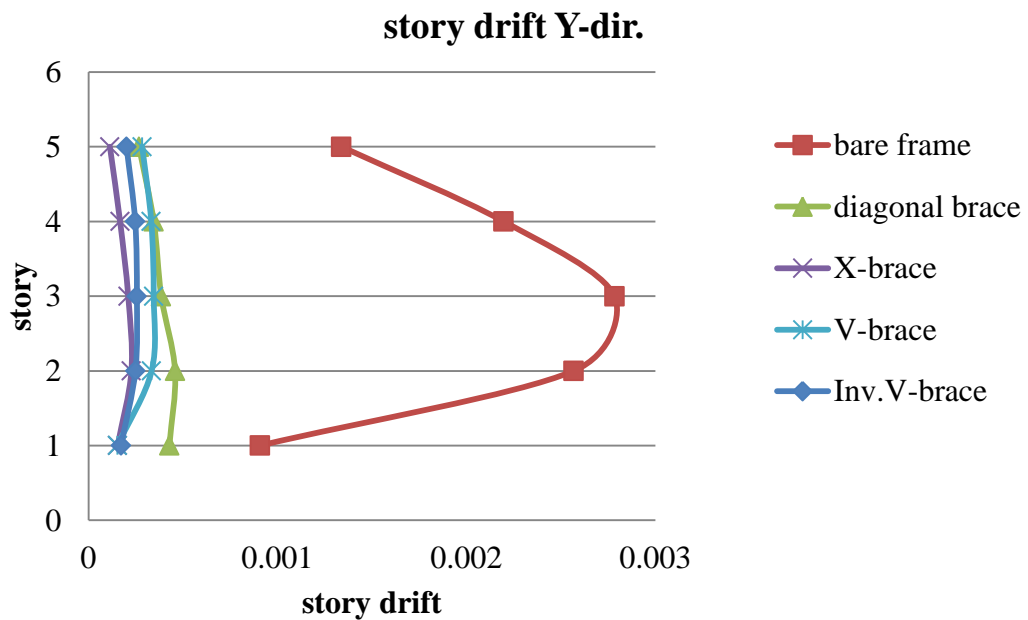
Table; 4.5 base overturning moment of 4story

Type of structure	overturning moment	
	X kN-m	Y kN-m
bare frame	3730	3844.2
Diagonal brace	3800	3916.3
X-brace	3870	3988.4
V-brace	3824	3941
Inv. V-brace	3823	3940



Figure; 4.5 overturning moment of structures of 4story

#### 4.1.4. Story drift Comparison of 4story



Figure; 4.6 story drift of story4 in Y-dir.

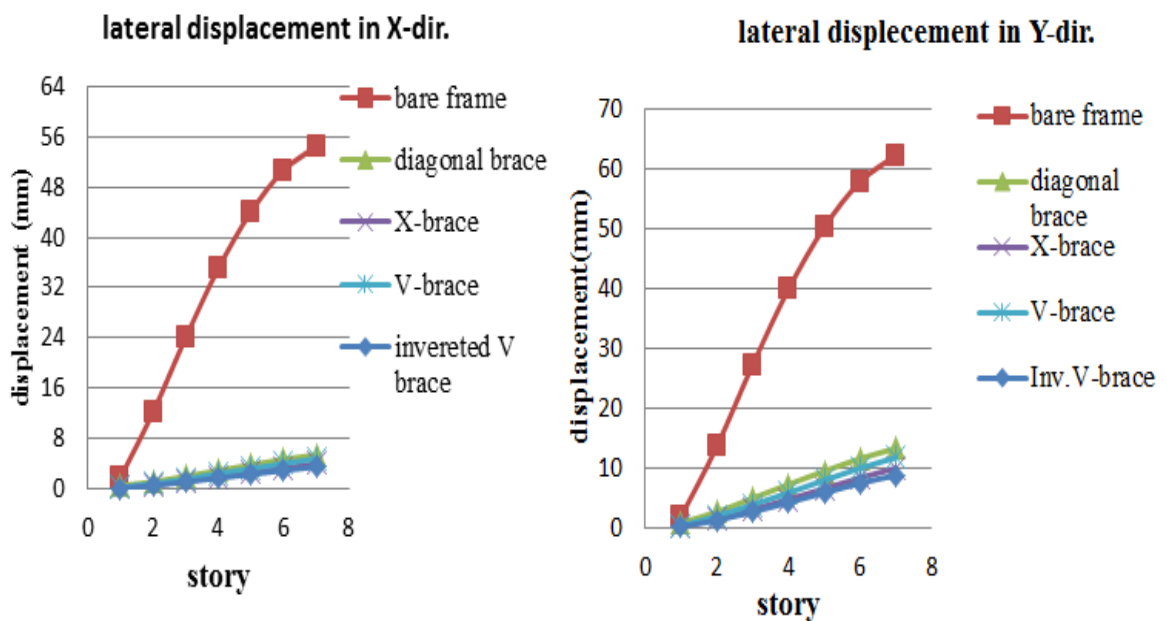
## 4.2. Six story

### 4.2.1. Lateral displacement Comparison of 6story

#### 4.2.1.1. Lateral displacement of 3x1 braced

Table; 4.6 lateral displacement of 6 story building where 3 of the 5 bays and 1 of the 3 bays are braced (3x1 braced)

story	elevatio n	bare frame		Diagonal		X-brace		V-brace		Inv.V-brace	
		X	Y	X	Y	X	Y	X	Y	X	Y
6	19.5	54.5	62.2	5.4	13.4	4	10.1	4.8	11.9	3.5	8.9
5	16.5	50.7	57.8	4.7	11.7	3.3	8.5	4	10.1	2.9	7.6
4	13.5	44.1	50.2	3.8	9.6	2.6	6.7	3.2	8.1	2.3	6
3	10.5	35.1	39.9	2.9	7.4	1.9	4.9	2.4	6	1.7	4.4
2	7.5	24.1	27.3	2	5.1	1.2	3.2	1.5	4	1.1	2.8
1	4.5	12.2	13.7	1.1	2.8	0.6	1.6	0.8	2.1	0.5	1.4
ground	1.5	1.8	2	0.5	1	0.2	0.5	0.2	0.5	0.1	0.4

