NON-LINEAR ANALYSIS OF STEEL BRACED MULTI-STORY REINFORCED CONCRETE FRAMES

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ABSTRACT

The motivation to develop seismic resistant structures stems from the substantial economic losses that can be caused by earthquake damage. One method of achieving seismic resiliency is to design buildings capable of resisting lateral earthquake loads, dissipating energy, and minimizing significant structural damage. In this study a regular ten, twelve and sixteen story with four bay buildings are designed for gravity loads and analyzed for seismic load cases. The bracings have been distributed at the middle and four corner periphery of the of structure over the height then nonlinear time history analysis was performed. Different kinds of steel bracings were used for example cross bracing, chevron (V bracing), V_{100} eccentric, V_{500} eccentric, V_{1000} eccentric, and V_{1500} eccentric. The numbers subscripted depict the eccentricity provision in chevron bracings at the tip in millimeters. For there is cross sectional and stiffness variation in resisting lateral loads between bracings, it was tried to equate the stiffness between cross bracings to the rest of bracings studied.The performance of the structure was improved up on the inclusion of steel bracings. Better than providing bracings at the corner periphery middle bracings displayed a significant lateral drift reduction. A reduction of maximum lateral storey displacement of about 58%,63%,49% was observed for the 10,12 and 16-storey middle cross braced buildings respectively, compared to the as-built one. The effect of including eccentricity in bracings interms of lateral displacement and interstory drift was discussed. Generally, middle bay braced buildings displayed reduced lateral displacement and interstory drift as we provide eccentricity. Overturning moment resistance increases with inclusion of steel bracing due to increase in the overall stiffness of the structure especially when using cross bracings. Using cross bracing has a better lateral displacement reduction capacity than the rest bracings studied in this paper.

In this study it was also demonstrated that performance criteria can be implemented not only for retrofitting new buildings but also for the design of new buildings through regorous iterations for a designer determined performance level. In this study it was desired to the immediated occupancy performance level.

ACKNOWLEDGMENT

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CHAPTER ONE

INTRODUCTION

1.1 Background of the study

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.

Remarkable earthquake happenstances can be put sequentianlly from 1961 in karakore of magnitude 6.5 killing 30 people down through to the most recent 2010 5.1 magnitude of eathquakes in jimma, hosanna and shenkola. (National Geophysical Data Center, 2017)

The active great rift valley make Ethiopia to two types of seismic hazard: Earthquake and Volcanic eruptions. Using data from one of the best known disaster databases shows that from 1900 to 2013 there were a total of ten earthquakes and eruptions – leading to a total of 93 deaths, 165 injured, 420 homeless and affecting 11,000 people. These are estimated to have an economic cost of more than US$7 million. (Herbert, 2013)

1.2 Statement of the problem

Unpredicted seismic ground motion caused massive damage to innumerable existing buildings through highly seismic areas of Ethiopia to various degrees. This damage to structures in turn causes loss of life, lots of causalities and huge cracks in the building. Strengthening of structures proves to be a better option outfitting to the economic considerations and immediate shelter problems rather than replacement of buildings. Since demolition and reconstruction will be uneconomical, time taking and difficult manuevership. Hence, retrofitting will cater the economic considerations and immediate shelter problems rather than replacement of buildings. Therefore,seismic strengthening of building structures is one of the most important aspects for mitigating seismic hazard especially in high earthquake seismic prone areas of Ethiopia.

Significant seismic excitations has been occuring in Ethiopia, especially in the southern part, and this shaking of the earth has caused immense damage on the building but this couldn't light the issue of seismic strengthening of the spared building stocks in private or governmental sect. Many buildings were designed based on old building codes. Therefore, they are characterized by insufficient reinforcement detailing (poor development length,lack of stirrups to enhance ductility level) ,poor quality materials, mass and stiffness irregularities, irregular structural configurations, change in fuction of a building and reinforcement corrrosion. Owing to their poor structural layout, the lack of a strength hierarchy engineered to control the inelastic response mechanism, deficient or discontinuous load paths,etc., existing substandard buildings may experience certain concentration of seismic deformation demands to few of their elements in the event of a strong earthquake. These deficiencies of the existing building will decrease the seismic performance and increase the vulnerability thereby fascillitate significant damage expected in the future ground excitation.

1.3 Research Objectives

1.3.1 General Objectives

- The main objective of this thesis work is to compare the dynamic response of a steel braced multi story RC frame with the bare RC frame by considering eccentric and concentric types of bracings and cross sections(Rectangular and I – Section) with regular story to bay configurations by considering seismic loading.

1.3.2 Specific Objectives

- To study the effect of increasing or decreasing eccentricity in eccentric types of bracings.
- To investigate the economical aspect of steel cross-section to be used for bracing
- To compare the behavior of the RC regular building when its corner is braced or middle bays braced
- Effectiveness of inclusion of steel bracing in multistory reinforced concrete buildings.

CHAPTER TWO

LITERATURE REVIEW

2.1 Steel Braced Reinforced Concrete Buildings

Many existing reinforced concrete buildings in Ethiopia were designed without considering the seismic criteria and ductile detailing hence this deficiency will highly affect the building during ground excitation. Inadequate transverse reinforcement in columns,beams and joints, bond slip of beam bottom reinforcement at the joint, poor confinement of the column has enhanced the non-ductile behavior of these buildings.

Designing reinforced concrete buildings by taking different load combinations in to account is a key for a high safety and its results can be very reliable. Material and mechanical properties, structural configuration, and soil-structure interactions are also an important aspects of structural design. After this design phase has settled to the ground, the anticipated performance of the building needs to be satisfied. Nevertheless, this ideal condition is not satisfied most of the time. Seismic Performance of structural building criteria postulated by FEMA 310 may not be satisfied due to various reasons such as alteration of building functions, changes of seismic load characteristics in the area,faulty in design and the current seismic performance code requirement is relatively higher. (FEMA 356, 2000)

In terms of seismic load characteristics, it is common to come across buildings which used to be meeting the seismic requirements and now their seismic performance are in question due to increase in the current seismic demand. (VANI PRASAD, 2014)

Commonly these buildings performance may degrade after it has been struck by earth quake hence their seismic performance will not meet the current code standards. Therefore, retrofitting the deficient existing building to enhance its perfomance capacity is a conduit to assure the strcuctural safety in the future ground excitation. There are several techniques that can be efficiently be chosen for this matter such as adding infill walls, Fibre reinforced polymer, steel bracing, base isolation etc.

But using steel bracing to strengthen the RC structures have the following advantages such as relatively economical, less structural dead weight, easy to manuvre and ease of customizing the strength and rigidity. (VANI PRASAD, 2014)

2.2 Strengthening with steel bracings

Global strengthening of buildings can be effectively achieved by using Steel bracing. As it will be discused below, some of the the advantages are the ability to put up openings, the minimal self weight to the structure and in the case of external steel systems minimum disruption to the function of the building and its occupants. (E.Thermou, 2009)

Steel braced frame is one of the structural systems used to resist earthquake loads in structures. Many existing reinforced concrete structures need retrofitting to overcome deficiencies and to resist seismic loads. The use of steel bracing systems for strengthening or retrofitting seismically inadequate reinforced concrete frames is a sustainable solution for enhancing earhquake resistance. Steel bracing is economical, easy to erect, occupies less space and has flexibility to design for meeeting the required strength and stiffness. (Philip, 2014)

2.3 Types of Steel Bracings

Depending on their geometric characteristics steel braced structures can be categorized as concentrically braced frames (CBF) or eccentrically braced frames (EBF). According to Bungale S. Taranath, in CBFs the axis of all members i.e., columns, beams, and braces intersect at a common point such that the member forces are axial. (BUNGALE S.TARANATH, 2005)

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

Eccentric bracings reduce the lateral stiffness of the system and improve the energy dissipantion 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 the frame. The vertical component of the bracing forces due to earthquake cause concentrated load on the beams at the point of connection of the eccentric bracings. (Viswanath, 2010)

2.3.1 Concentric Steel Bracing

Retrofitting concrete frames using Concentric bracing are the most widely used systems. The axial tension developed in their inclined members through the horizontal projection of the axial force have enhanced the lateral load resitance to the structure. According to JRC Science and Policy Report appropriate concentric bracing systems are those with:
Experiments were carried out on a reference and a strengthened one-third scale model of a two-storey RC frame. External steel shear walls improved the lateral load bearing capacity and stiffness of the reference model by 248 and 160 % respectively. Beyond a drift ratio of 1.0 %, diagonal elements of the wall started to buckle at the compressed ends, thus reducing the total base shear resistance. No damage was observed at the anchorages, which successfully transferred the load between the RC frame and the steel shear walls.

El-Sokkary and Galal (2009) analytically investigated the effectiveness of different rehabilitation patterns in upgrading the seismic performance of existing nominally non ductile RC frame structures. Nine different accelerogram records has been used to analyze low to high rise structures. Three ground motion records represent low frequency, the other three represent medium and high frequency each. RC shear wall, steel bracing, FRP (Fiber Reinforced Polymer) strips on the infills and jacketing of columns and beams with FRP sheets has been analyzed. The X bracings were introduced in one bay along the full height of the frames. As the maximum peak ground acceleration (PGA) resisted by the frames was increased on average 1.8 times for the five-storey frame and 1.2 times for the 15-storey frame. A reduction of maximum storey drifts of about 20 % was observed for the 15-storey braced building, compared to the as-built one. The dissipated energy was also increased, for both low- and high-rise buildings, particularly for the frames with soft masonry infills. The numerical analyses confirmed also that strengthening all frames of a building will provide higher increase (not proportional) of the shear resistance and energy-dissipation capacity, compared to strengthening half of the frames, and similar decrease in deformation demand.

2.3.2 Eccentric Steel Bracing

Eccentrically braced frames are an efficient technique for enhancing the seismic resistance of existing frame buildings because, in addition to strength and stiffness, they provide ductility. Forces are transferred to the brace members through bending and shear forces developed in the ductile steel link. The link is designed to yield and dissipate energy, while preventing buckling of the brace members. Different patterns are used: K, Y and inverted Y bracing. One further advantage of eccentric braces is the possibility to select the dimensions of the links and braces almost independently of each other, thus allowing modulating stiffness and strength as required. In fact, the cross-section of the link determines the storey shear strength, whereas the link length and the brace cross-section quantify the stiffness of the bracing system. Nevertheless, the use of eccentric bracing in the rehabilitation of RC structures lags behind concentric bracing applications due to the lack of sufficient background on the design and modelling of the combined concrete and steel system.

2.4 Methods of bracings

Shear walls have been commonly used in Reinforced Concrete buildings to enhance a global lateral force resistance where as steel bracings only for steel structures. According to impact international journals of research reported in the last two decades effective usage of steel bracings to the RC buildings. There are two methods of bracing for the structure:

2.4.1 Exterior bracing

Exterior bracing system where the existing buildings are strengthened by attaching a local or global steel bracing system to the peripheral frames. In this type of bracing systems making a good bond between the steel and concrete is big problem.

2.4.2 Interior bracing

In the interior bracing method, the buildings are retrofitted by incorporating a bracing system inside the individual units or panels of the RC frames. Direct application to the joint of the frame is possible.

2.5 Configuration of Bracings

The steel braces are usually placed in vertically aligned spans. This system allows to obtain 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.

Alternative configurations of bracing systems may be used in selected bays of a reinforced concrete frame to provide a significant increase in horrizontal capacity of the structure. Concentric steel bracing systems have been investigated for the rehabilitation of non-ductile buildings by many researchers (Masri, et al., 1996) The use of eccentric bracings in the rehabilitation of reinforced concrete structures gains ground.

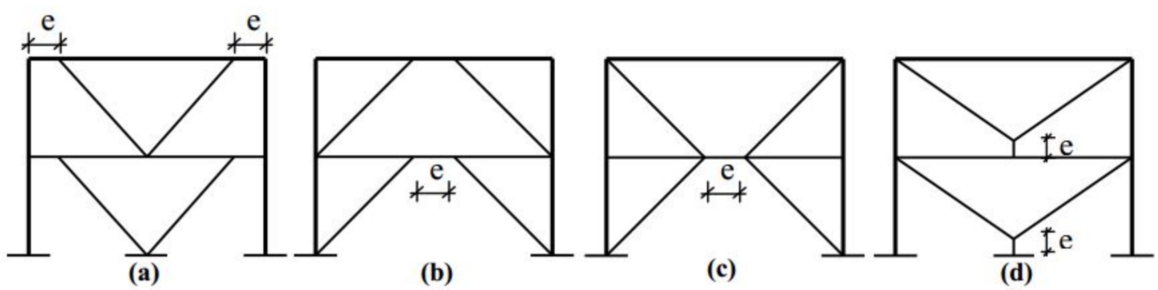


Figure 2.1 Configuration of Eccentric steel bracing (a)-V (b) –K (c) –X (d) -Y (Georgia, et al., 2009)

2.6 Beam –to –column connection

Steel moment frames have traditionally been designed with rigid full-strength connections, usually of fully welded or hybrid welded/bolted configuration. Typical design provisions ensured that connections are provided with sufficient overstrength such that dissipative zones occur mainly in the beams. However, the reliability of these commonly used forms of full-strength beam to-column connection has come under question following poor performance in large events in the mid 1990s, particularly in the Northridge earthquake of 1994 (Bertero et al, 1994) and the Hyogo-ken Nanbu (Kobe) earthquake of 1995 (EERI, 1995). The extent and repetitive nature of damage observed in several types of welded and hybrid connections have directed considerable research effort not only to repair methods for existing structures but also to alternative connection configurations to be incorporated in new designs.

Laboratory tests confirmed that connections designed and manufactured strictly to code requirements and conventional shop practice failed to provide the necessary levels of ductility. Observed damage was attributed to several factors including defects associated with weld and steel materials, welding procedures, stress concentration, high rotational demands and scale effects, as well as the possible influence of strain levels and rates. In addition to the concerted effort dedicated to improving seismic design regulations for new construction, several proposals have been forwarded for the upgrading of existing connections (FEMA, 1995, 1997, 2000; PEER, 2000). This may be carried out by strengthening of the connection through haunches, cover or side plates, or other means.

Alternatively, it can be achieved by weakening of the beam by trimming the flanges (i.e. reduced beam section 'RBS' or 'dog-bone' connections), perforating the flanges, or by reducing stress concentrations through slots in beam webs, enlarged access holes, etc. In general, the design can be based on either prequalified connections or on prototype tests. (Eurocode, 2009)

2.7 New feature about ETABS 2016

An integrated design analysis and design software ETABS 2016 finite element has incorporated Pushover analysis, Response spectrum analysis, linear as well as nonlinear time history analysis-which analyzes the timely response of a structure for an imposed seismic ground excitations. Response spectrum analysis gives result for a peak ground acceleration values then combines the maximum response. However, THA gives a structural response result for a desired time step.

This new software is capable doing matching the 1940 Elcentro's earthquake acceleration result to any response spectrum (RSP) curve for a specified frequency and damping ratio. The Elcentro's earth quake has peak ground acceleration of 0.1g. Though it is classified as low excitation earthquake it had a significant damage on the structures. Therefore, the user can define the response spectrum data in the RSP function dialogue then program can automatically generate the time history function by matching to the program default or user defined target RSP functions. The software is able to analyze the geometric nonlinear behavior of the space frames under static or dynamic loadings, taking into account both geometric nonlinearity and material inelasticity.