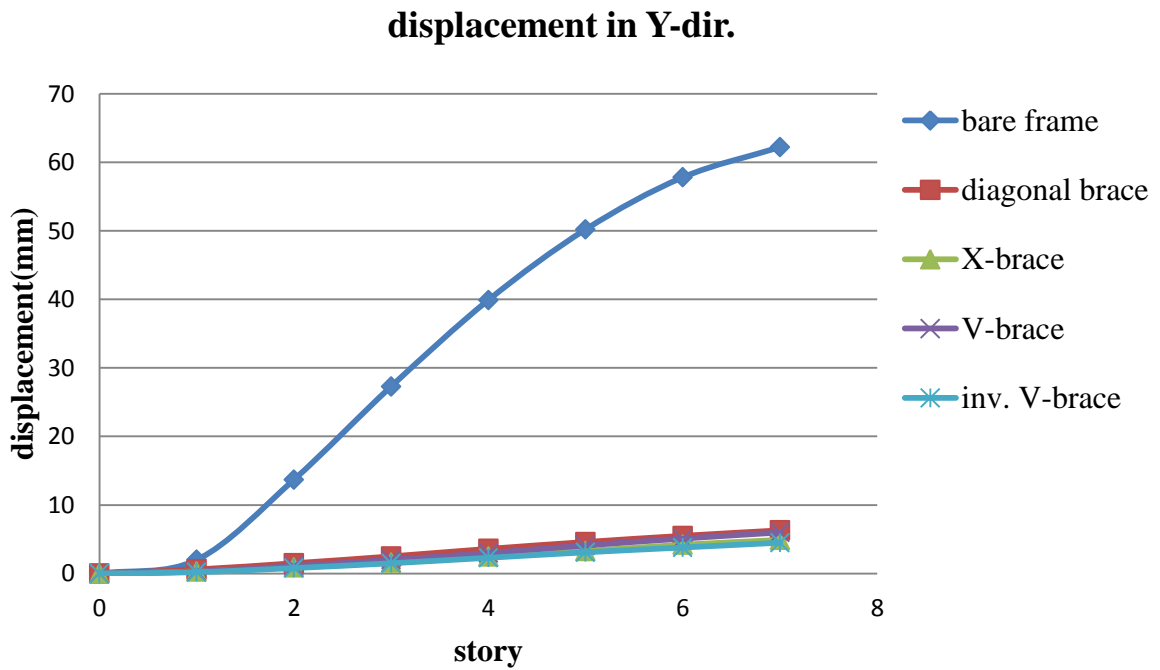


Figure; 4.7 Max. Lateral Displacements of 6story 3x1 braced

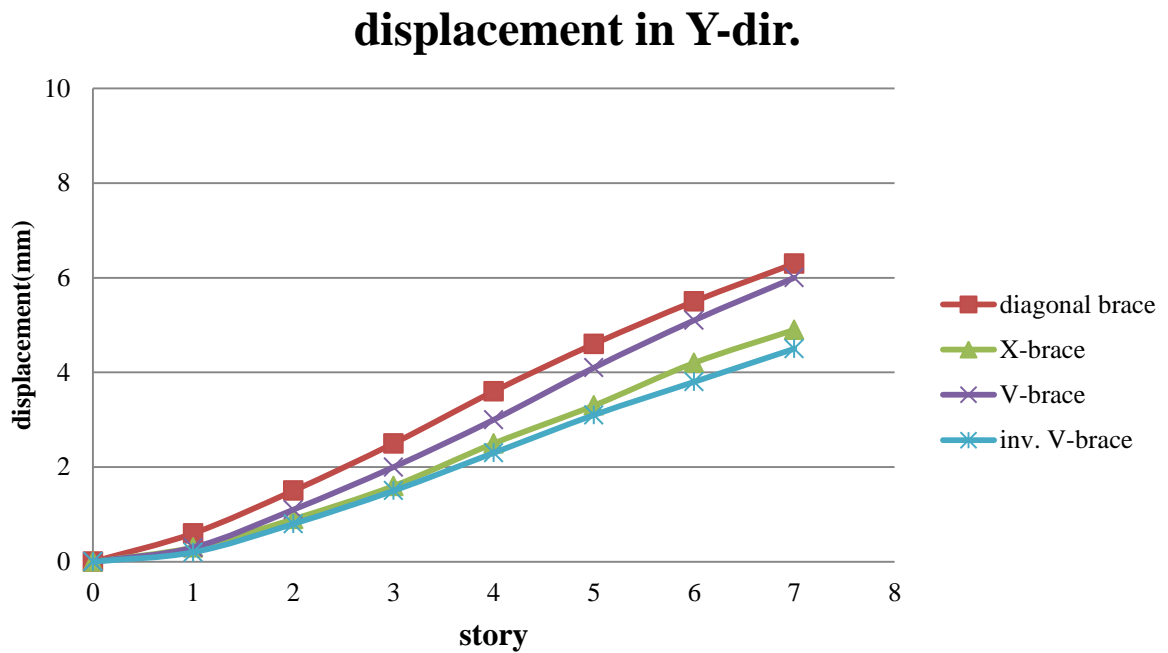
4.2.1.2. Lateral displacement of 6story corner braced

Table; 4.7 maximum lateral displacements of 6story corner braced

story	bare frame		diagonal brace		X-brace		V-brace		Inv.V-brace	
	X	Y	X	Y	X	Y	X	Y	X	Y
	mm		mm		mm		mm		mm	
6	54.5	62.2	6.2	6.3	4.9	4.9	5.9	6	4.4	4.5
5	50.7	57.8	5.4	5.5	4.1	4.2	5	5.1	3.8	3.8
4	44.1	50.2	4.5	4.6	3.3	3.3	4	4.1	3	3.1
3	35.1	39.9	3.5	3.6	2.4	2.5	3	3	2.2	2.3
2	24.1	27.3	2.4	2.5	1.6	1.6	2	2	1.4	1.5
1	12.2	13.7	1.4	1.5	0.8	0.9	1.1	1.1	0.7	0.8
ground	1.8	2	0.5	0.6	0.2	0.3	0.3	0.3	0.2	0.2



Figure; 4.8 Max. Lateral Displacements of 6story corner braced



Figure; 4.9 Max. Lateral Displacements of braced frame of 6story corner braced

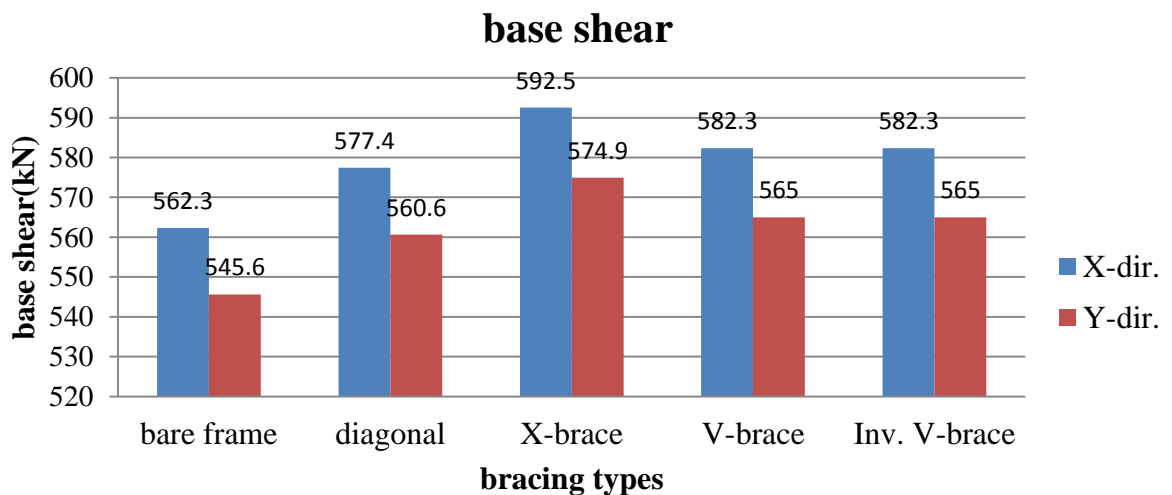
#### 4.2.2. Story force and base shear Comparison of 6story

Table; 4.8 story force and base shear of 6story

Story	Elevation m	bare frame		diagonal		X-brace		V-brace		Inv V-brace	
		X	Y	X	Y	X	Y	X	Y	X	Y
		kN									
6	19.5	139.2	135	141.6	137.4	144.1	139.8	142.4	138	142.4	138.2
5	16.5	131.1	127.2	135	131	138.8	134.7	136.2	132	136.2	132.2
4	13.5	107.2	104	110.4	107	113.6	110.2	111.4	108	111.4	108.1
3	10.5	83.4	81	85.9	83.3	88.3	85.7	86.7	84.1	86.7	84.1
2	7.5	59.6	57.8	61.3	59.5	63.1	61.2	61.9	60.1	61.9	60.1
1	4.5	35.7	34.7	36.8	35.7	37.9	36.7	37.2	36.1	37.2	36.1
ground	1.5	6.2	6	6.5	6.3	6.8	6.6	6.6	6.4	6.6	6.4
<b>base shear</b>		<b>562.3</b>	<b>545.6</b>	<b>577.4</b>	<b>560.6</b>	<b>592.5</b>	<b>574.9</b>	<b>582.3</b>	<b>565</b>	<b>582.3</b>	<b>565</b>

Table; 4.9 base shear of 6story

type of structures	base shear	
	X-	Y-dir.
	kN	
bare frame	562.3	545.6
diagonal	577.4	560.6
X-brace	592.5	574.9
V-brace	582.3	565
Inv. V-brace	582.3	565

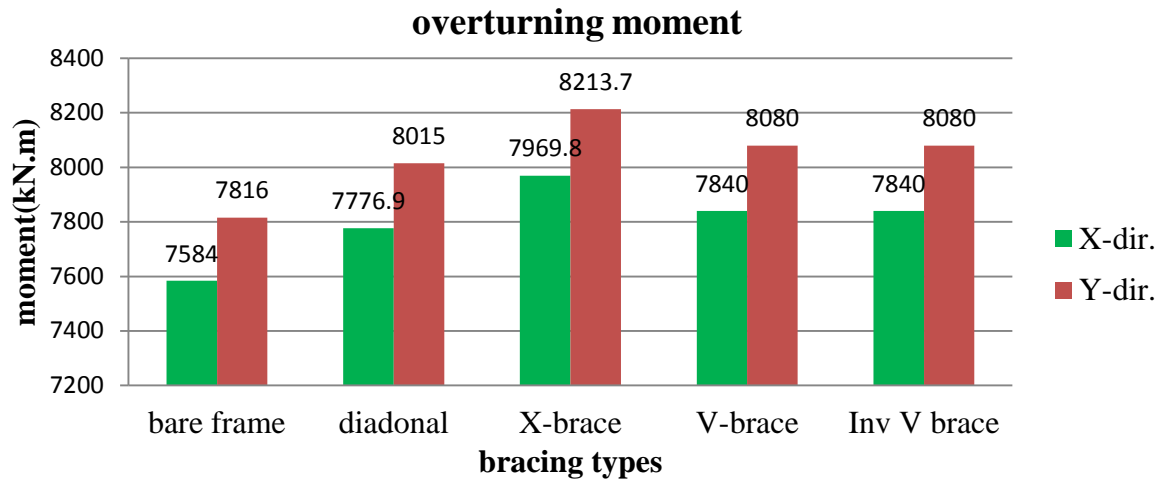


Figure; 4.10 base shear of 6story

#### 4.2.3. Base overturning moment Comparison of 6story

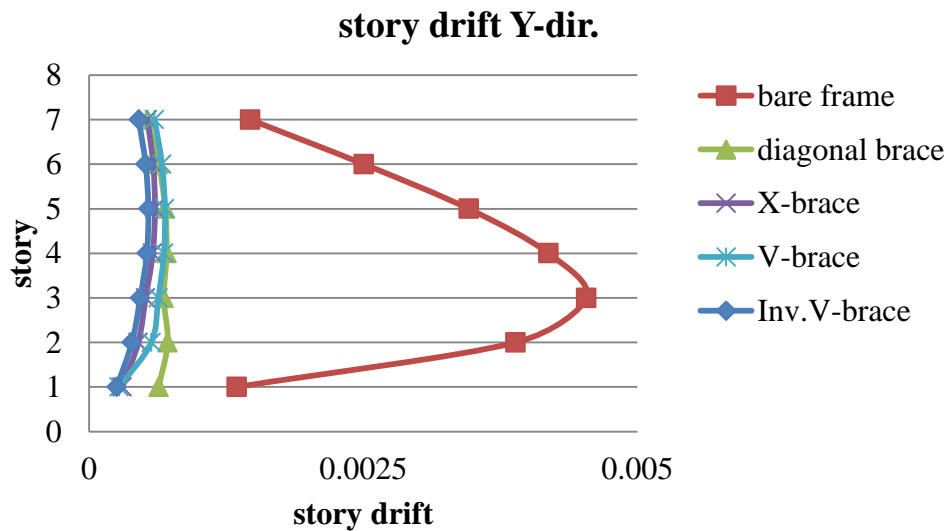
Table; 4.10 base overturning moment of 6story

type of structures	overturning moment	
	X kN-m	Y kN-m
bare frame	7584	7816
Diagonal	7776.9	8015
X-brace	7969.8	8213.7
V-brace	7840	8080
Inv V-brace	7840	8080



Figure; 4.11 base moment graph of 6story

#### 4.2.4. Story drift Comparison of 6story



Figure; 4.12 story drift of 6story

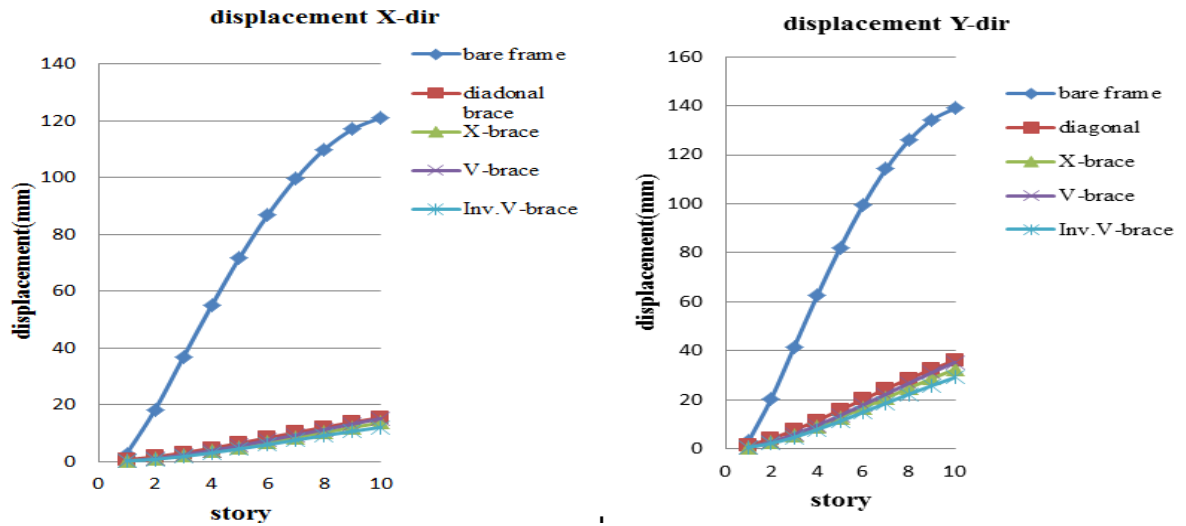
### 4.3. Nine Story

#### 4.3.1. Lateral displacement comparison of 9story

##### 4.3.1.1. Lateral displacement of 9story 3x1 braced

Table; 4.11 lateral displacement of 9 story building where 3 of the 5 bays and 1 of the 3 bays are braced (3x1 braced)

story	Elevation	bare frame		Diagonal-brace		X-brace		V-brace		Inv.V-brace	
		X	Y	X	Y	X	Y	X	Y	X	Y
	m	mm		mm		mm		mm		mm	
9	28.5	121.2	139	15.5	36	13.7	32.4	15.2	35.3	12.1	29.1
8	25.5	117.1	134.2	13.9	32.5	12.1	28.7	13.3	31.1	10.7	25.8
7	22.5	110	125.9	12.1	28.6	10.3	24.8	11.3	26.8	9.2	22.3
6	19.5	99.8	114.2	10.3	24.5	8.6	20.7	9.4	22.3	7.6	18.6
5	16.5	87	99.4	8.4	20.1	6.8	16.7	7.5	17.9	6	14.9
4	13.5	71.9	82	6.5	15.8	5.1	12.7	5.6	13.6	4.5	11.2
3	10.5	54.9	62.5	4.7	11.5	3.6	8.9	3.9	9.5	3.1	7.8
2	7.5	36.7	41.6	3.1	7.6	2.2	5.5	2.4	5.9	1.8	4.7
1	4.5	18.2	20.5	1.7	4.1	1.1	2.7	1.1	2.8	0.8	2.2
ground	1.5	2.7	3	0.7	1.4	0.3	0.7	0.3	0.7	0.2	0.6

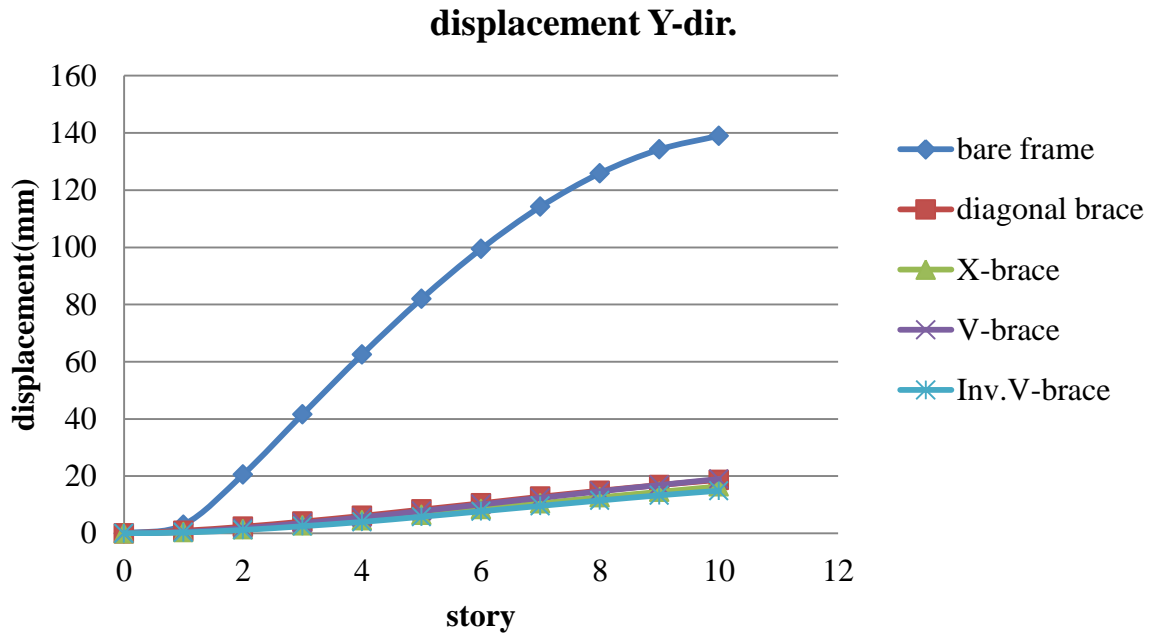


Figure; 4.13 lateral displacement graph of 9story 3x1 braced

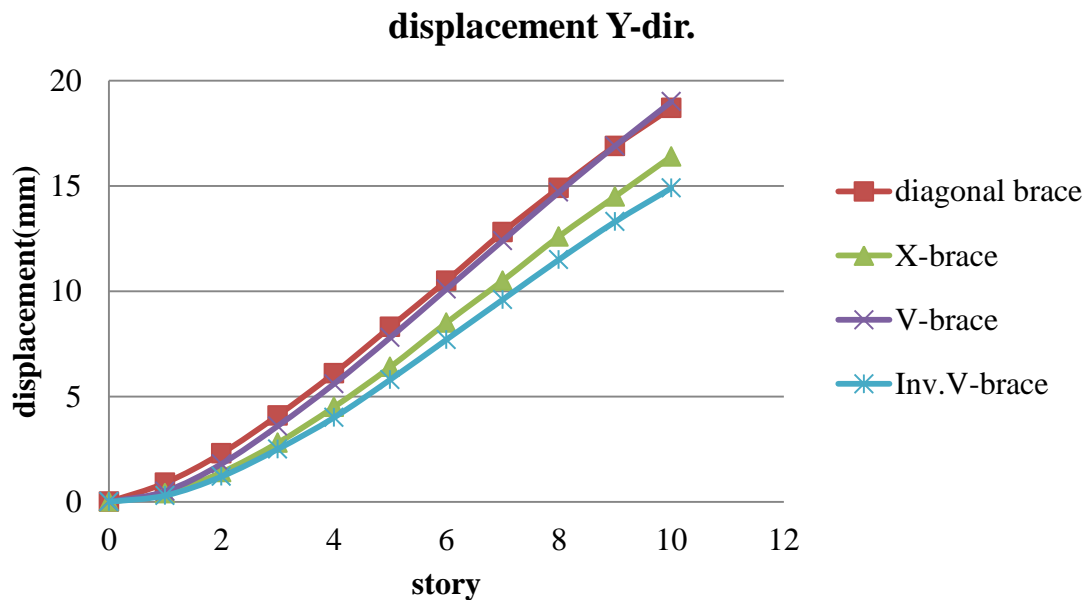
4.3.1.2. Lateral displacement of 9story corner braced