2.8 Time History Records

An accelerograph can be referred to as a strong motion seismograph, or simply an earthquake accelerometer. Accelerographs are useful for when the earthquake ground motion is so strong that it causes the more sensitive seismometers to go off-scale. There is an entire science of strong ground motion that is dedicated to placing Accelerographs in the vicinity of major faults. The type of information gathered (such as rupture velocity) would not be possible with the standard seismometers. The best known example is the Park field Experiment which involved a massive set of strong motion instrumentation. They record peak ground acceleration (PGA), velocity (PGV), ground displacement (PGD) and spectral intensity (SI).

Within the accelerograph, there is an arrangement of 3 accelerometer sensing heads. These are usually micro-machined (MEMS) chips that are sensitive to one direction. Thus constructed, the accelerometer can measure full motion of the device in three dimensions.

Unlike the continually recording seismometer, accelerometers nearly always work in a triggered mode. That means a level of acceleration must be set which starts the recording process. This makes maintenance much more difficult without a direct Internet connection (or some other means of communication). Many trips have been made to accelerometers after a large earthquake, only to find that the memory was filled with extraneous noise, or the instrument was malfunctioning.

Accelerometers are used to monitor structures for earthquake response. Sometimes, with the data, a response spectrum is computed. Other analysis is used to improve building design, or to help locate important structures in safer areas.

2.9 Classification of subsoil conditions according to the new Ethiopian Building code EBCS EN 1998-1-1:2013

Depending on the importance class of the structure and the particular conditions of the project, ground investigations and/or geological studies should be performed to determine the seismic action (EBCS EN 1998-1-1:2013). The construction site and the nature of the supporting ground should normally be free from risks of ground rupture, slope instability and permanent settlements caused by liquefaction or densification in the event of an earthquake.

Table 2.1 Classification of subsoil conditions according to the new Ethiopian Building code EBCS EN 1998-1-1:2013

Ground type	Description of stratigraphic profile	Parameters		
		$V_{s,30}$(m/s)	N_{SPT} (blows/30cm)	Cu (kpa)
A	Rock or other rock-like geological formation, including at most 5 m of weaker material at the surface.	>800	-	-
B	Deposits of very dense sand, gravel, or very stiff clay, at least several tens of meters in thickness, characterized by a gradual increase of mechanical properties with depth.	360-800	>50	>250
C	Deep deposits of dense or medium-dense sand, gravel or stiff clay with thickness from several tens to many hundreds of meters.	180-360	15-50	70-250
D	Deposits of loose-to-medium cohesionless soil (with or without some soft cohesive layers), or of predominantly soft-to-firm cohesive soil.	<180	<15	<70
E	A soil profile consisting of a surface alluvium layer with vs values of type C or D and thickness varying between about 5 m and 20 m, underlain by stiffer material with vs > 800 m/s.			
S1	Deposits consisting, or containing a layer at least 10 m thick, of soft clays/silts with a high plasticity index (PI > 40) and high water content	<100 (indicative)	-	10-20
S2	Deposits of liquefiable soils, of sensitive clays, or any other soil profile not included in types A – E or S1			

2.10 Non-Linear Static (Push over) Analysis

Pushover analysis is a nonlinear static analysis carried out under conditions of constant gravity loads and monotonically increasing horizontal loads. It may be applied to verify the structural performance of newly designed and of existing buildings either to verify or revise the over strength ratio values ,to estimate the expcted plastic mechanisms and the distribuion of damage and or to assess the structural performannce of existing or retrofitted buildings.

2.11 Nonlinear static analysis for different steel bracings

The use of nonlinear static analysis nominally known as push over analysis came in 1970's but the potential of pushover analysis has been recognized for the last 10 to 15 years. Our code suggests the use of this procedure for the performance analysis of existing or new model. Furthermore over strength ratio and drift capacity of existing structure and the seismic demand for the structure subjected to selected earthquake. This procedure can be used for checking the adequacy of new structural design as well. Pushover is defined as an analysis wearing a mathematical model directly incorporating the normal load deformation characteristics of individual components and elements of the building shall be subjected to monotonically increasing lateral loads representing inertia forces in an earthquake until a target displacement is excised.

Nonlinear static analysis can be used for many purposes. 1. To perform an initial P-delta or large-displacement analysis to get the stiffness used for subsequent superpose able linear analyses
2. To perform staged (incremental, segmental) construction analysis, including material time-dependent effects like aging, creep and shrinkage. 3. To change the Mass Source to be used for subsequent analyses 4. To perform pushover analysis etc. Pushover analysis assess the structural performance by estimating the force and deformation capacity and seismic demand using a nonlinear static analysis. Federal Emergency Agency (FEMA) and Applied Technical Council (ATC) are the two agencies which formulated and suggested pushover analysis under rehabilitation programs and guidelines. These included documents are FEMA 356, FEMA 273 and ATC 40.

The primary purpose of FEMA-356 document is to provide technically sound and nationally acceptable guidelines for seismic rehabilitation of buildings. The guidelines for the seismic rehabilitation of the buildings are intended to serve as a ready tool for design professional for carrying out the design and analysis of the buildings, a reference document for the building regulatory officials and a foundation for the future development and implementation of the building code provisions and standards.

Seismic evaluation and retrofit of concrete buildings commonly reffered to as ATC-40 was developed by the Applied Technology Council(ATC) with funding from California safety comission. Though the procedures recommend in this document are for concrete buildings, tey are applicable to most building types.

2.12 Types of pusover analysis

There are two nonlinear static analysis procedures available, one termed as the Displacement Coefficient Method (DCM), documented in FEMA-356 and other the Capacity Spectrum Method (CSM) documented in ATC-40. Both methods depend on lateral load-deformation variation obtained by nonlinear static analysis under the gravity loading and idealized loading due to the seismic action. This analysis is called Pushover Analysis.

2.12.1 Capacity Spectrum Method

Capacity Spectrum Method is a nonlinear static analysis procedure which provides a graphical representation of the expected seismic performance of the structure by intersecting the structure's capacity spectrum with the response spectrum (demand spectrum) of the earthquake. The intersection point is called as the performance point, and the displacement coordinate d_D of the performance point is the estimated displacement demand on the structure for the specified level of seismic hazard.

2.12.2 Displacement Coefficient Method

Displacement Coefficient Method is a nonlinear static analysis procedure which provides a numerical process for estimating the displacement demand on the structure by using a bilinear representation of the capacity curve and a series of modification factors or coefficients to calculate the target displacement. The point on the capacity curve at the target displacement is the equivalent of the performance point in the capacity spectrum method.

2.13 Pushover Terminological definitions

2.13.1 Capacity

It is defined as the expected ultimate strength (in flexure, shear and axial loading) of the structural components excluding the reduction factors commonly used in the design of concrete members. The capacity generally refers to the strength at the yield point of the element or structure's capacity curve. For deformation controlled component's, capacity beyond the elastic limit generally includes the effect of strain hardening.

2.13.2 Capacity curve

The plot between base shear and roof displacement is referred to as capacity curve. Also mentioned as pushover curve.

2.13.3 Capacity spectrum

The capacity curve transformed from base shear v/s roof displacement (V v/s D) to spectral acceleration v/s spectral displacement (S_a v/s S_d) is referred as capacity spectrum.

2.13.4 Demand

Demand is represented by an estimation of the displacement or deformation that the structure is expected to undergo. This is in contrast to conventional linear elastic analysis procedures in which demand is represented by prescribed lateral forces applied to the structure.

2.13.5 Demand Spectrum

It is plot between spectral acceleration versus time period. It represents the earthquake ground motion in capacity spectrum method.

2.13.6 Plastic hinge

Location of inelastic action of the structural member is called a plastic hinge. Plastic hinges are formed near the ends of beams or columns due to the maximum moments caused by earthquake. For this reason most ductility requirements apply to section near the junctions.

2.14 Performance levels

According to FEMA definition Building performance is a combination of the performance of both structural and nonstructural components. Performance Levels are discrete damage states selected from among the infinite spectrum of possible damage states that buildings could experience during an earthquake. The particular damage states identified as target Building.

Performance Levels in FEMA standard have been selected because they have readily identifiable consequences associated with the post-earthquake disposition of the building that are meaningful to the building community. These include the ability to resume normal functions within the building, the advisability of post-earthquake occupancy, and the risk to life safety.

The performance levels are based on force versus displacement curve. The graph represents from A to B range we can expect linear and elastic material behavior. From B to C range we can expect there exists some plastic mechanism formation in the structure. In this level performance level FEMA divides in to three discrete levels of performance according to plastic hinge formation and level of damage in the structural components.

i) Immediate Occupancy (IO)

Immediate Occupancy, shall be defined as the post-earthquake damage state that remains safe to occupy, essentially retains the pre-earthquake design strength and stiffness of the structure. Level of risk is very low. Some minor repairs may be necessary. However, the building is fully habitable after a design earthquake.

ii) Life Safety (LS)

Life Safety, means the post-earthquake damage state in which significant damage to the structure has occurred, but some margin against either partial or total structural collapse remains. Some structural elements and components are severely damaged, but this has not resulted in large falling debris hazards, either within or outside the building. Injuries may occur during the earthquake; however, the overall risk of life-threatening injury as a result of structural damage is expected to be low. It should be possible to repair the structure; however, for economic reasons this may not be practical. While the damaged structure is not an imminent collapse risk, it would be prudent to implement structural repairs or install temporary bracing prior to re-occupancy.

iii) Collapse Prevention (CP)

Collapse Prevention, means the post-earthquake damage state in which the building is on the verge of partial or total collapse. Substantial damage to the structure has occurred, potentially including significant degradation in the stiffness and strength of the lateral-force resisting system, large permanent lateral deformation of the structure, and—to a more limited extent—degradation in vertical-load-carrying capacity. However, all significant components of the gravity load-resisting system must continue to carry their gravity load demands. Significant risk of injury due to falling hazards from structural debris may exist. The structure may not be technically practical to repair and is not safe for re-occupancy, as aftershock activity could induce collapse. (FEMA 356, 2000)

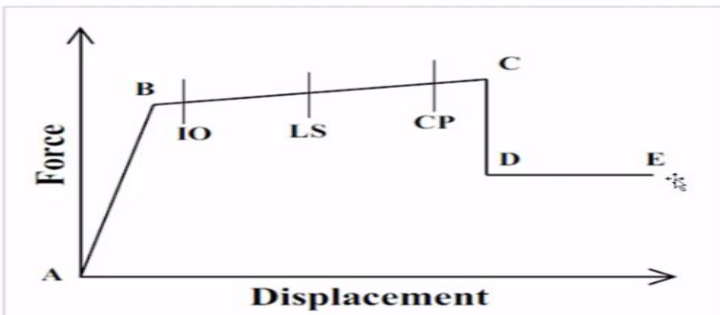


Figure 2.2 Force versus displacement curve

2.15 Non –Linear Time History (Dynamic) Analysis

The time-dependent rersponse of the structure may be obtained through direct numerical integration off its differential equations of motion, using the accelerograms to reprsent the ground motions.

2.16 Conceptual Seismic Resistant Design principles according to revised Ethiopian Code/ Eurocode8

2.16.1 Structural simplicity

According to Euro code 8: EN1998.1.2004, structural simplicity are characterized by the existence of clear and direct paths for the transmission of the seismic forces. Modelling, analysis, dimensioning, detailing and construction of simple structures are subject to much less uncertainty and thus the prediction of its seismic behavior is much more reliable.

2.16.2 Uniformity, symmetry and redundancy

The structural modelling and analysis procedure is based on the evenly distribution of structural elements, which includes no setbacks in plan or elevation, equal column length, homogeneous material, symmetrical mass and stiffness etc. evenly distribution of structural elements allows short and direct transmission of the inertial forces created in the distributed masses of the building. The code provides for the seismic joints by subdividing the entire building into dynamically independent units for the realization of uniformity provided that pounding effect is well designed. In this paper all modelling is done for regular and symmetrical buildings only.

2.16.3 Bi-directional resistance and stiffness

According to Euro code 8: EN1998.1.2004, horizontal seismic motion is a bi-directional phenomenon and thus the building structure shall be able to resist horizontal actions in any direction. For this phenomenon the code provides the structure to be arranged in an orthogonal in plan structural pattern, ensuring similar resistance and stiffness characteristics in both main directions.

2.16.4 Torsional resistance and stiffness

Torsional resistance is another very important aspect besides lateral resistance and stiffness in order to limit the development of torsional motions which tend to stress the different structural elements in a non-uniform way.

2.16.5 Diaphragmatic behavior at story level

In buildings, floors (including the roof) play a very important role in the overall seismic behavior of the structure. They act as a horizontal diaphragms that collect and transmit the inertia forces to the vertical structural systems and ensure that those systems act together in resisting the horizontal seismic action. The action of floors as vertical structural systems, or where systems with different horizontal deformability characteristics are used together (e.g. in dual or mixed systems) *(Euro code 8, 2004).*

2.16.6 Adequate foundation

With regard to the seismic action, the design and construction of the foundations and of the connection to the superstructure shall ensure that the whole building is subjected to a uniform seismic excitation.

2.17 Importance factor

According to Euro code 8: EN1998.1.2004, Buildings are classified in four importance classes, depending on the consequences of collapse for human life, on their importance for public safety and civil protection in the immediate post-earthquake period, and on the social and economic consequences of collapse.

Table 2.2 Importance factor

Importance class	Buildings
I	Buildings of minor importance for public safety, e.g. agricultural buildings, etc.
II	Ordinary buildings, belonging in the other categories
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.
IV	Buildings whose integrity during earthquakes is of vital importance for civil protection, e.g. hospitals, fire stations, power plants, etc.

The value of γ_1 for importance class II shall be, by definition, equal to 1.0. Thus, in this paper all structural modelling is for importance class of 1.0.

2.18 Structural types and behavior factors

According to EBCS EN 1998-1-1:2013 Concrete buildings are classified into one of the following structural types according to their behavior under horizontal seismic actions:

a) Frame system
b) Dual system (frame or wall equivalent);
c) Ductile wall system (coupled or uncoupled);
d) System of large lightly reinforced walls;
e) Inverted pendulum system;

2.18.1 Behavior factors for horizontal seismic actions

According to the new Ethiopian building code the upper limit value of the behavior factor, to account for energy dissipation capacity, shall be derived for each design direction as follows:

$$q = qo * kw \geq 1.5$$

Where:

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

$kw =$ is the factor reflecting the prevailing failure mode in structural systems with walls

2.18.2 Basic values of the behavior factor, q_0 for systems regular in elevation (EBCS EN 1998-1-1:2013)

Table 2.3 Basic values of behavior factor for regular frame systems

Structural type	DCM	DCH
Frame system, dual system, coupled wall system	3.0αu/α1	4.5αu/α1
Uncoupled wall system	3.0	4.0αu/α1
Torsionally flexible system	2.0	3.0
Inverted pendulum system	1.5	2.0

Where:
α1 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;
αu 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 αu may be obtained from a nonlinear static (pushover) global analysis
NB. For multistory, multi-bay frames or frame-equivalent dual structures the new Ethiopian code provides: αu/α1=1.3.

2.19 Control of strong ground motion

The response of reinforced concrete structures to strong ground motion can be controlled through judicious balancing of three ratios: (Bozorgnia, 2006)

- ✓ Ratio of mass to stiffness
- ✓ Ratio of weight to strength
- ✓ Ratio of lateral displacement to height

The stated ratios defy precise definitions. In earthquake-resistant design that is almost universally the case, start with the definition of the ground motion. Earthquake resistant design of reinforced concrete is closer to art than to science. One must expect the unexpected. The three ratios cited are simply vehicles for understanding and projecting experience.