Table; 4.12 lateral displacement of 9story corner braced

story	Elevation	bare frame		Diagonal brace		X-brace		V-brace		Inv.V-brace	
		X	Y	X	Y	X	Y	X	Y	X	Y
		mm		mm		mm		mm		mm	
9	28.5	121.2	139	18.5	18.7	16.4	16.4	18.9	19	14.9	14.9
8	25.5	117.1	134.2	16.7	16.9	14.5	14.5	16.8	16.9	13.2	13.3
7	22.5	110	125.9	14.7	14.9	12.6	12.6	14.6	14.7	11.4	11.5
6	19.5	99.8	114.2	12.6	12.8	10.5	10.5	12.3	12.4	9.5	9.6
5	16.5	87	99.4	10.3	10.5	8.5	8.5	10	10.1	7.6	7.7
4	13.5	71.9	82	8.1	8.3	6.5	6.4	7.7	7.8	5.7	5.8
3	10.5	54.9	62.5	6	6.1	4.6	4.5	5.5	5.6	4	4
2	7.5	36.7	41.6	3.9	4.1	2.9	2.8	3.5	3.6	2.4	2.5
1	4.5	18.2	20.5	2.2	2.3	1.5	1.4	1.8	1.8	1.2	1.2
ground	1.5	2.7	3	0.8	0.9	0.8	0.4	0.4	0.5	0.3	0.3



Figure; 4.14 graph of lateral displacement of 9story corner braced



Figure; 4.15 graph of lateral displacement of braced frame of 9story corner braced

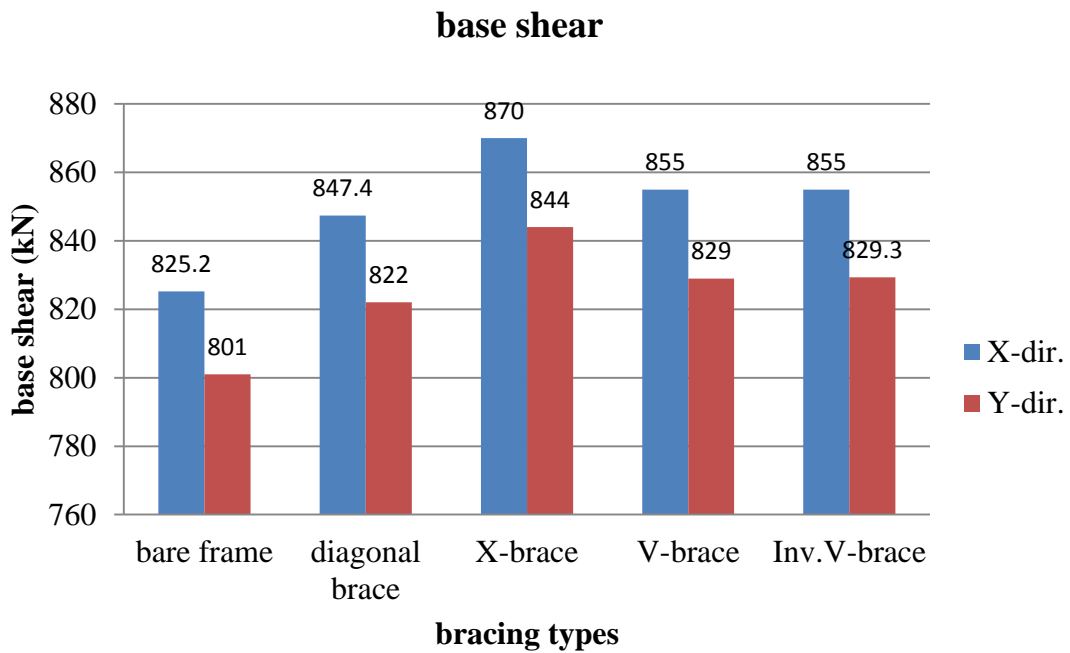
### 4.3.2. Story force and base shear Comparison of 9story

Table; 4.13 story force and base shear of 9story

Story	Elevatio m	bare frame		Diagonal		X-brace		V-brace		Inv.V-brace	
		X	Y	X	Y	X	Y	X	Y	X	Y
		kN									
9	28.5	144.4	140.1	146.8	142.5	149.2	144.	147.	143.	147.	143.
8	25.5	143.8	139.5	147.9	143.5	152.1	147.	149.	144.	149.	144.
7	22.5	126.8	123.1	130.5	126.6	134.1	130.	131.	127.	131.	127.
6	19.5	109.9	106.7	113.1	109.7	116.3	112.	114.	110.	114.	110.
5	16.5	93.1	90.3	95.7	92.9	98.4	95.5	96.6	93.7	96.6	93.7
4	13.5	76.1	73.8	78.3	76	80.5	78.1	79	76.7	79	76.7
3	10.5	59.2	57.4	60.9	59.1	62.6	60.7	61.5	59.6	61.5	59.6
2	7.5	42.3	41.1	43.5	42.2	44.7	43.4	43.9	42.6	43.9	42.6
1	4.5	25.4	24.6	26.1	25.3	26.8	26	26.3	25.6	26.3	25.6
ground	1.5	4.4	4.23	4.6	4.6	4.8	4.8	4.7	4.5	4.7	4.5
<b>base shear</b>		<b>825.2</b>	<b>800.7</b>	<b>847.3</b>	<b>822.2</b>	<b>869.5</b>	<b>843.</b>	<b>854.</b>	<b>829.</b>	<b>854.</b>	<b>829.</b>

Table; 4.14 base shear of 9story

types of structure	base shear(kN)	
	X	Y
bare frame	825.2	801
diagonal brace	847.4	822
X-brace	870	844
V-brace	855	829
Inv.V-brace	855	829.3