The most convenient definition of the mass-to-stiffness ratio is the translational vibration period of the notional structure corresponding to its lowest natural frequency. That is the quantity we will refer to whenever we invoke “period” unless we specify that it refers to a higher mode. For preliminary proportioning, the period is the most important characteristic of a structure threatened by strong ground motion.

The ratio of weight to strength is usually expressed in terms of the "base shear strength" of the structure, the maximum base shear strength calculated for an arbitrary distribution of lateral forces applied at floor levels. The ratio of tributary weight to strength (usually called the "base shear strength coefficient") is an approximate measure of how strong a structural system is in reference to its tributary load. It is not an important property of the structure as long as it is above a threshold value related to the ground motion demand.

The third ratio is more properly called "drift capacity" or the "limiting-drift ratio." It can be defined as the capacity for the "mean drift ratio," the roof drift divided by the height to roof above base, or the capacity for the "story drift ratio," the lateral displacement in one story divided by the height of that story. It is a measure of the ability of the structure to distort without losing its integrity. For a given ground-motion demand, the period and the drift capacity are the two critical considerations for proper proportioning and detailing of a structure. The base shear strength coefficient is not likely to be important unless the engineer has made unreasonable choices (such as assigning an entire parking structure to two lateral-force resisting frames) or unless the peak ground velocity turns out to be high (more than one m/sec).

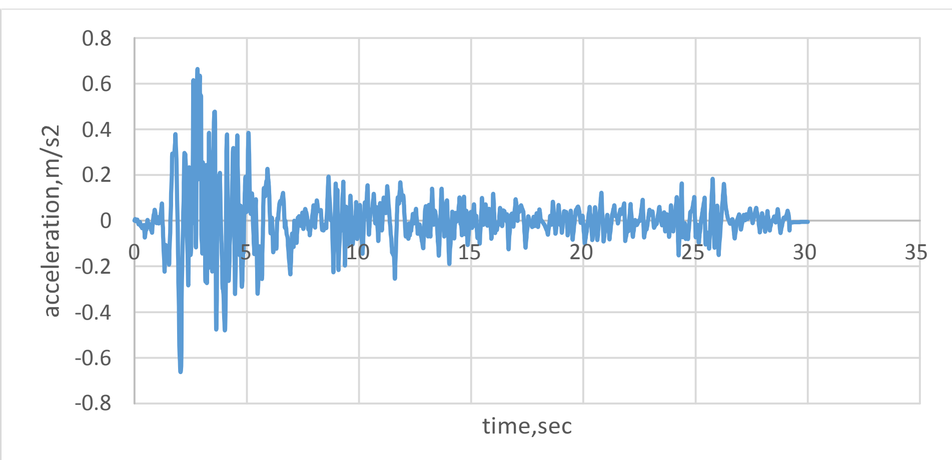


Figure 2.3 Ground acceleration versus time

2.20 Nonlinear behavior

An important decision in a structural analysis is to assume whether the relationship between forces and displacements is linear or nonlinear. Linear analysis for static and dynamic loads has been used in structural design for decades. Nonlinear analysis methods are widely used, because emerging performance-based guidelines require representation of nonlinear behavior. There are two major sources of nonlinear behavior. The first is a nonlinear relationship between force and deformation resulting from material behavior such as ductile yielding, stiffness and strength degradation or brittle fracture. The second type of nonlinear behavior is caused by the inclusion of large displacements in the compatibility and equilibrium relationships. (Bozorgnia, 2006)

In this particular scenario the second type of nonlinearity (geometric nonlinearity or p-delta) analysis is not entertained. Hence, material nonlinearity and inelasticity is considered throughout the analysis.

CHAPTER THREE

STRUCTURAL MODELLING AND ANALYSIS

3.1 Introduction

Reinforced concrete (RC) frames are considered as one of the most efficient lateral load resisting systems due to their ductility capacity, which increases the energy dissipation capacity of the structure. In order to achieve that required level of ductility, special attention should be considered for the detailing of reinforcement of beams, columns, and joints of the moment resisting frames. Most of the existing buildings that were designed according to pre-1970 design codes don't possess this ductility level. Consequently, this makes them susceptible to the hazard of partial damage or total collapse in the event of strong ground motion.

Pre-1970 design codes adopted a strength- based philosophy, and hence once the ultimate strength of the structure is reached, a non-ductile deterioration follows, which reduces the energy dissipated by the structure and results in a brittle failure. Therefore, structures designed according to old codes need to be strengthened in order to meet the requirements of the newly adopted performance-based design approach. This approach is expected to decrease the probability of brittle failure of the structures, and increase the energy dissipation capacity when subjected to the design ground motions.

This chapter discusses the modeling assumptions and procedures used to investigate the behavior of the studied frames, including the different models that were used to represent different structural elements, and the properties of each model.

3.2 Structural layout

Three regular building models that have been designed for gravity loading using previous Building code are selected for this study. The buildings are of heights ten, twelve and sixteen stories that represent medium and high-rise buildings, respectively. The three buildings have the same floor plan that consists of four symmetrical bays in both directions, where the bay width is 5m. The floor height is 3m and the total heights of the three buildings are 30m, 36m and 48m, respectively. The plan and elevations of the three buildings, the concrete dimensions for the beams and column sections and the steel reinforcement ratios varied along the height of the frames according to the change of axial load acting on each group of columns, while the beam dimensions and steel reinforcement were assumed to be the same for the entire frame.

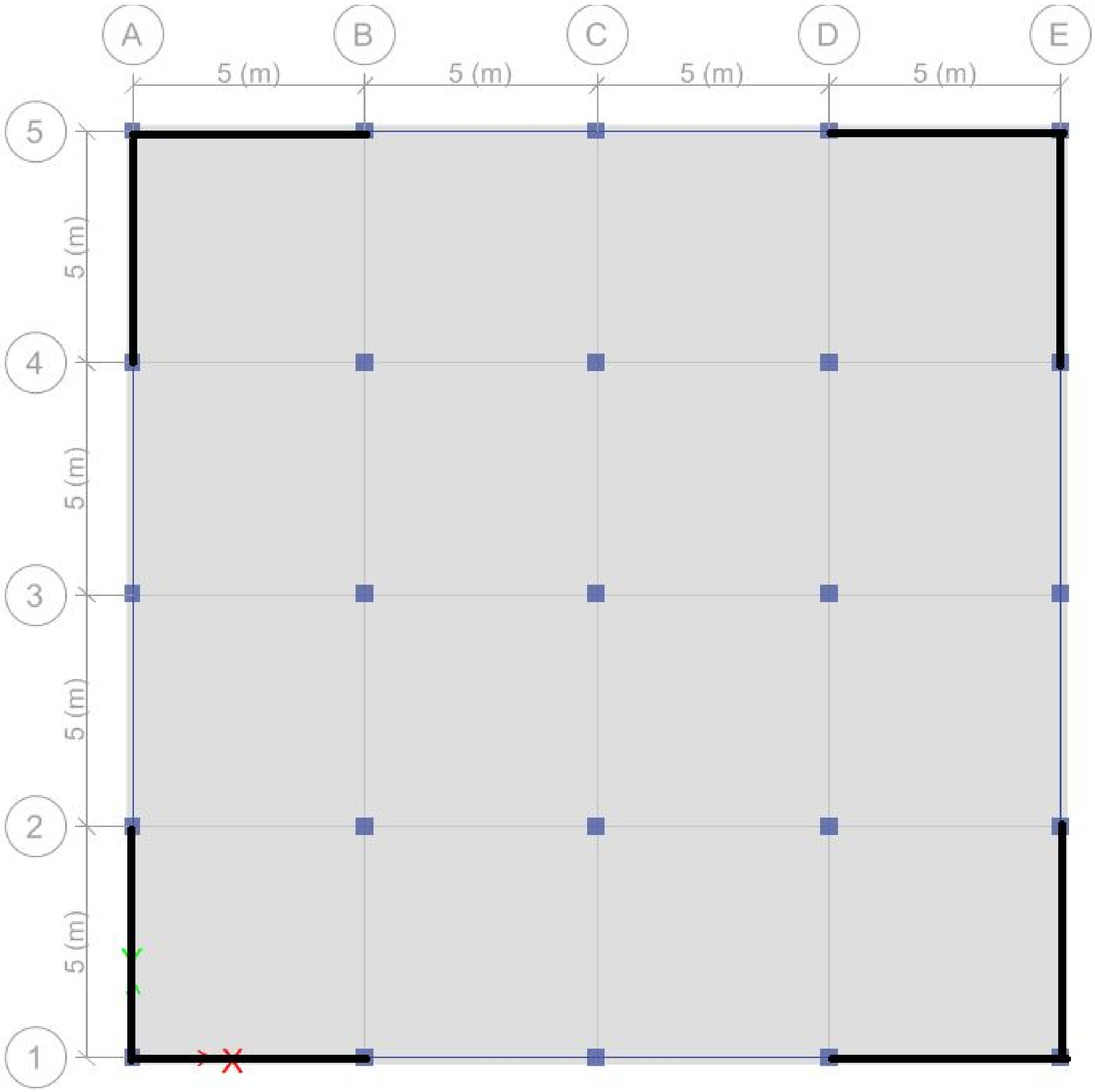


Figure 3.1Corner Braced 4x4 bay configuration in the X and Y direction

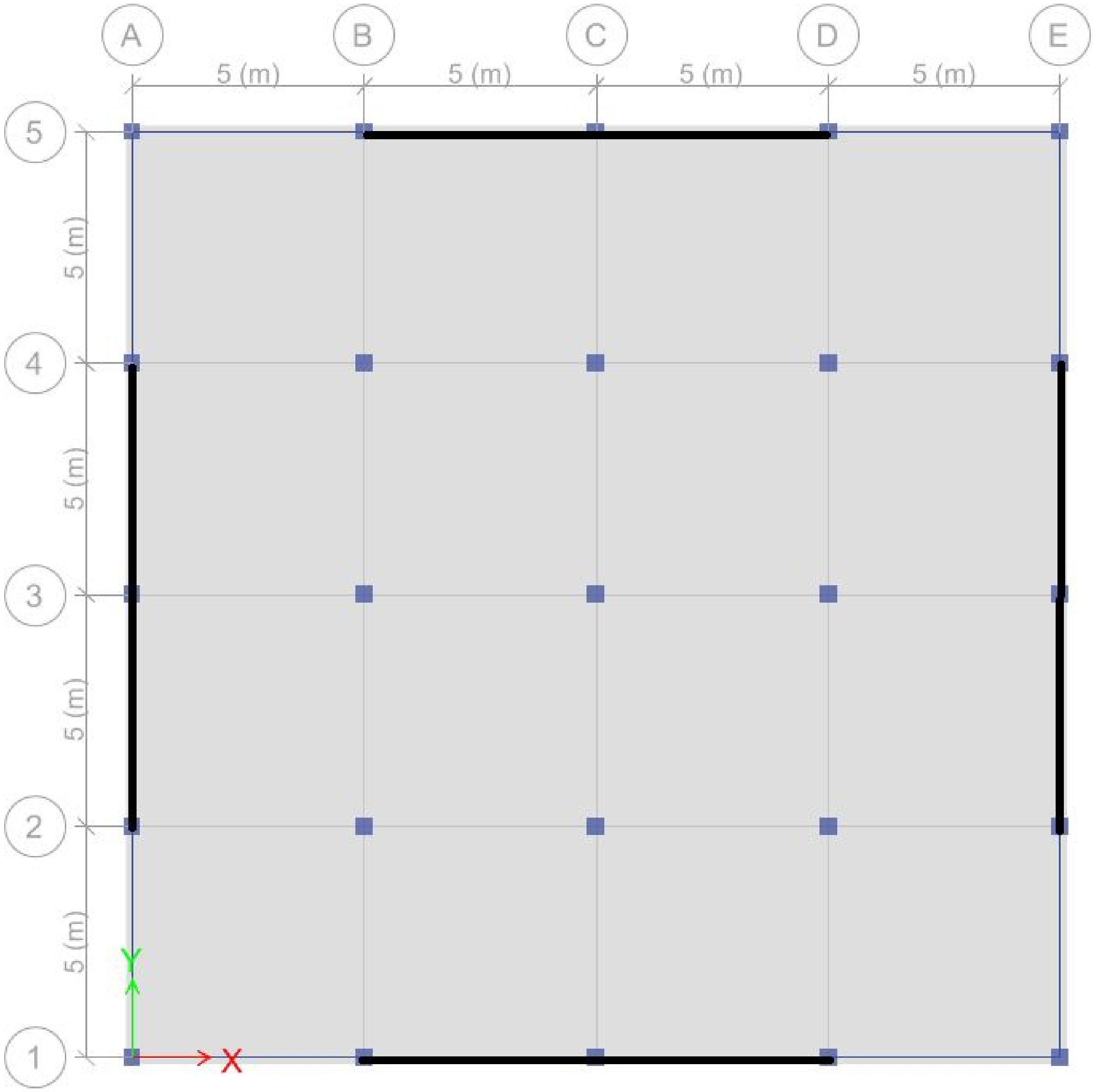


Figure 3.2Middel Braced 4x4 bay configuration in the X and Y direction

Ten story models

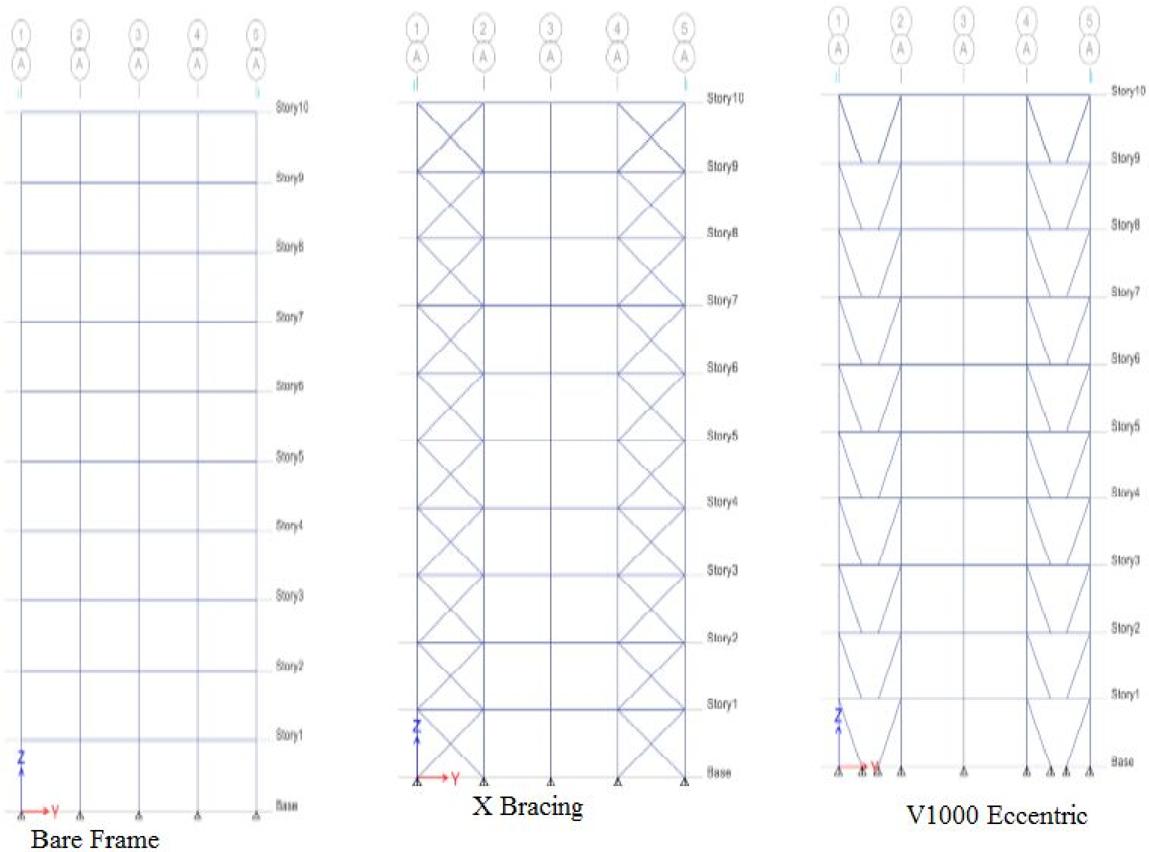


Figure 3.3Ten story model bare frame, X Bracing, and Chevron bracing with 1000mm eccentricity at the tip

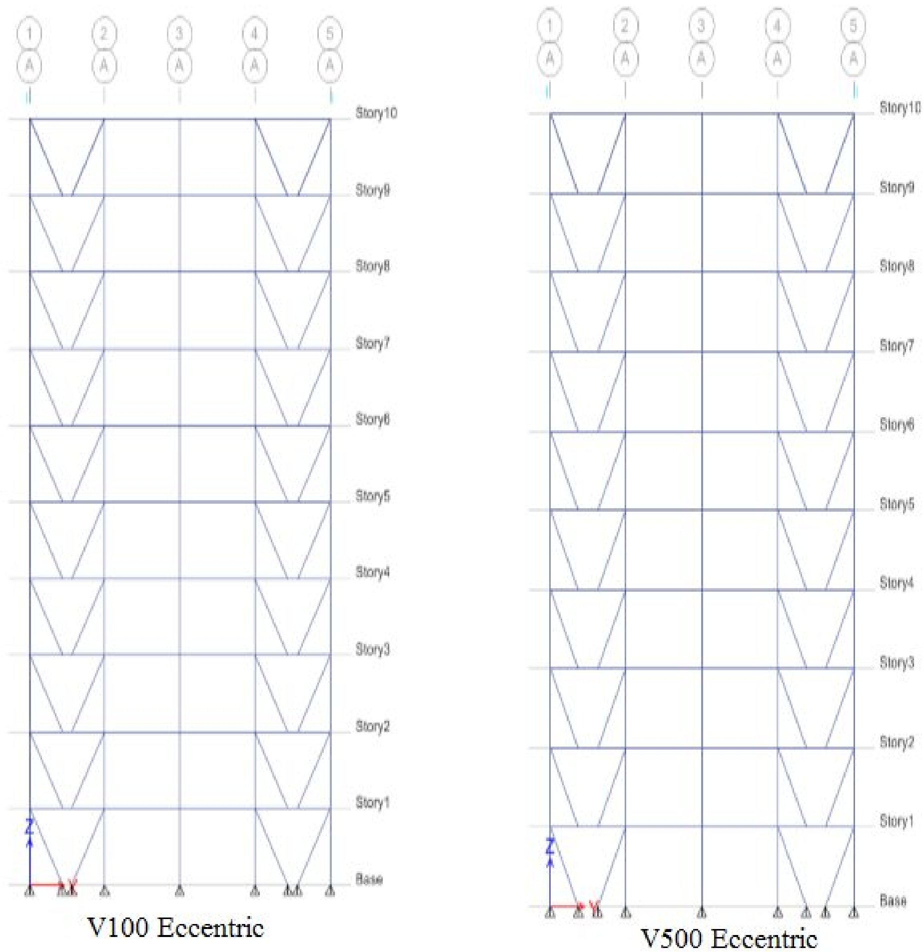


Figure 3.4Ten story model off Chevron bracing with 100mm and 500mm eccentricity at the tip