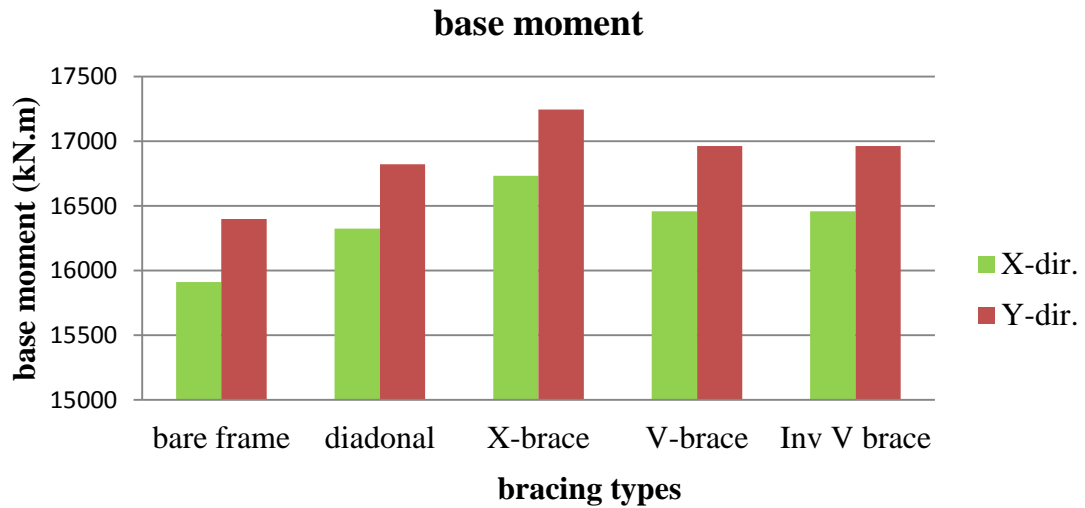


Figure;4.16 base shear of 9story

### 4.3.3. Base Overturning moment Comparison of 9story

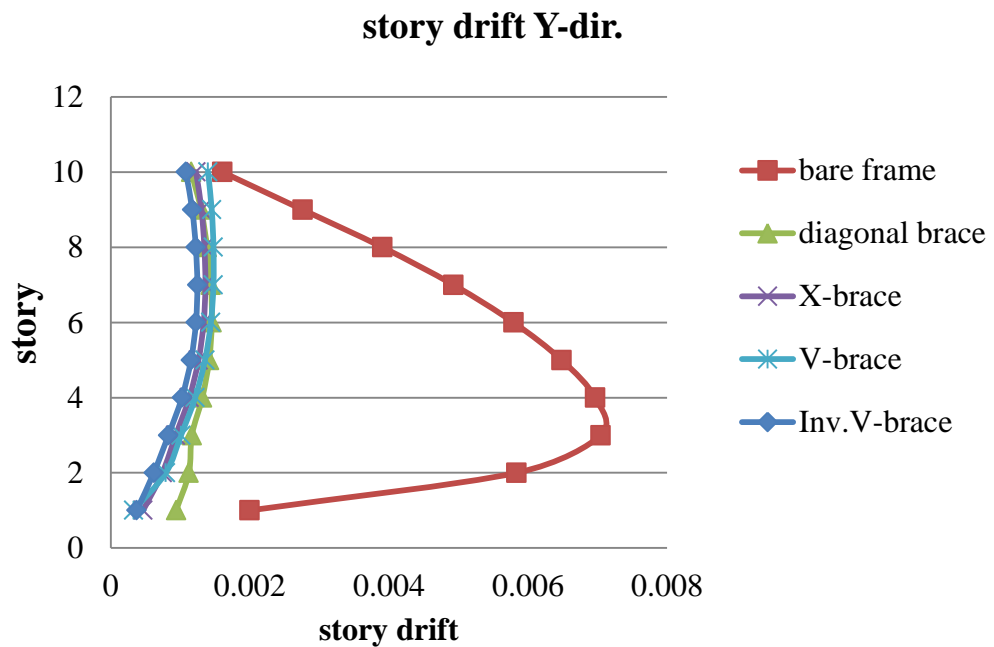
Table; 4.15 overturning moment of 9story

type of structures	overturning moment	
	X ( kN-	Y (kN-m)
bare frame	15912	16399
Diagonal brace	16323	16822
X-brace	16733	17245
V-brace	16459	16962
Inv V brace	16459	16962



Figure; 4.17 base overturning moment of 9story

#### 4.3.4. Story drift Comparison of 9story



Figure; 4.18 story drift graph of 9story

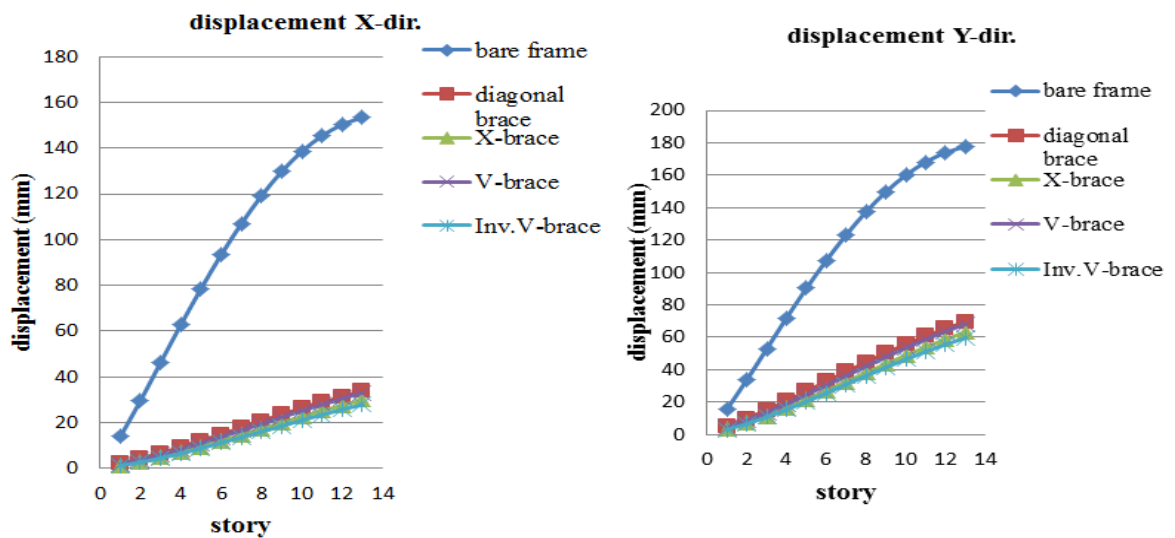
## 4.4. Thirteen Story

### 4.4.1. Lateral displacement comparison of 13story

#### 4.4.1.1. Lateral displacement of 3x1 braced

Table; 4.16 1 lateral displacement of 13 story building where 3 of the 5 bays and 1 of the 3 bays are braced (3x1 braced)

Story	Elevation m	bare frame		diagonal brace		X-brace		V-brace		Inv.V-brace	
		X mm	Y mm	X mm	Y mm	X mm	Y mm	X mm	Y mm	X mm	Y mm
13	40.5	153.4	177.8	33.7	69.4	30	63.2	33.1	68.2	27.9	59.7
12	37.5	150.3	174	31.4	65.4	27.6	58.8	30.6	63.7	25.8	55.7
11	34.5	145.4	168.2	28.9	60.8	25.2	54	27.9	58.7	23.5	51.2
10	31.5	138.6	160.1	26.3	55.8	22.6	48.9	25.2	53.5	21.1	46.5
9	28.5	129.8	149.8	23.6	50.4	20	43.6	22.4	47.9	18.6	41.5
8	25.5	119.2	137.5	20.7	44.7	17.3	38.2	19.5	42.2	16.1	36.3
7	22.5	107	123.3	17.8	38.8	14.6	32.6	16.6	36.3	13.6	31
6	19.5	93.4	107.5	14.9	32.8	12	27.1	13.8	30.4	11.2	25.6
5	16.5	78.6	90.3	12.1	26.7	9.5	21.6	11	24.6	8.8	20.4
4	13.5	62.8	72	9.3	20.8	7.1	16.4	8.4	18.9	6.5	15.4
3	10.5	46.3	52.9	6.7	15.1	5	11.5	6	13.5	4.5	10.8
2	7.5	29.6	33.7	4.4	9.8	3.1	7.2	3.8	8.6	2.7	6.6
1	4.5	13.9	15.6	2.3	5	1.5	3.5	1.9	4.3	1.3	3.2
ground	1.5	2	2.2	0.8	1.4	0.4	0.9	0.5	1	0.3	0.7

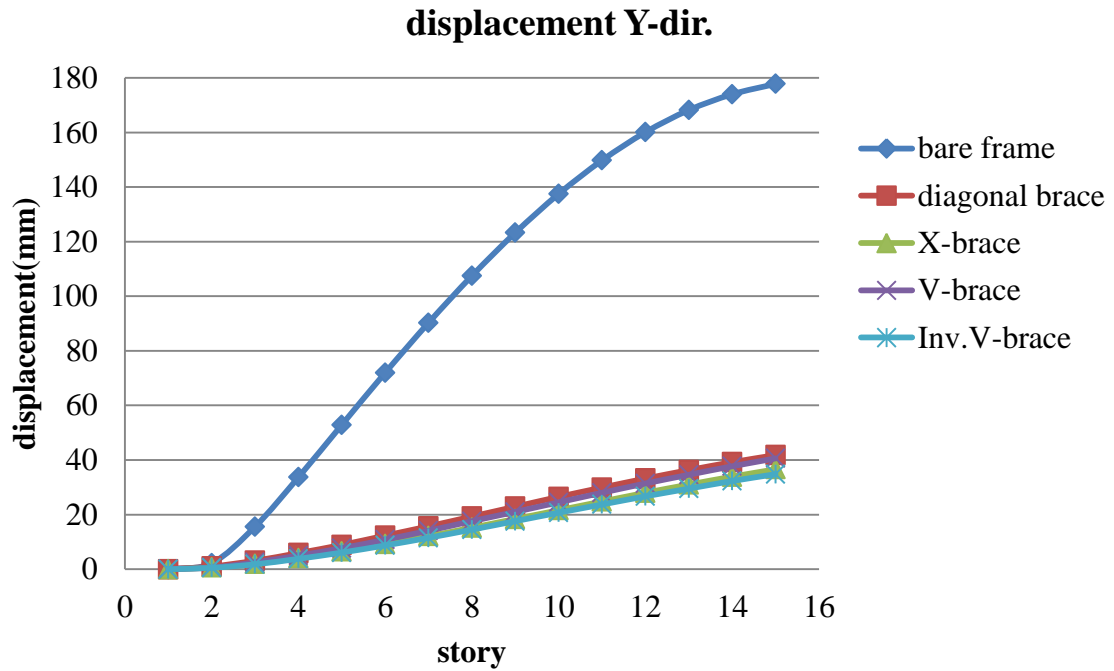


Figure; 4.19 lateral displacement of 13story (3x1braced)