Twelve story models

Corner Braced

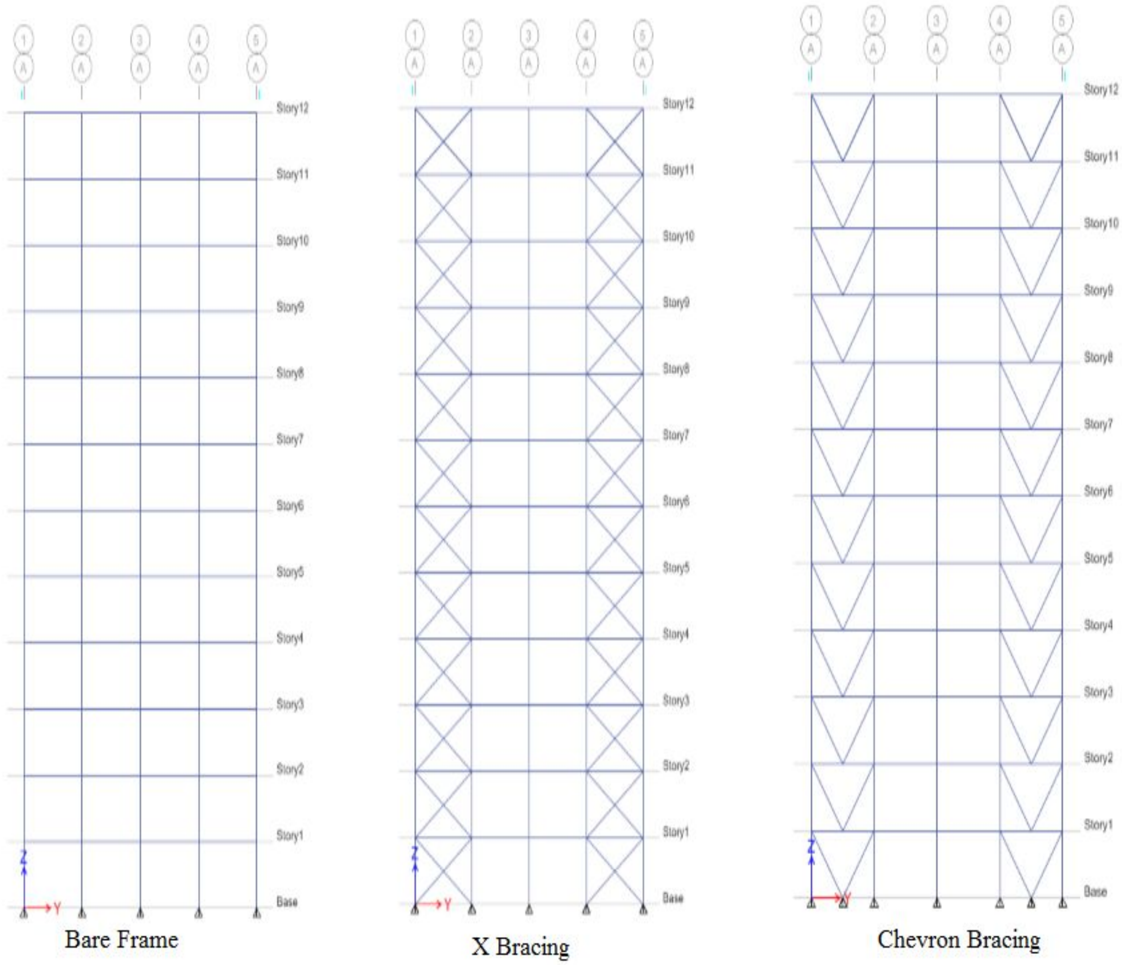


Figure 3.5 concentrically braced frames braced at the corner of the structure

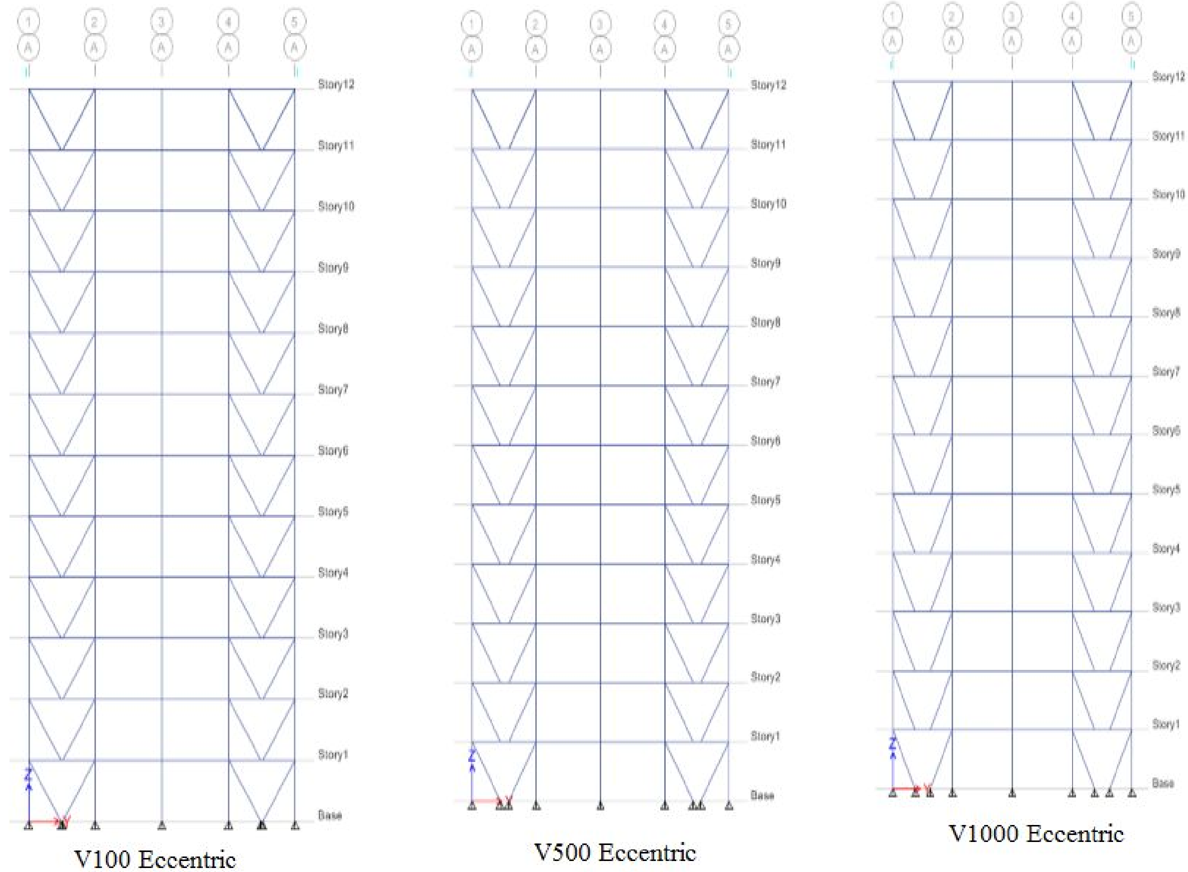


Figure 3.6 Eccentrically braced frames braced at the corner of the structure

Middle Braced

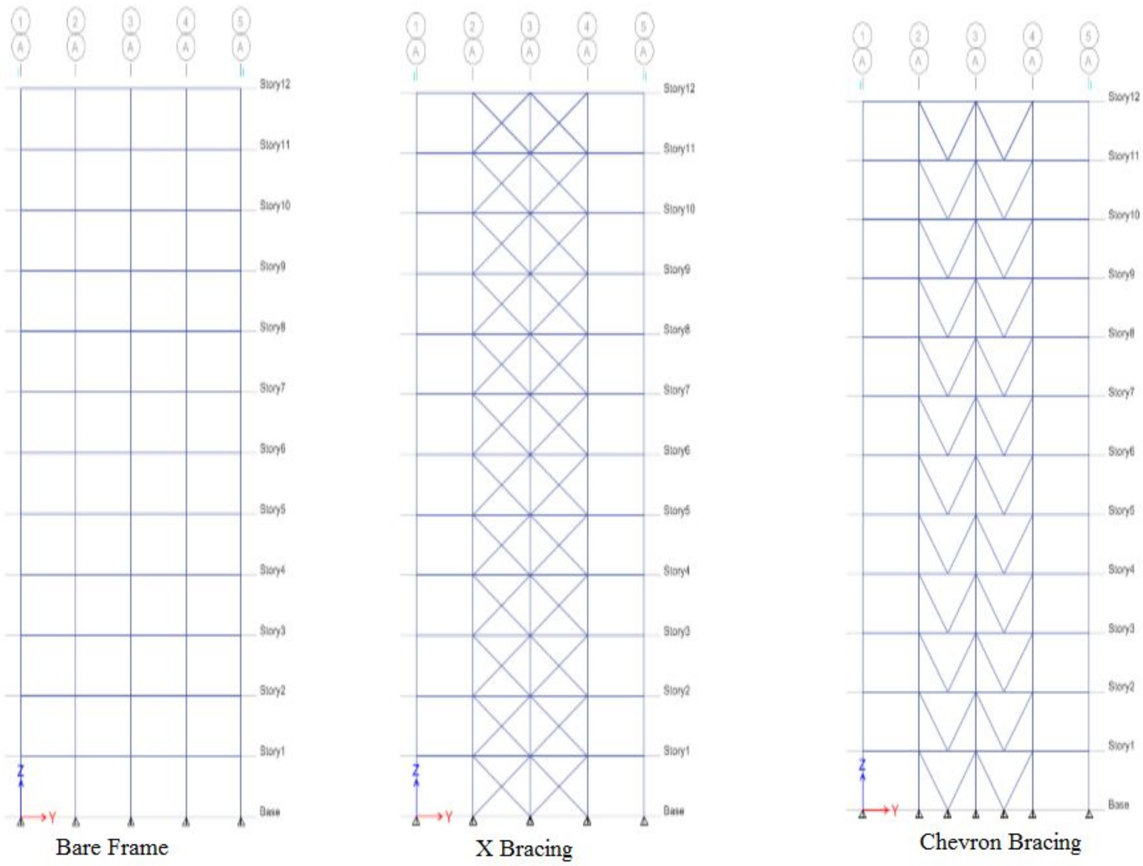


Figure 3.7 Concentrically braced frames braced at the middle of the structure

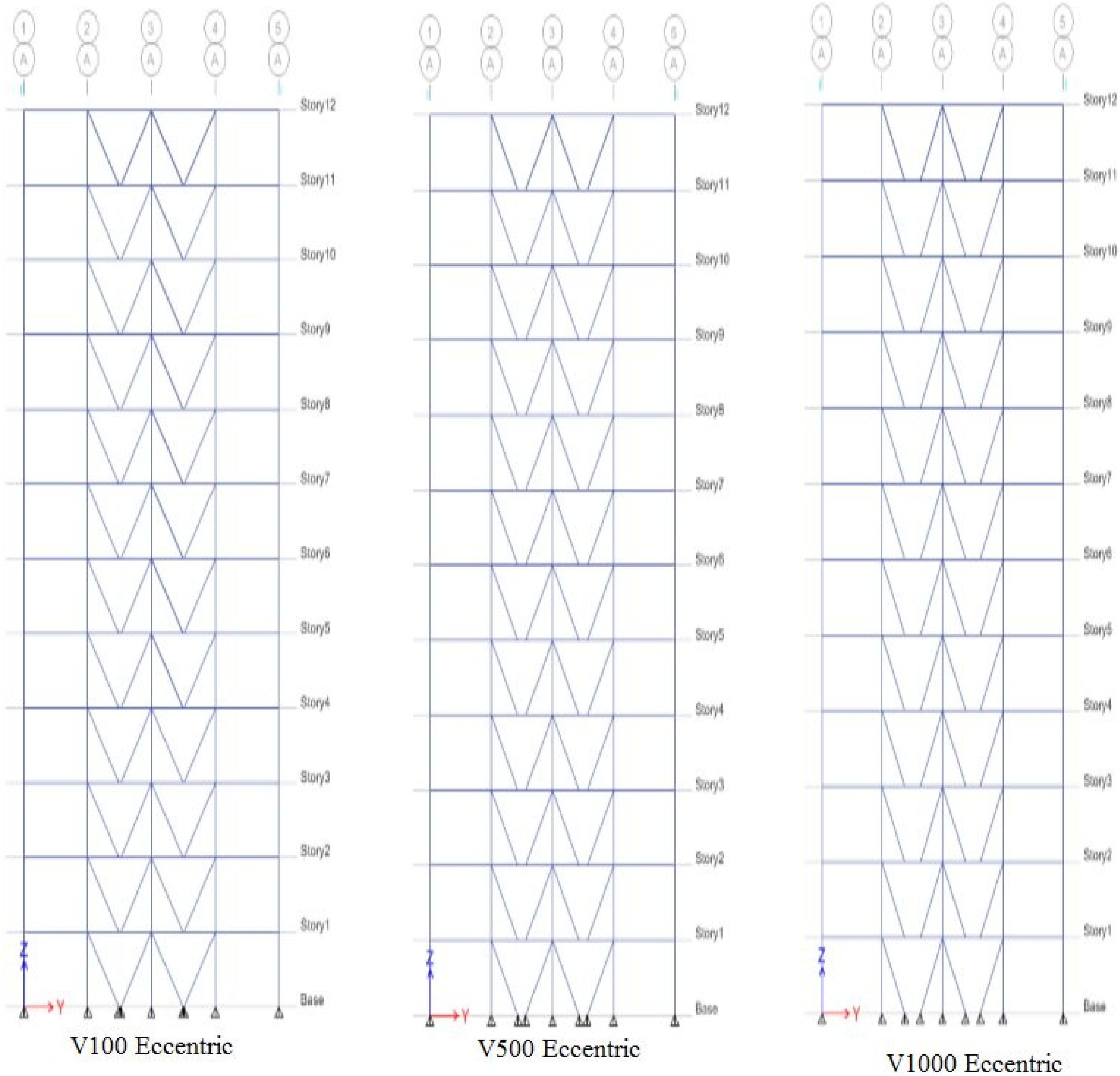


Figure 3.8 Eccentrically braced frames braced at the middle of the structure

***NB. The same configuration and parameter apply for the sixteen story frame.*

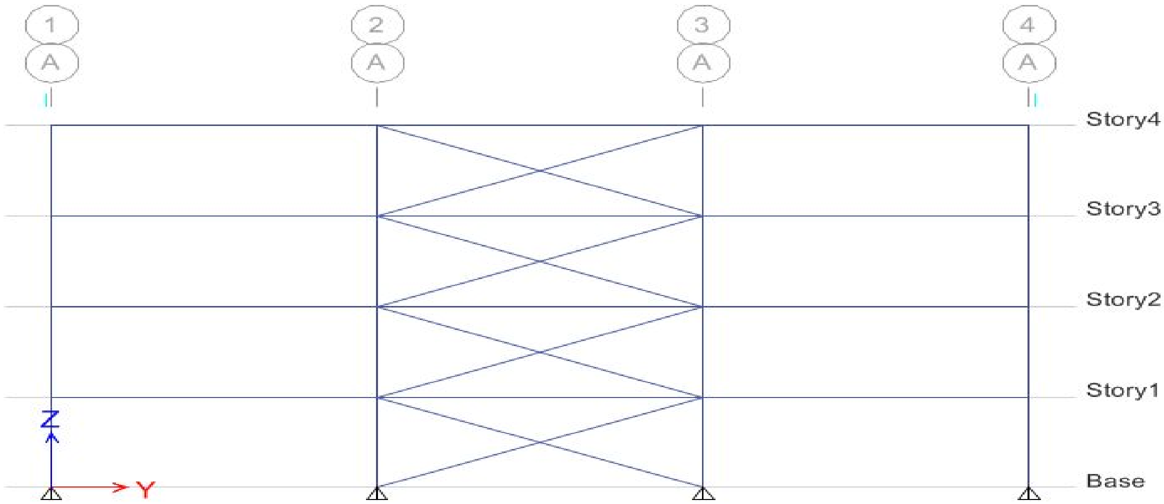


Figure 3.9 Structural layout for bracing section optimization

3.3 Material properties

Table 3.1 Material Property definition

Material Type	Concrete	Steel
Weight per unit volume	$25 KN/m^3$	$77 KN/m^3$
Isotropic Property Data		
• Modulus of Elasticity	30GPa	200GPa
• Poisson's ratio	0.2	0.3
• Shear Modulus	12500.00	76923.08
Compressive strength, f'ck	C25	
Tensile Strength		Fyk=460
Directional Symmetry	Isotropic	Isotropic
Material properties Temperature dependent?	No	No
Hysteresis curve type	Takeda	Takeda
SAP Druker-Prager Parameters		
• Friction angle	0	0
• Dilatational angle	0	0

3.4 Steel bracings selection for economic section determination using pushover analysis

The existing bare frames were strengthened by introducing steel X, and Chevron type of bracings in the middle bay as well as corner along the full heights of the structure. I-section and rectangular cross section have been selected for steel bracing. Different first class steel sections of an I-sections and rectangular sections have been used for strengthening.

3.5 Bracing Section Designation for Economic section determination using pushover analysis

I-section

Table 3.2 I section Steel cross section properties and designation for performance based optimization

	Cross Bracing section list (Euro code 3)			
Beam: 350x350mm Column: 400x400mm	IPE100	IPE240	IPE450	IPE750X137
	IPE120	IPE240 O	IPE500	IPE750X147
	IPE140	IPE270	IPE550	IPE750X161
Slab: 150mm	IPE160R	IPE270 R	IPE600	IPE750X173
	IPE180	IPE300	IPE600O	IPE750X185
	IPE180 O	IPE330	IPE600R	IPE750X196
	IPE180 R	IPE360	IPE600V	IPE750X210
	IPE200	IPE400	IPE750X137	IPE750X222

3.6 Structural Element Properties

Table 3.3 Structural members' section property

	Frame Element	Section	Story
Member Properties	Column	400mmx400mm	10
		500x500	12&16
	Beam	350mmx350mm	10
		400x400	12&16
	Slab	150 mm	All

3.7 Modelling Assumptions:

A. Material: Material nonlinearity is considered in the analysis. The self-weight of concrete assumed to be 25KN/m^3 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 1998-1-1:2014 and used in practical applications of most buildings.

B. 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.

C. Floor slabs**:** are assumed to be rigid in plane, with thickness equal to *150mm* in all models. This assumption causes the vertical elements at any level undergo the same components of translational displacement and rotation in the horizontal plane.

The bay configuration for modelling is four bay in the X by four bay in the Y direction with the constant bay width of five meters and constant story height of three meters

3.8 Analysis Methodology

Time history analysis and nonlinear static pushover analysis is utilized to compare the modal analysis, performance level and economic considerations in selecting steel cross sections. All the analysis steps and procedures is based on the new revised Ethiopian building code or the European code 8.

The analysis is performed for high seismic zone and very dense sand soil typically classified according to new Ethiopian building code as Ground type C as shown in table 2.1 which is deep deposits of dense or medium-dense sand, gravel or stiff clay with thickness from several tens to many hundreds of meters. It is assumed that soft soils are a good medium in propagating seismic waves for this reason soft ground type was selected.

3.8.1 Structural Modelling

Since the characteristics of these earthquakes are different from other places, they have to be scaled to a scale before using them for non-linear dynamic analyses of the studied models. The structure is first designed to resist gravity load and this same building is subjected to a ground motion to evaluate the performance using different bracings. And for the eccentric types of bracings by inducing an eccentricity a rigorous analysis has been carried out by changing the eccentricity value to evaluate the performance in terms of displacement ductility and energy dissipation. So different kinds of models has been simulated and tested by various eccentricities.

3.8.2 Structural configuration

The thesis work considered regular building with plan and elevation. The models under considerations are ten, twelve and sixteen story reinforced concrete buildings the bay configuration for all buildings is four in the X by four in the Y direction with the constant bay width of five meters. The study covered both eccentric (inverted V with eccentricity at the pitch) and concentric (cross diagonal and chevron bracing) types of bracings with no infilled walls.

3.8.3 Time History Analysis Methodology

Time history analysis is a step- by-step analysis of the dynamical response of a structure to a specified loading that may vary with time. The analysis may be linear or nonlinear. It is used to determine he dynamic response of a structure to arbitrary loading. The dynamic equilibrium equations to be solved are given by:

$$K u(t) + C\dot{u}(t) + M\ddot{u}(t) = r(t)$$

Where K is the stiffness matrix; C is damping matrix M is the diagonal mass matrix.

3.8.3.3 Synthetic acceleration

When there is no accelerogram data available the revised Ethiopian building code provides the usage of an artificial acceleration. For this particular analysis in this paper the synthetic acceleration data which has been collected by curve fitting the response spectrum curve for a damping ratio of 5%. The design ground acceleration, a_g, is 0.2 in g units, the ground type used is class C for a spectrum type 1. The response spectrum has been defined using the mentioned parameters then the time history has been matched.

3.8.4 Material Nonlinearity

Concrete and steel are the two constituents of RC braced frame. Among them, concrete is much stronger in compression than in tension (tensile strength is of the order of one-tenth of compressive strength). While its tensile stress–strain relationship is almost linear, the stress–strain relationship in compression is nonlinear from the beginning.

There are various forms to apply material nonlinearity in a structure: (Anon., 2016)

- Various type of nonlinear properties in Link/Support elements
- Tension and/or compression limits in Frame elements
- Hinges in Frame elements
- Material nonlinearity in layered Shell elements
- Geometric nonlinearity in Cable elements

3.9 Types of analysis

The seismic effects and the effects of the other actions include in the seismic design situation may be determined on the basis of the linear -elastic behavior of the structure. As an alternative method the code provides nonlinear analysis methods:

3.10 Fundamental period determination procedure

According to EN.1998.1.2004 (8) section 4.3.3.2.2 for buildings with height up to 40m the fundamental period of vibration may be approximated by $T1 = C_t . H^{3/4}$ where Ct=0.075 for moment resistant concrete frames otherwise the code recommends the Rayleigh method of determination. Alternatively the code suggests other options using the lateral elastic displacement at the top of the building. (*ibid*) Therefore, the same period will be used for the analysis of the base shear and natural frequencies.

3.14 Structural members

Primary elements are specified as being those elements that contribute to the seismic resistance of the structure and are designed and detailed to the relevant provisions of EC8 for the designated ductility class. Elements that are not part of the main system for resisting seismic loading can be classed as secondary elements. They are assumed to make no contribution to seismic resistance, and secondary concrete elements are designed to EC2 to resist gravity loads together with imposed seismic displacements derived from the response of the primary system. In this case, no special detailing requirements are imposed upon these elements. Therefore, stair cases, cladding,floor finishings, masonary infill walls and all secondary structual elements are neglected in this paper.

3.15 ETABS2016.EDB specification

Recently updated nonlinear finite element analysis software ETABS 2016 was used to assess the nonlinear seismic behavior of a building. The program is able to perform linear as well as nonlinear analysis very quickly. The Direct X graphics mode has been enhanced for speed and appearance, including shading and lighting. User settings are provided for advanced control. The size of the saved analysis results files has been reduced for multi-step nonlinear static, nonlinear direct-integration time-history, and nonlinear modal (FNA) time-history load cases. This will reduce the amount of disk space required for these types of load cases in models containing isotropic frame hinges, parametric P-M-M frame hinges, and layered shell elements using concrete materials. This may also result in some speed increase when running the analysis and displaying results, particularly for load cases with many steps. Note that for FNA load cases, this enhancement only affects frame hinges when modeled in links, since layered shells and frame hinges modeled in elements behave linearly in FNA load cases. In this case nonlinear direct integration is used and hinge property is defined for frame elements.

Many structures have been modelled and analyzed considering the structural simplicity, planar and spatial regularity/ irregularity, diaphragm rigidity, and bay configuration as it has been discussed in section 2.16.

3.16 Stiffness based sizing

As a complement to the methodology discussed by (Coeto, 2012), this paper discusses a stiffness-based methodology for the stiffness-based sizing of a bracing system. Once the braces have been sized, the drifts of the bracing system due to global shear drift mode can be estimated. For this purpose, it is reasonable to assume that this drift mode is exclusively a consequence of the axial deformation of braces. Within this context, the lateral shear stiffness provided by the braces to the i^{th} story can be estimated as:

$$K_{Si} = N_i \frac{E_{BR}\, A_{BR} \cos^2\theta i}{L_i} \frac{}{L_{RF}}$$

where N_i is the total number of braces located at the i^{th} story A_{BR} is the area initially proposed for each one of the axial deformation of braces, θi their inclination angle, Li their total length (distance that separates the two nodes that delimit the ends of one brace in the analytical model), and $L_{RF\,i}$ a stiffness adjusting factor

$$L_{RF\,i} = \gamma + \mu(1 + \gamma)$$

Where γ is the ratio of the length of the brace core segment (L_c) to the total brace length L and μ is the ratio of the average axial stress in the brace outside the brace core to the stress in the brace core. In this case the stiffness modification factor is assumed to be equal to 1.5 for all cases.

The main purpose of this stiffness based sizing by modifying the area of the steel braces is to create an equal comparison ground by having an equal stiffness of a cross bracing with other chevron bracing types of bracings entertained in this paper keeping other design parameters, structural members sizes and applied loads constant using a UC I section .

Table 3.4 Stiffness based bracing size determination for THA

Section	X bracing	V Bracing	V100 Bracing	V500 Bracing	V1000 Bracing	V1500 Bracing
Area, cm^2	362.5	109.4	106.5	96.1	85.6	76.5
L, m	5.8301	3.9051	3.8733	3.75	3.605	3.473
θ	59.04	39.8	39.24	36.86	33.69	30.25
Depth (h),mm	500	300	300	275	278	250
Width (w),mm	500	300	300	275	278	250
Flange (tf),mm	25	12.5	12	12	10.5	10.5
Web (tw),mm	25	12.5	12.5	12	10.5	10.5

3.17 Formulation of design optimization problem for steel bracing

Structural optimization seeks optimal values of design variables that achieve the best outcome of a given objective while satisfying code or designer-specified criteria. An objective function, often known as a cost or performance criterion, is expressed in terms of the design variables and serves as a guide for the decision maker. The optimal design is the one providing the best value for the objective function while satisfying all the constraints; thus, the selection of an appropriate objective function is extremely important. (Mohammadi, 2008)

The software is applicable for the optimal design of concentric braced frames with any number of stories and spans and with any location of bracings.

The optimal design of structure minimizes the structural weight subjected to performance constraints on axial deformations of braces and plastic hinge rotation of beam-columns and also the force interactions relationships for them. Assuming that the cost of a member is proportional to its material weight, that the unit material cost for each member is the same, and that the member has a prismatic section throughout its length, the least-cost design can be interpreted as the least-weight design of the structure, and the weight objective function (OBJ) to be minimized can be formulated as:

$$OBJ = \sum_{i=1}^{n_e} \rho_i l_i A_i$$

Where: n_e is the number of members ρ_i is the material density and l_i and A_i are the fixed length and variable cross-section area of member *I,* respectively. The design variables are chosen as the cross sectional areas braces and constraints can be summarized as

$$\Delta_p^{pl} \leq \dot{\Delta}^{pl}$$

Where Δ_p^{pl} is the bracing axial deformation and $\dot{\Delta}^{pl}$ is the allowable amount based on Table 5-7 of FEMA-356 guidelines.

CHAPTER FOUR

RESULT AND DISCUSSION

4.1 Lateral Displacement

When we compare bracings distributed along the middle of the structure with bracings around the corner periphery of the building, middle braced buildings have shown a greater reduction in lateral displacement. Having single bay braced or double bay braced has a big difference in lateral displacement. In this case double bay is braced at the middle of the frame and two orthogonal sides braced at the corner periphery. When the structure was braced at the corner the lateral displacement reduced from 111.497mm to 41.592mm for cross bracing but for the middle bracing the reduction was 34.84mm which is quite high performance. When we compare the individual bracing configurations eccentric bracings displayed a high reduction for the corner braced frames than concentric bracings. Steel links in Eccentric bracings should be designed to modulate strength and stiffness. The eccentricity may be given at the corner or at the tip in chevron bracings or the configuration may take Y, K as it has been discussed in chapter two. As we can see from the table below inducing an eccentricity reduces story displacement.

4.2 Building drift

Drift is generally defined as the lateral displacement of one floor relative to the floor below. The response for this particular level of ground shaking in terms of an inter story drift for the aforementioned building frames have been carried out. Hence, we can see that cross bracings displayed huge drift reduction than the bare frame. For this level of ground motion the response confirmed that the bare frame building portrayed a setback in the geometry but when it was braced the building displayed a uniform lateral drift as shown in the figure 4.3 below. Cross bracings reduced the maximum inter story (I.S) drift by 26.12% in the corner bracings and 21.27% in the middle bracing configuration. Ten centimeters eccentricity provided chevron bracing displayed a reduction of about 18% there by giving a complete lateral-force-resisting system that forms a continuous load path between the foundation, all diaphragm levels, and all portions of the building for proper seismic performance.

Drift limitations

Drift control is necessary to limit damage to interior partitions, elevator and stair enclosures, glass, and cladding systems. Stress or strength limitations in ductile materials do not always provide adequate drift control, especially for tall buildings with relatively flexible moment-resisting frames or narrow shear walls. Total building drift is the absolute displacement of any point relative to the base. Adjoining buildings or adjoining sections of the same building may not have identical modes of response, and therefore may have a tendency to pound against one another. Building separations or joints must be provided to permit adjoining buildings to respond independently to earthquake ground motion.

For structures with a period less than 0.7 seconds, the maximum story drift is limited to $\Delta_S \leq 0.025$ h ($T \leq 0.7$ seconds) where h is the story height
For structures with a period greater than 0.7 seconds, the story drift limit is $\Delta_S \leq 0.020$ h ($T \geq 0.7$ seconds). As per the Euro code provision the period of a building has been estimated by $T = 0.075(?)^{3/4}$, in seconds, hence in this case the period is above 0.7 seconds. Therefore, the design drift limitation is 0.6. The effect of P-Δ has been neglected because the drift seismiceablity coefficient or the overall drift results are within the drift limitation range.

4.3 Overturning moment

According to the FEMA definition response to earthquake ground motion results in a tendency for structures and individual vertical elements of structures to overturn about their bases. FEMA 356 suggests for Structures to be designed to resist overturning effects caused by seismic forces. Each vertical-force-resisting element receiving earthquake forces due to overturning have been investigated for the cumulative effects of seismic forces applied at and above the level under consideration. An increase in shear force means an increase in deflection and a decrease in moment. As we can see from the table below minimum moment is displayed at the top story but maximum displacement occurs at the top story.
Compared to the corner braced building the middle braced structure displayed a high overturning moment. Cross bracings displayed a significant amount of overturning moment amount among the corner braced buildings. Generally, middle braced buildings displayed a high overturning moment resistance capacity.

4.4 Response of Ten Story Regular Reinforced Frame subjected to ground motion

4.4.1 Response of a Corner braced regular frame

4.4.1.1. Maximum story displacement of corner bracing

Table 4.1 Maximum story displacement in the X direction off corner braced ten story 4x4 bay configuration

Story	Bare frame (mm)	X- Bracing (mm)	V-Bracing (mm)	V100 Eccentric (mm)	V500 Eccentric (mm)	V1000 Eccentric (mm)	V1500 Eccentric (mm)
10	111.497	41.592	53.915	53.989	44.925	40.056	34.843

Where V100, V500, V1000 etc. means an eccentricity of 100mm, 500mm and 1000mm is provided at the tip center of the chevron bracings respectively.

As shown in table 4.1 when using a cross bracing it displayed a significant reduction in the story displacement than unbraced frame. When using cross bracing it showed about 37% reduction in overall story displacement. A chevron bracings have shown quite significant story displacement reduction of 48%. Compared to the cross bracings V1500 Eccentric chevron bracings show better reduction in story displacement for this particular scenario. We can see that increasing eccentricity at the center of the eccentric bracing decreases the lateral story displacement. This result is valid keeping the beam-column connection rigidity factor to nil i.e. the analysis is carried out by centerline to centerline analysis method.

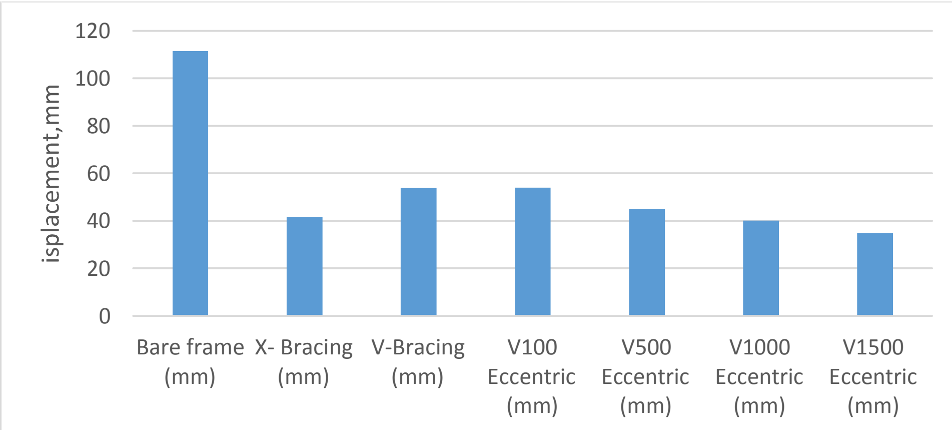


Figure 4.1 Maximum story displacement for corner braced ten story frame

Table 4.2 maximum bare frame displacement to maximum braced frame ratio

PGA	X Bracing	V Bracing	V100 Eccentric	V500 Eccentric	V1000 Eccentric	V1500 Eccentric
0.2g	2.68	2.07	2.06	2.48	2.00	3.2

As we can see from the table above, retrofitting concrete frames using steel bracings generally reduced lateral displacement and increased stiffness. However, from the analysis and models under considerations we can observe that providing an eccentricity at the center tip gives a better displacement ductility much better than cross bracings at immediate occupancy performance level for a specified damping ratio.

4.4.1.2 Maximum story drift of corner bracing

Table 4.3 Maximum story drift in X-Direction for 4x4 bay configuration (Single bay braced in every corner over the height)

	Bare frame	X- Bracing	V-Bracing	V100 Eccentric	V500 Eccentric	V1000 Eccentric	V1500 Eccentric
Max. Drift	0.4571	0.1194	0.1208	0.001211	0.1208	0.1096	0.1108

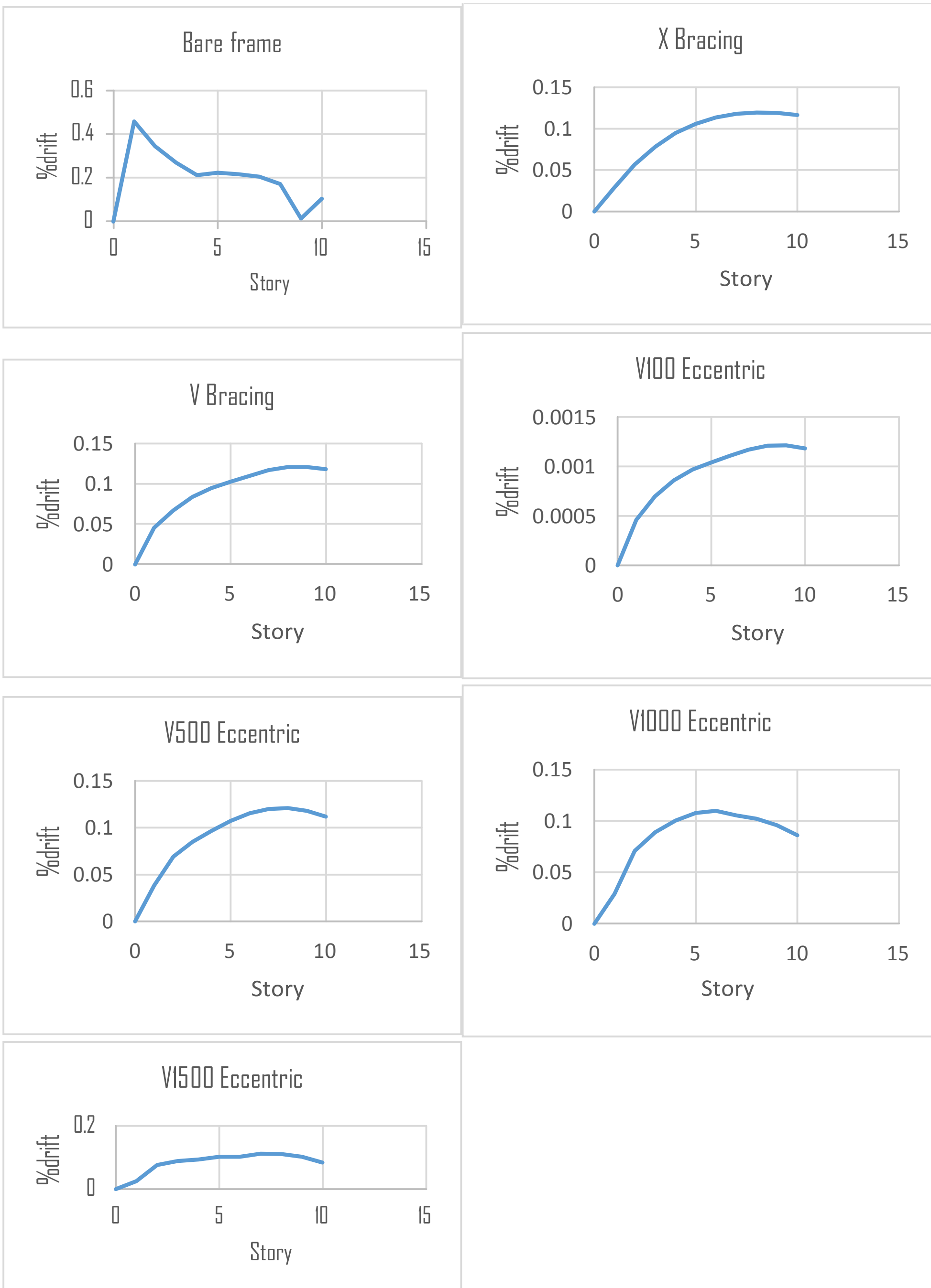


Figure 4.2Maximum inter story drift in the X direction for corner bracing of ten story frame

4.4.1.3 Maximum overturning moment in the Y direction of corner bracing

Table 4.4 maximum base overturning moment for eccentric and concentric bracing, corner braced ten story frame

Story	Bare frame (KN-m)	X- Bracing (KN-m)	V-Bracing (KN-m)	V100 Eccentric (KN-m)	V500 Eccentric (KN-m)	V1000 Eccentric (KN-m)	V1500 Eccentric (KN-m)
Base	18355.22	75146.38	70714.77	62296.73	51517.1	36673.05	25952.04

From the analysis result we can see that a cross bracing displayed the maximum overturning moment than the rest of other bracings under consideration due to their high shear strength capacity.

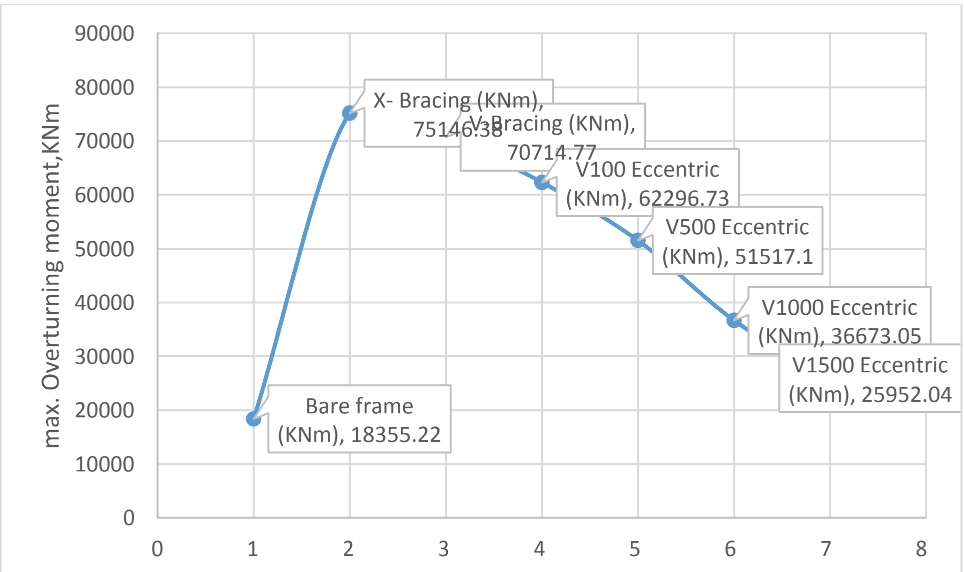


Figure 4.3Maximum base overturning moment for corner braced ten story frame

4.4.2 Response of a Middle braced regular frame

4.4.2.1 Maximum story displacement of middle bracing in the ten story

Table 4.5 Maximum story displacement in the X direction off middle braced ten story 4x4 bay configuration

Story	Bare frame (mm)	X- Bracing (mm)	V-Bracing (mm)	V100 Eccentric (mm)	V500 Eccentric(mm)	V1000 Eccentric (mm)	V1500 Eccentric (mm)
10	111.497	34.845	38.572	32.839	53.612	41.686	39.569