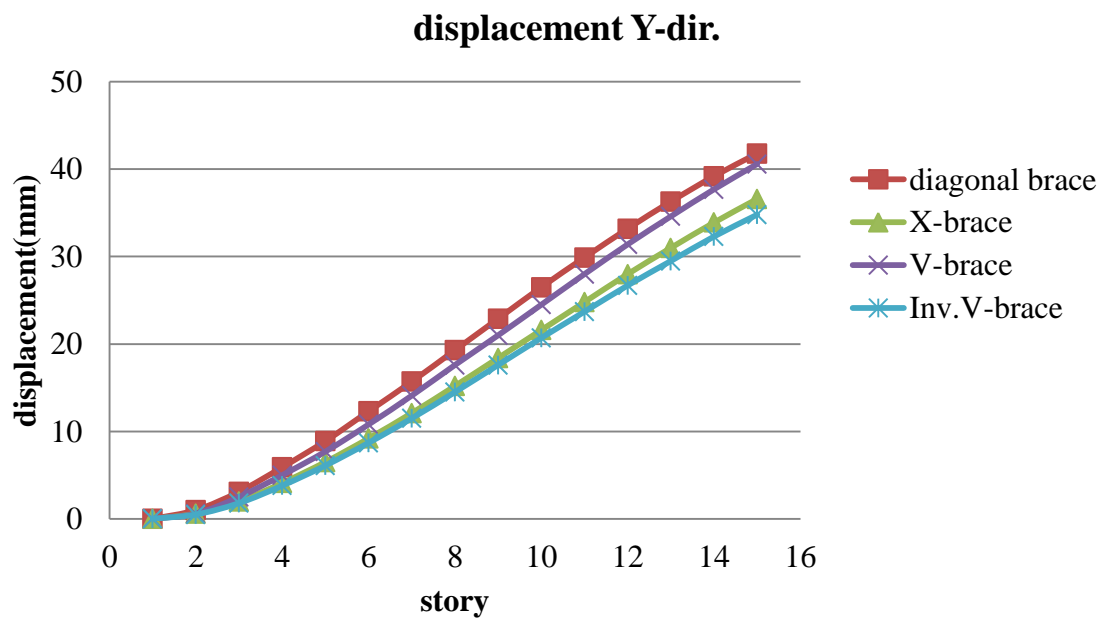
4.4.1.2. Lateral displacement of corner braced

Table; 4.17 lateral displacement of 13story corner braced

Story	Elevati m	bare frame		diagonal brace		X-brace		V-brace		Inv.V-brace	
		X	Y	X	Y	X	Y	X	Y	X	Y
		mm		mm		mm		mm		mm	
13	40.5	153.4	177.8	40.8	41.8	36.1	36.6	39.9	40.6	34.2	34.8
12	37.5	150.3	174	38.3	39.2	33.4	33.9	37.1	37.7	31.7	32.3
11	34.5	145.4	168.2	35.4	36.3	30.5	31	34	34.6	29.1	29.5
10	31.5	138.6	160.1	32.4	33.2	27.5	28	30.8	31.4	26.3	26.7
9	28.5	129.8	149.8	29.2	29.9	24.4	24.8	27.5	28	23.3	23.7
8	25.5	119.2	137.5	25.8	26.5	21.3	21.6	24.1	24.5	20.3	20.7
7	22.5	107	123.3	22.3	22.9	18.1	18.4	20.7	21	17.3	17.6
6	19.5	93.4	107.5	18.8	19.3	15	15.2	17.2	17.6	14.3	14.5
5	16.5	78.6	90.3	15.3	15.7	11.9	12.1	13.9	14.1	11.3	11.5
4	13.5	62.8	72	11.9	12.3	9	9.2	10.6	10.8	8.5	8.7
3	10.5	46.3	52.9	8.6	8.9	6.3	6.5	7.6	7.7	5.9	6.1
2	7.5	29.6	33.7	5.7	5.9	4	4.1	4.9	5	3.7	3.8
1	4.5	13.9	15.6	3	3.1	2	2	2.5	2.5	1.8	1.8
ground	1.5	2	2.2	0.9	1	0.5	0.6	0.6	0.6	0.4	0.5



Figure; 4.20 lateral displacement of 13story (corner braced)



Figure; 4.21 lateral displacement of braced frame of 13story (corner braced)

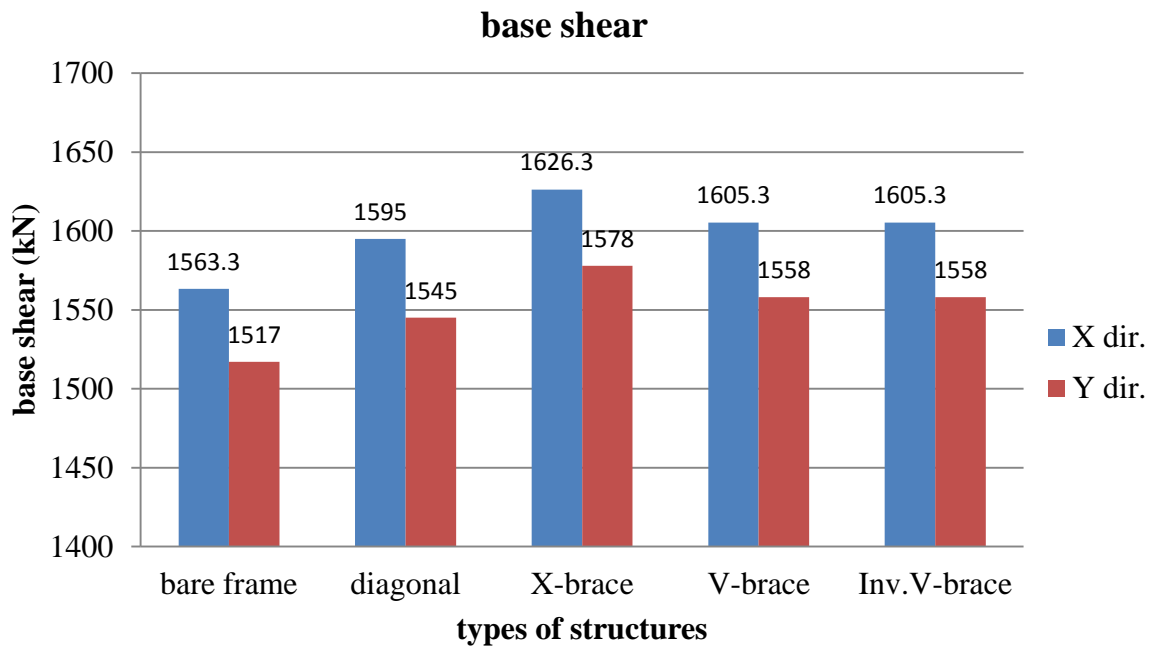
#### 4.4.2. Story force and base shear Comparison of 13story

Table; 4.18 story force and base shear of 13story

Story	Elevation	bare frame		diagonal		X-brace		V-brace		Inv.V-brace	
		X	Y	X	Y	X	Y	X	Y	X	Y
	m	kN									
13	40.5	186.2	180.6	188.5	182.9	190.9	185.3	189.3	183.7	189.3	183.7
12	37.5	204.2	198.1	208.5	202.3	212.8	206.5	210	203.7	210	203.7
11	34.5	187.9	182.3	191.8	186.1	195.8	190	193.2	187.4	193.2	187.4
10	31.5	171.5	166.4	175.1	169.9	178.8	173.5	176.4	171.1	176.4	171.1
9	28.5	155.2	150.6	158.5	153.8	161.7	156.9	159.6	154.8	159.6	154.8
8	25.5	138.9	134.7	141.8	137.6	144.7	140.4	142.8	138.5	142.8	138.5
7	22.5	122.5	118.9	125.1	121.4	127.7	123.9	126	122.2	126	122.2
6	19.5	106.2	103	108.4	105.2	110.7	107.4	109.2	105.9	109.2	105.9
5	16.5	89.9	87.18	91.7	89.02	93.6	90.86	92.4	89.6	92.4	89.6
4	13.5	73.5	71.3	75.1	72.8	76.6	74.34	75.6	73.3	75.6	73.3
3	10.5	57.2	55.5	58.4	56.7	59.6	57.82	58.8	57.1	58.8	57.1
2	7.5	40.8	39.6	41.7	40.5	42.6	41.3	42	40.7	42	40.7
1	4.5	24.5	23.8	25.1	24.3	25.5	24.78	25.2	24.5	25.2	24.5
ground	1.5	4.9	4.7	5.1	4.9	5.2	5.038	5.1	4.9	5.1	4.9
<b>base shear</b>		<b>1563.3</b>	<b>1516.9</b>	<b>1595</b>	<b>1547</b>	<b>1626</b>	<b>1578</b>	<b>1605</b>	<b>1558</b>	<b>1605</b>	<b>1558</b>

Table; 4.19 base shear of 13story

types of structures	base shear(kN)	
	X-Dir.	Y-Dir.
bare frame	1563.3	1517
Diagonal brace	1595	1547.4
X-brace	1626.3	1578
V-brace	1605.3	1558
Inv.V-brace	1605.3	1558