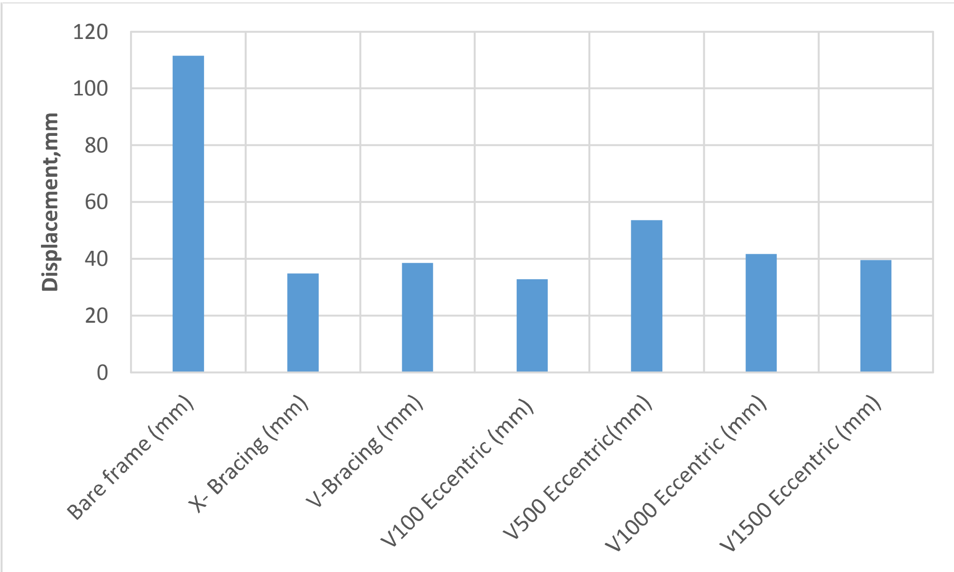
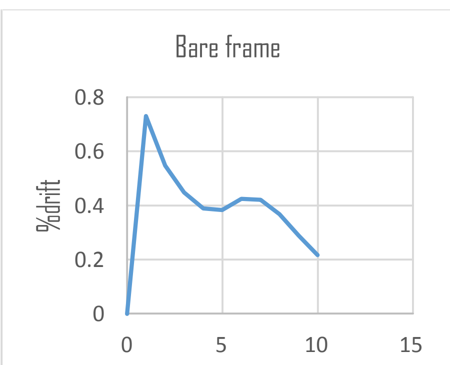


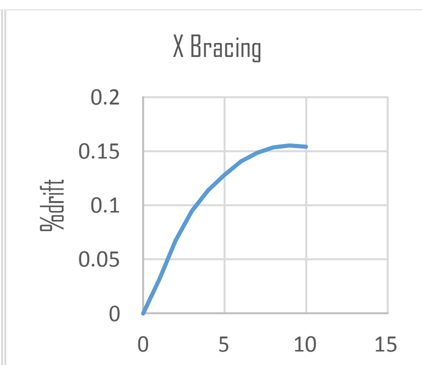
Figure 4.4Maximum story displacement for middle braced ten story frame

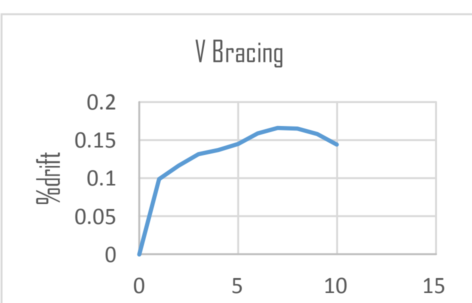
4.4.2.2 Maximum story drift of middle bracing in the ten story

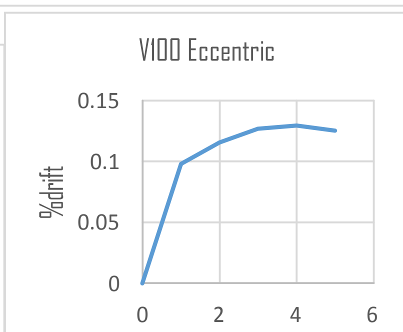
Table 4.6 Maximum story drift in X-Direction for 4x4 bay configuration (double bay braced in middle face over the height)

	Bare frame	X- Bracing	V-Bracing	V100 Eccentric	V500 Eccentric	V1000 Eccentric	V1500 Eccentric
Max. Drift	0.4571	0.1551	0.1658	0.1354	0.2048	0.1944	0.1838









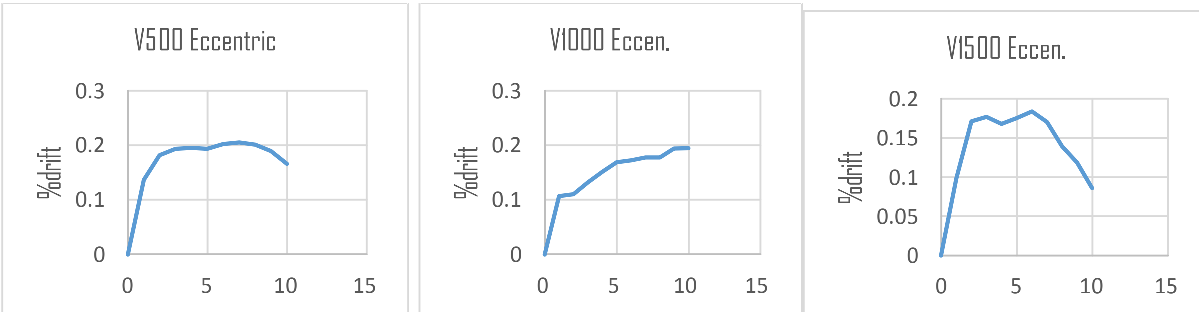


Figure 4.5Maximum inter story drift in the X direction for middle bracing of ten story frame

4.4.2.3 Maximum overturning moment in the Y-dir. Middle bracing for ten story frame

Table 4.7 maximum base overturning moment for eccentric and concentric bracing, middle braced ten story frame

Story	Bare frame (KN-m)	X- Bracing (KN-m)	V-Bracing (KN-m)	V100 Eccentric (KN-m)	V500 Eccentric (KN-m)	V1000 Eccentric (KN-m)	V1500 Eccentric (KN-m)
Base	18355.22	109723.1	69914.48	56542.53	76356.68	54598.5	32989.9

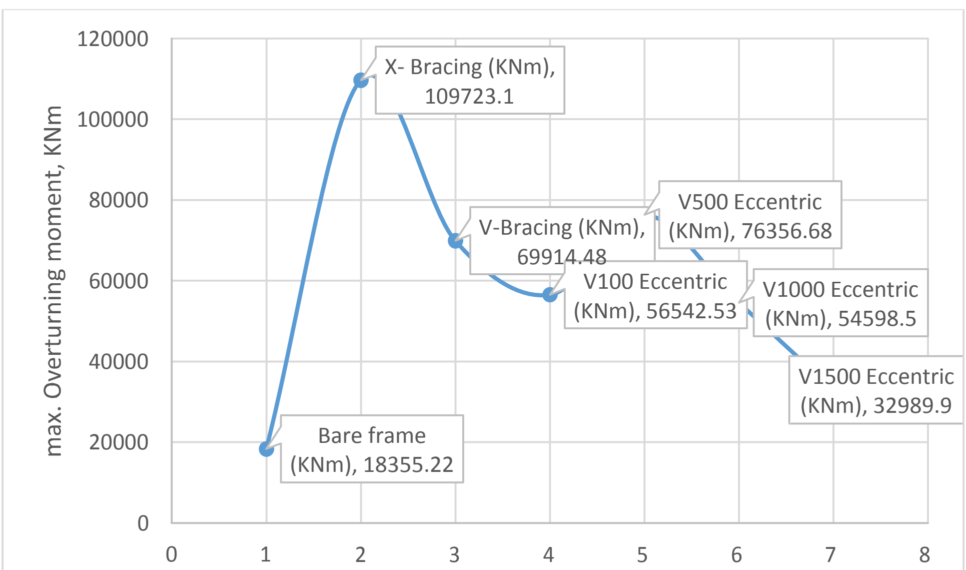


Figure 4.6 Maximum base overturning moment for middle braced ten story frame

4.5 Performance Comparison of middle braced and corner braced ten story building

4.5.1 Lateral Displacement

Compared to bracings distributed along the middle of the structure with bracings around the corner of the building, middle braced buildings have shown a greater reduction in lateral displacement.

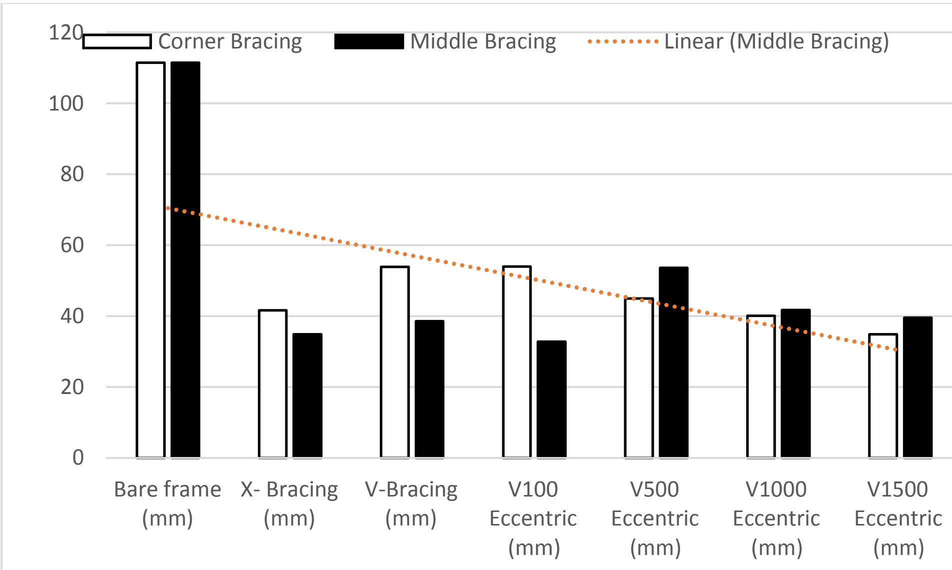


Figure 4.7 lateral displacement comparison of middle braced versus corner braced frames in the ten story regular frame

4.5.2 Overturning moment

As we can see from the comparison table below, the bracings distributed along the middle bay of the structure have a better shear resistance capacity than the corner braced ones especially when using cross bracings. This enhancement is mainly due to the energy dissipative capacity of core bracing system. Two consecutive bays have been braced by cross bracings using I section, this will enhance high energy dissipation. The results of G+12 and G+16, are listed in the appendix but we can see that having bracings at the middle bays generally increase shear stiffness and high overturning moment resistance capacity there by reducing lateral displacement.

Table 4.8 comparison of maximum overturning moment in corner braced versus middle braced ten storied frames

Story	Bare frame (KN-m)	X-Bracing (KN-m)	V-Bracing (KN-m)	V100 Eccentric (KN-m)	V500 Eccentric (KN-m)	V1000 Eccentric (KN-m)	V1500 Eccentric (KN-m)
Corner Braced	18355.22	75146.38	70714.77	62296.73	51517.1	36673.05	25952.04
Middle Braced	18355.22	109723.1	69914.48	56542.53	76356.68	54598.5	32989.9

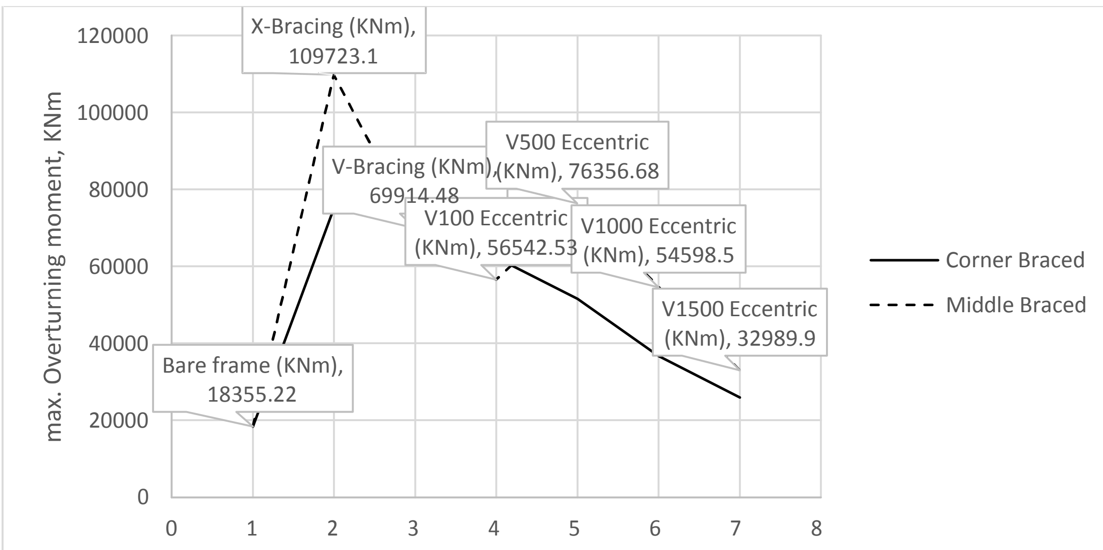


Figure 4.8 Maximum overturning moment in corner braced versus middle braced ten storied frames

4.10 Effect of eccentricity on the performance of braced frames

According to FEMA 310 inducing eccentricity is intended to force a concentration of inelastic activity at a predetermined location that will control the behavior of the system. Modern eccentrically braced frames are designed with strict controls on member proportions and special out-of-plane bracing at the connections to ensure the frame behaves as intended. (FEMA 356, 2000)

4.10.1 Effect of eccentricity in terms of lateral displacement

Eccentricity provision should be properly designed to modulate strength and stiffness. As we can see from the figure below increasing eccentricity showed a decrease in lateral displacement for corner braced frames and vice versa for middle braced frames. Optimizing the steel member is indispensable for stiffness and mass consideration. When the steel member cross section increases there would cause an increase in overall weight on the structure. Irregularity in mass and stiffness in any structure should be taken into account. Even though structural performance can be increased by proper structural or architectural configurations, proper detailing for ductility, weak beam strong column design (capacity design), taking structural regularity into account, removing setbacks in structure, avoiding irregularity in mass/stiffness, providing a rigid diaphragm, torsional rigidity and uniform load pattern. In this paper a regular building in both X and Y direction is considered. However, in both cases there is a significant reduction in story displacement.

As we can see from the figures below, there is different pattern of displacement variation along the eccentricity provision. This is due to difference in story elevation and frame section variation. However, generally providing bracings at the middle of the structure proves to be a better lateral displacement reduction method. But when we look at each story how the lateral displacement vary, they all vary like a down ward parabolic type of variation. For the ten storied frame there is a conflexure point at (1.5, 25.54) and the corner bracing displayed a steep decline in slope as we increase eccentricity from 0.5 to 1.5. The figures for the twelve story frame behaved independently. At first the corner braced frame displayed a steep positive slope as the eccentricity increase however decreased later on as eccentricity kept increasing. The middle braced frame displayed a mild change in slope as the eccentricity increased however from some level of eccentricity it behaved sinusoidally. The case of the sixteen story is very different from the past two frames. The corner braced frame displayed a linear type of variation with eccentricity. It increased with the provision of eccentricity. Middle bracings displayed open parabolic kind of variation. But, in both cases there is increase in displacement with increase in eccentricity for this level earthquake which is not necessary.