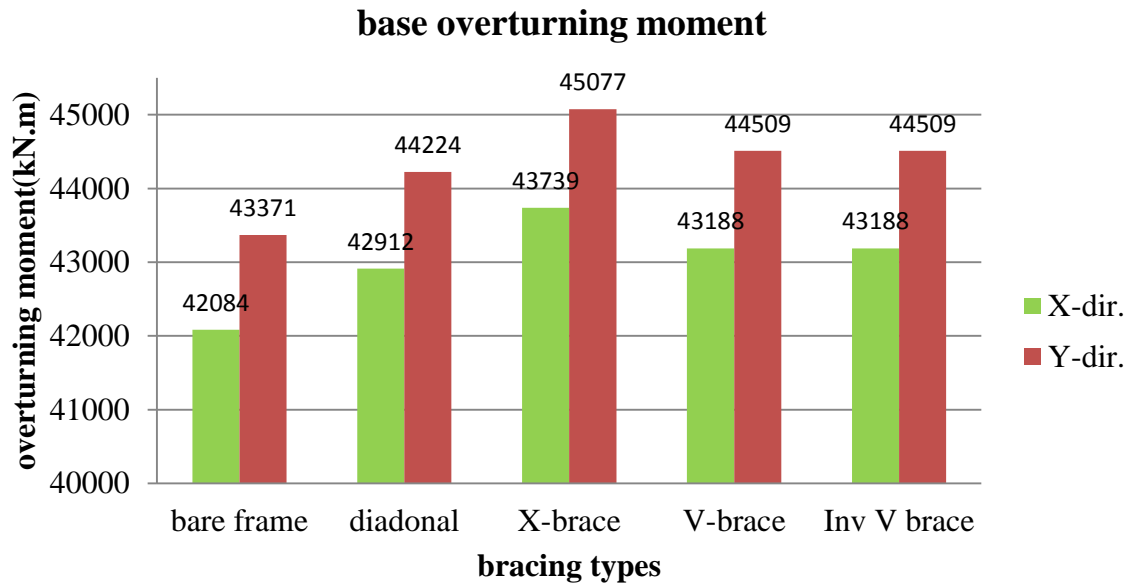


Figure; 4.22 base shear graph of 13story

#### 4.4.3. Base overturning moment comparison of 13story

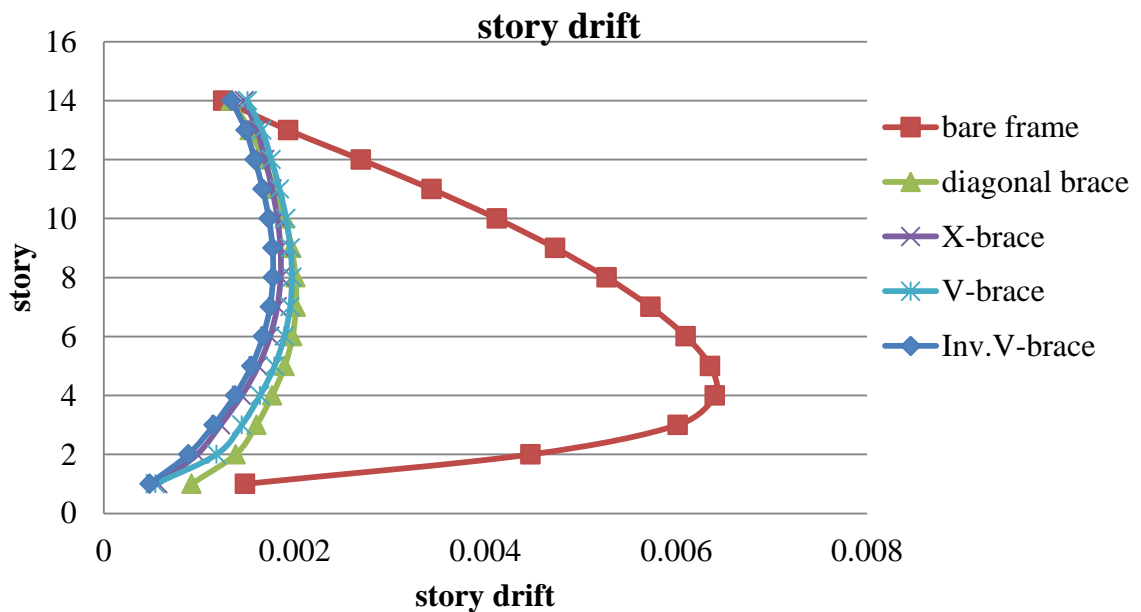
Table; 4.20 base overturning moments

type of structures	overturning moment	
	X (kN-	Y (kN-
bare frame	42084	43371
Diagonal brace	42912	44224
X-brace	43739	45077
V-brace	43188	44509
Inv V brace	43188	44509



Figure; 4.23 base overturning moment graph of 13story

#### 4.4.4. Story drift Comparison of 13story



Figure; 4.24 story drift of 13story

## 4.5. Summarized Discussions

### 4.5.1. Discussion on lateral displacement

From the tables and graphs plotted above the lateral displacement results has been discussed as follow. Figure 4.1, 4.7, 4.13, 4.19 and figure 4.2, 4.3, 4.14, 4.15, 4.20, 4.21 shows that the maximum lateral displacement for seismic load for 3x1braced case and corner braced case respectively at different story level and lateral displacements of the structures for various bracing systems have been compared.

The percentage reduction in top story displacement of all bracing types with respect to bar frame (without bracing) of all storied shown in table 4.21 and table 4.22 for 3x1braced and table 4.23 and table 4.24 for corner braced below noted that the lateral displacement drastically reduced after the application of bracing systems, the story displacement of building are reduced sequentially for bracings diagonal brace, V-brace and X-brace and Inv.V-brace respectively and the maximum reduction in the lateral displacement is observed after the application of Inv.V-bracing (chevron bracing) and X-bracing. As the story/height of building increase the rate of reduction of displacement decrease in a little amount.

Table; 4.21 percentage reduced of top story lateral displacement in X-dir. where 3 of the 5 bays and 1 of the 3 bays are braced (3x1 braced)

building	top story displacement comparison X-dir.								
	bare frame	diagonal brace		X-brace		V-brace		Inv.V-brace	
	mm	mm	% reduced compared to bare frame	mm	% reduced compared to bare frame	mm	% reduced compared to bare frame	mm	% reduced compared to bare frame
g+3	24.6	2.6	89.4	1.2	95.1	2.3	90.7	1.6	93.5
g+5	54.5	5.4	90.1	4	92.7	4.8	91.2	3.5	93.6
g+8	121.2	16	87.2	14	88.7	15	87.5	12	90
g+12	153.4	34	78	30	80.4	33	78.4	28	81.8

Table; 4.22 percentage reduced of top story lateral displacement in Y-dir. where 3 of the 5 bays and 1 of the 3 bays are braced (3x1 braced)

		top story displacement comparison Y-dir.							
buildin g	bare frame	diagonal		X-brace		V-brace		Inv.V-brace	
	mm	mm	% reduced compared to bare frame	mm	% reduced compared to bare frame	mm	% reduced compared to bare frame	mm	% reduced compared to bare frame
g+3	28	4.7	83.2	2.3	91.8	4.1	85.4	3	89.3
g+5	62.2	14	78.5	10	83.8	12	80.9	8.9	85.7
g+8	139	36	74.1	33	76.7	35	74.6	29	79.1
g+12	177.8	69	61	62	64.5	68	61.6	60	66.4

Table; 4.23 percentage reduced of top story lateral displacement in X-dir. For corner braced

		top story displacement comparison X-dir.							
building	bare frame	diagonal brace		X-brace		V-brace		Inv.V-brace	
	mm	mm	% reduced compared to bare frame	mm	% reduced compared to bare frame	mm	% reduced compared to bare frame	mm	% reduced compared to bare frame
g+3	24.6	2.4	90.2	1.7	93.1	2.1	91.5	1.6	93.5
g+5	54.5	6.2	88.6	4.9	91	5.9	89.2	4.4	91.9
g+8	121.2	18.5	84.7	16.4	86.5	18.9	84.4	14.9	87.7
g+12	153.4	40.8	73.4	36.1	76.5	39.9	74	34.2	77.7

Table; 4.24 percentage reduced of top story lateral displacement in Y-dir. For corner braced

		top story displacement comparison Y-dir.							
building	bare frame	diagonal		X-brace		V-brace		Inv.V-brace	
	mm	mm	% reduced compared to bare frame	mm	% reduced compared to bare frame	mm	% reduced compared to bare frame	mm	% reduced compared to bare frame
g+3	28	2.5	91.1	1.7	93.9	2.1	92.5	1.6	94.3
g+5	62.2	6.3	89.9	4.9	92.1	6	90.4	4.5	92.8
g+8	139	19	86.5	16.4	88.2	19	86.3	14.9	89.3
g+12	177.8	41.8	76.5	36.6	79.4	40.6	77.2	34.8	80.4

From the two cases studied in this paper the overall reduction of lateral displacement is better in the case of corner braced frame. When the frame is braced in corner Y-dir configuration the

displacement in the longer direction (5 bay) shows a little increase in value, but in the shorter direction (3 bay) the value decrease in large amount. As seen from the table below the percentage decrease in shorter direction is greater than the percentage increase in the longer direction when corner braced is compared with 3x1 braced for all story buildings. It is noticed that the corner braced from is better than 3x1 braced frames in reducing lateral displacement.

Table; 4.25 comparison of top displacement for 3x1 braced and corner braced in X-dir.

percentage increase of displacement when bracing applied on corners of the frame in X-direction												
type of brace	g+3			g+5			g+8			g+12		
	3x1 braced	corner braced	% d/ce	3x1 braced	corner braced	% d/ce	3x1 braced	corner braced	% d/ce	3x1 braced	corner braced	% d/ce
	mm	mm		mm	mm		mm	mm		mm		
diagonal	2.6	2.65	8.1	5.4	6.2	15	15.5	18.5	19	33.7	40.8	21
X-brace	1.2	1.7	42	4	4.9	23	13.7	16.4	20	30	36.1	20
V-brace	2.3	2.4	11	4.8	5.9	23	15.2	18.9	24	33.1	39.9	21
Inv.V-brace	1.6	1.6	0	3.5	4.4	26	12.1	14.9	23	27.9	34.2	23

Table; 4.26 comparison of top displacement for 3x1 braced and corner braced in Y-dir.

percentage decrease of displacement when bracing applied on corner of the frame in Y-direction												
type of brace	g+3			g+5			g+8			g+12		
	3x1 braced	corner braced	% d/ce	3x1 braced	corner braced	% d/ce	3x1 braced	corner braced	% d/ce	3x1 braced	corner braced	% d/ce
	mm	mm		mm	mm		mm	mm		mm		
diagonal	4.7	2.5	46.9	13.4	6.3	53	36	18.7	48.1	69.4	41.8	39.8
X-brace	2.3	1.7	26.1	10.1	4.9	51.5	32.4	16.4	49.4	63.2	36.6	42.1
V-brace	4.1	2.1	48.8	11.9	6	49.6	35.3	19	46.2	68.2	40.6	40.5
Inv.V-brace	3	1.6	46.7	8.9	4.5	49.5	29.1	14.9	48.8	59.7	34.8	41.8

#### 4.5.2. Discussion on base shear

The seismic forces of the upper stories are transmitted to lower stories and finally to the ground through the foundation. Base shear for cases, 3x1 braced and corner braced are similar.

Table 4.3, table 4.8, table4.13 and table 4.18 shows the story force and story shear at different floor levels for all structural systems in both X and Y directions. Table4.4, table4.9, table4.14 and table4.19 shows the comparison of base shear of all bracing types .There is a considerable change of shear force and base shear in both directions when bracing has applied to the RC frame.

Table 4.27 below shows that the summery of base shear of all story buildings. The maximum shear force of all systems are at base level and its highest for all brace frame as compared to bare frame (without bracing) and from table 4.27 the result of X-braced frame is found high in both longer(X) and shorter(Y) directions as compared to other bracing systems.

Table; 4.27 base shear comparison of all story of all systems

types of structures	base shear X-dir.				base shear Y-dir.			
	g+3	g+5	g+8	g+12	g+3	g+5	g+8	g+12
	kN				kN			
bare frame	387	562.3	825.2	1563.3	376	545.6	801	1517
diagonal	395	577.4	847.4	1595	383	560.6	822	1547.4
X-brace	403	592.5	870	1626.3	391	574.9	844	1578
V-brace	398	582.3	855	1605.3	386	565	829	1558
Inv.V-brace	398	582.3	855	1605.3	386	565	829.3	1558

Table; 4.28 base shear of X-bracing compared to bare frame.

building	base shear X-dir.(kN)		
	bare frame	X-brace	% increase
g+3	387	403	4.1
g+5	562.3	592.5	5.4
g+8	825.2	870	5.4
g+12	1563.3	1626.3	4

From reduction of displacement seen both Inv.V-bracing and X-bracing type increases shear resisting capacity of the building.

#### 4.5.3. Discussion on base overturning moment

The overturning moment of all brace frame structures increase as compared with bare frame (without bracing) as seen on figure 4.3, 4.11, 4.17 and 4.23 above. On table 4.29 below the base overturning moment of all buildings of all story are compared and the result shows the moment is high for all braced frames as compared to bare frame (without bracing). Base overturning moment of X-bracing has found high as compared to other bracing systems as seen in table 4.30 below.

Table; 4.29 base overturning moments of all stories of all systems

types of structure	base overturning moment X-dir.				base overturning moment Y-dir.			
	g+3	g+5	g+8	g+12	g+3	g+5	g+8	g+12
	kN.m				kN.m			
bare frame	3730	7584	15912	42084	3844.2	7816	16399	43371
diagonal	3800	7776.9	16323	42912	3916.3	8015	16822	44224
X-brace	3870	7969.8	16733	43739	3988.4	8213.7	17245	45077
V-brace	3824	7840	16459	43188	3941	8080	16962	44509
Inv.V-brace	3823	7840	16459	43188	3940	8080	16962	44509

Table; 4.30 base overturning moment of X-brace compared to bare frame

building	base overturning moment		
	bare frame	X-brace	% increase
g+3	3844.2	3988.4	3.8
g+5	7816	8213.7	5.1
g+8	16399	17245	5.2
g+12	43371	45077	3.9

#### 4.5.4. Discussion on story drift

The story drift is very important parameter in the analysis and design of buildings. If the story drift values at each floor level reach their maximum allowable limit, then roof displacement will reach undesirable values.

Maximum story drift for bare frame (frame without bracing) is in the story just above 2<sup>nd</sup> and 3<sup>rd</sup> story for all buildings, this result shows the frame structure deflects in shear configuration where the rate of change of deflection goes on reduced with height as seen from the analysis result in figure4.6, figure4.12, figure4.18, and figure4.24 for story4 (G+3), story6 (G+5), story9 (G+8), and story13 (G+12) respectively.

In braced frame maximum story drift is found around middle height and above the middle height of the building. As shown from analysis results on figure listed above. Deflection pattern is showing flexural deformation at a lower height in which rate of deflection increased and follows the shear configuration in upper height. This indicated the bracing governs flexural deflection, which is good. The Inv.V-bracing (chevron) is found better in the reduction of story drift as compared to other bracing system. X-bracing, V-bracing and diagonal bracing has also reduced the story drift in large extent respectively.

## CHAPTER FIVE

### 5. CONCLUSION AND RECOMMENDATION

#### 5.1. Conclusion

The steel bracing system has not only improved displacement capacity of reinforced concrete structures, but also the lateral stiffness and strength capacity of the structures by increasing its shear capacity.

Inv.V-brace (chevron) and X-bracing of steel bracing types has found in the most efficient in terms of story displacement and story drift reduction when bracing is provided on two parallel sides of the building.

Story drift should be limited because deflection must be limited during the earth quake to protect the damage of structural elements, especially nonstructural elements, and hence the provisions of steel bracing for the RC structure give adequate stiffness for the structure and among the used bracing type Inv.V and X-bracing types have been given better result in reduction of story drift.

The base shear capacity of steel braced frame is increased as compared to bare frame (without bracing) building which indicates that the stiffness of building has increased.

Inv.V-bracing and X-bracing type is found most efficient in increasing the shear capacity of RC frame building which indicates that Inv.v-brace and X-brace type of steel bracing significantly contributes to the structural stiffness.

The base overturning moment of RC frame has increased after the application of all bracing systems.

Finally we can conclude that both X-bracing and Inv V-bracing system may be used to new design or retrofit for damage level earthquake, however, X-bracing system is more suitable to use.

The corner bracing configuration is better lateral displacement reduction arrangement from the other bay wise arrangement of steel braced reinforced concrete structures.

## **5.2. Recommendation**

From all results shown it is recommended that the RC building with concentric steel X-bracing and inverted V-bracing (chevron bracing) at the peripheral of the frame are best to improve the overall performance of buildings

From the easy erecting and practical use point of view its better using concentric X-bracing type of steel bracing system than inverted V-bracing system.

Using corner bracing configuration is better for maximum lateral displacement reduction from the other bay wise arrangement of equal amount steel bracing.

## **5.3. For further study the following points are need attention**

- 1) Experimental investigation of the design of connections between braced member and RC frame required.
- 2) Since this study is restricted to concentric bracing, to increase the ductility of the brace building in addition to the strength and stiffness more studies needed by using other bracing types like eccentric bracing system
- 3) Design of economical sections for bracing members need to be studied in detail.
- 4) In this study for the purpose of comparison only tube section of steel is selected for bracing member, it's possible for the future study uses other types of steel section.
- 5) For future study one can consider the behavior of irregular buildings for those types of bracing systems.
- 6) Other types of bracing arrangements, such as different bay wise arrangement, different story wise arrangement can be studied in future studies.

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