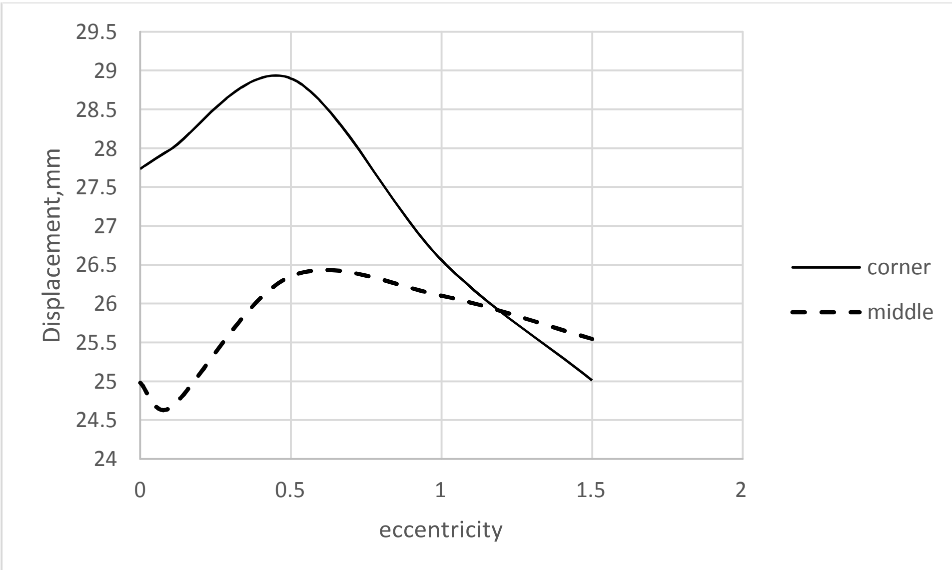


Figure 4.9 Displacement versus eccentricity comparison for ten story building

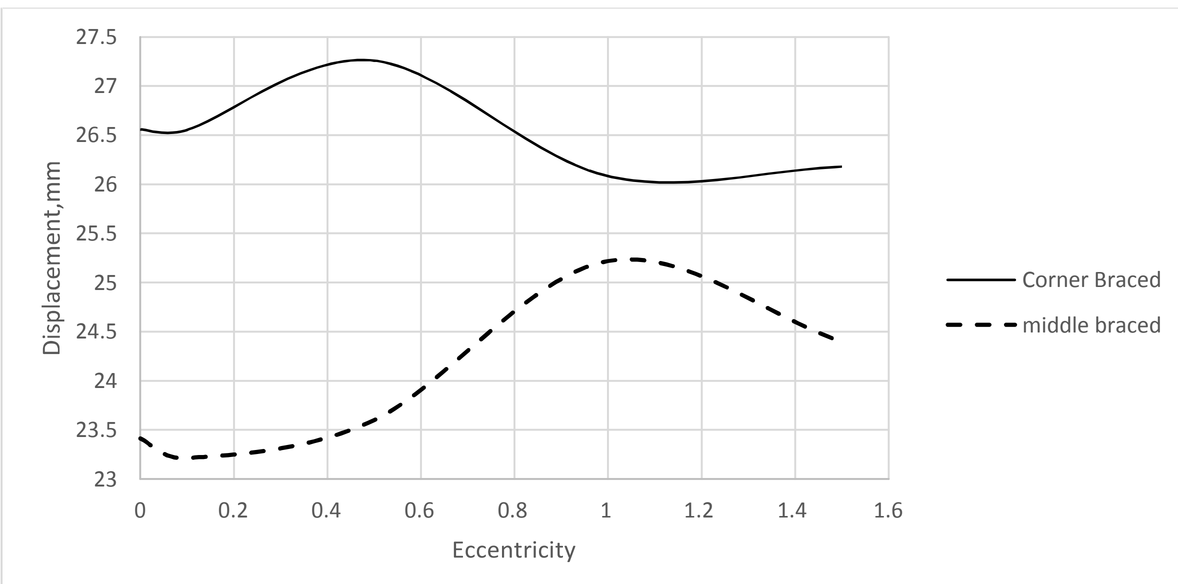


Figure 4.10 Displacement versus eccentricity comparison for twelve story building

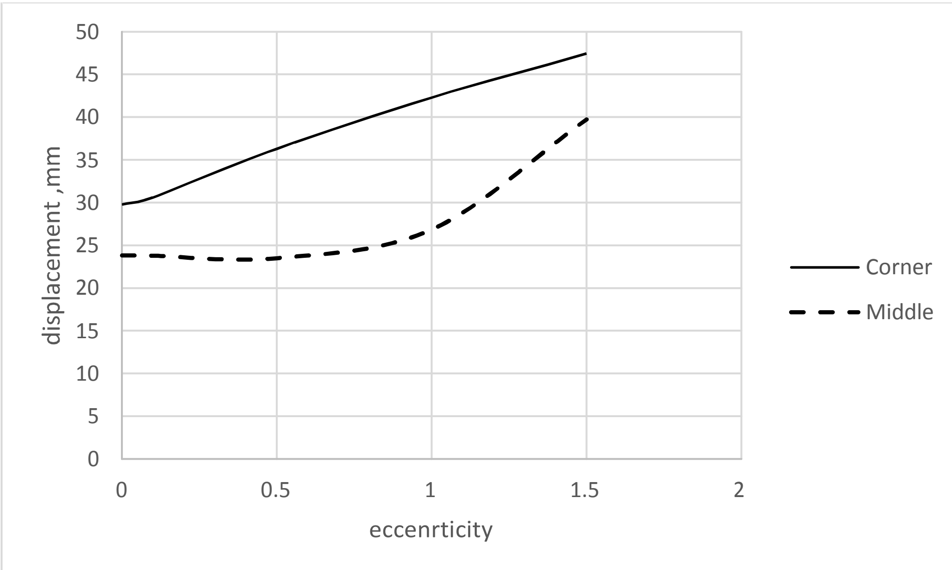


Figure 4.11 Displacement versus eccentricity comparison for sixteen story building

4.10.2 Effect of eccentricity in terms of inter story drift

The figure below shows the maximum inter-story drift (I.D) ratio for twelve storied frame for the case of eccentric bracing when subjected to a medium frequency content ground motion. From this frame it can be noticed that maximum I.D value increases with the increase in eccentricity which leads to a more ductile structure. As it can be seen from the figure approximately a constant increase in inter story drift is displayed for corner braced building and a quadratic type increase for middle braced frames. Therefore, it can be deduced that having middle bracing provides a higher inter story drift capacity.

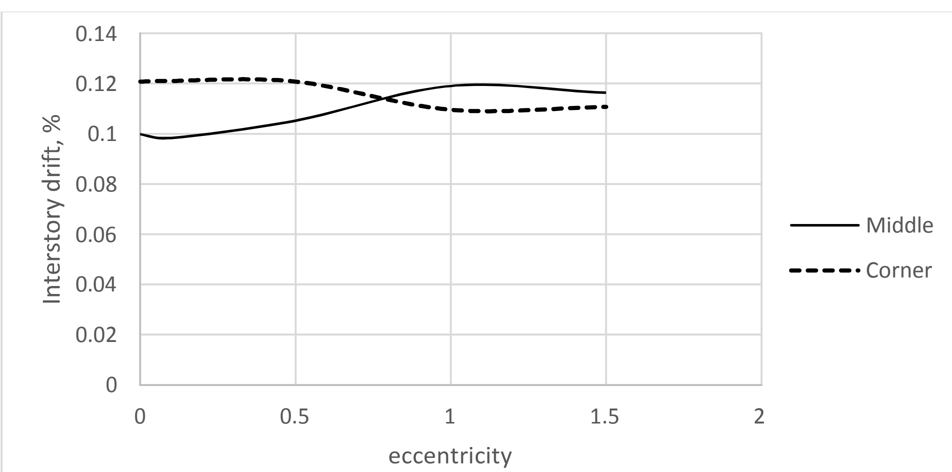


Figure 4.12 maximum inter story drift capacity with middle and corner braced building for ten story building

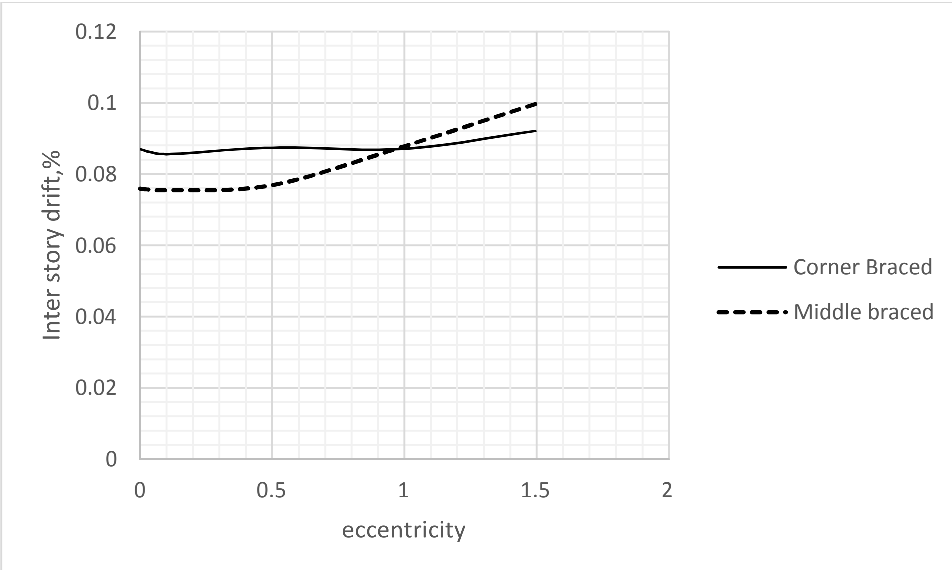


Figure 4.13 maximum inter story drift capacity with middle and corner braced building for twelve story building

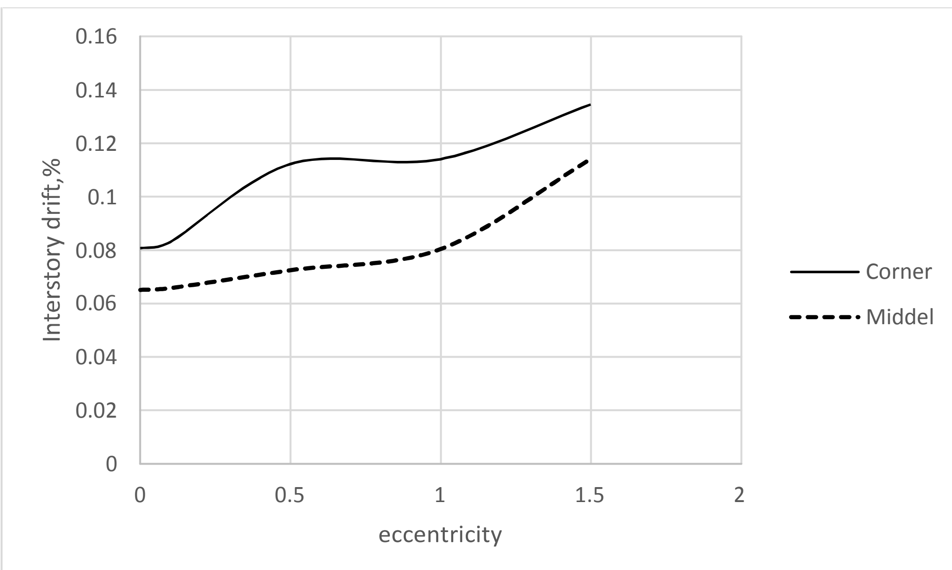


Figure 4.14 maximum inter story drift capacity with middle and corner braced building for sixteen story building

4.11 Performance based brace section optimization

The three most pillars in structural design are Strength, Aesthetics and Economy in any design considerations. When have seen many structures collapsed or damaged otherwise forming cracks due to various reasons but primarily due to earthquake. In this paper it is strived to propose an economic considerations solely based on the nonlinear behavior of a structure.

A four story three bay in the X and Y direction reinforced concrete structure is used for this analysis. As we have seen previously cross bracings have displayed a significant displacement ductility in lateral drift for this reason cross bracing is used at middle bay of the structure. Different

I section cross sections have been analyzed using pushover analysis to check the performance of the structure while getting the optimized section.

It is assumed the cost of steel per kilogram to be fixed birr per meter for any sections. Hence, there is a linear relationship between the steel mass and the corresponding cost. When the thickness increase the mass increases thereby increasing the cost. Therefore, this kind of relationship is quite necessary as part of structural analysis and design. The reality of the matter is cost considerations should come first in terms of how much strength it is intended first. There are some sort of structures that the code categorized as importance factor, that need a special attention for that cases the designer may not bother about the cost except the strength.

As we have discussed before increase in steel cross sections decreased the effective period at the specified performance point in pushover curve. I sections performed highly in reducing lateral drift and preventing local collapse.

Table 4.23 Optimum brace sections

story	Optimized Brace sections
4	IPE600
3	IPE750X137
2	IPE750X222
1	IPE750X222

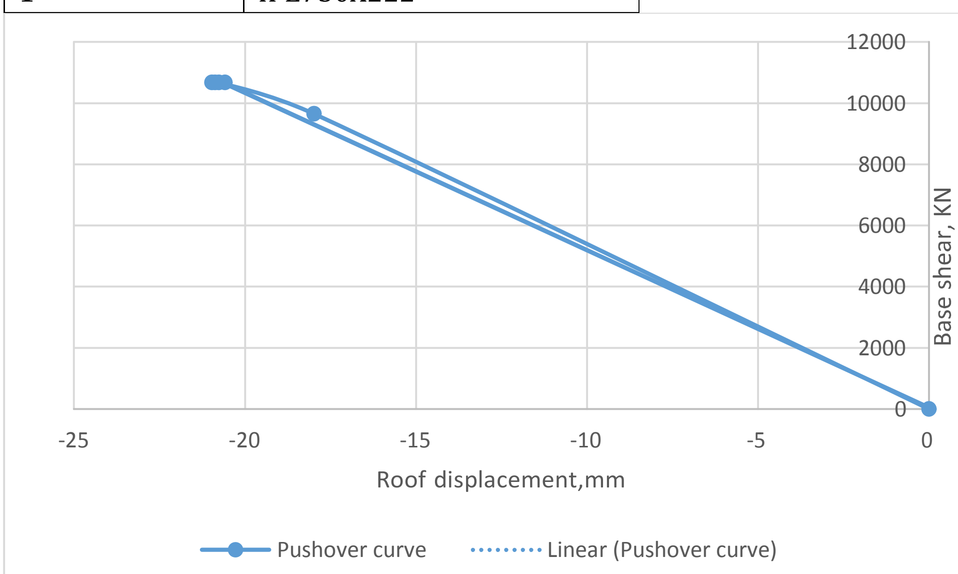


Figure 4.15 Pushover curves with lateral loading proportional to the fundamental mode shape

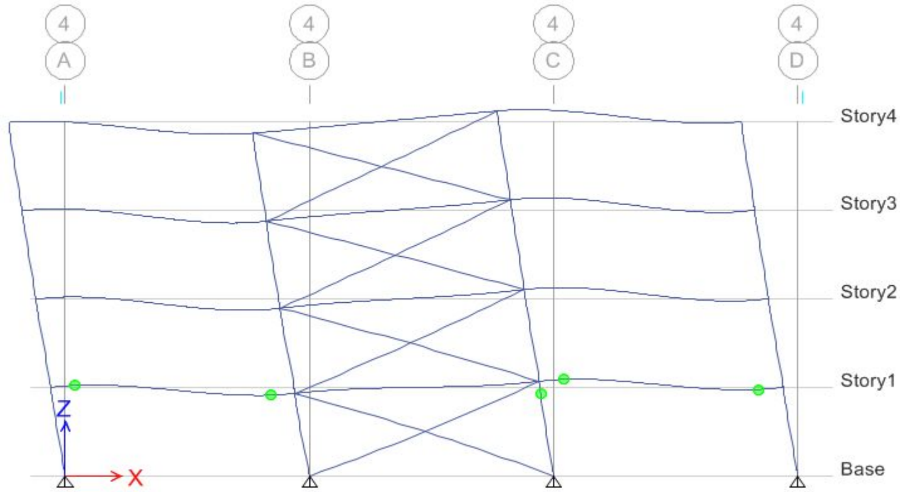


Figure 4.16 Formation of nonlinear hinges in the members of the frame due to Lateral loading proportional to the fundamental mode (Immediate Occupancy performance level)

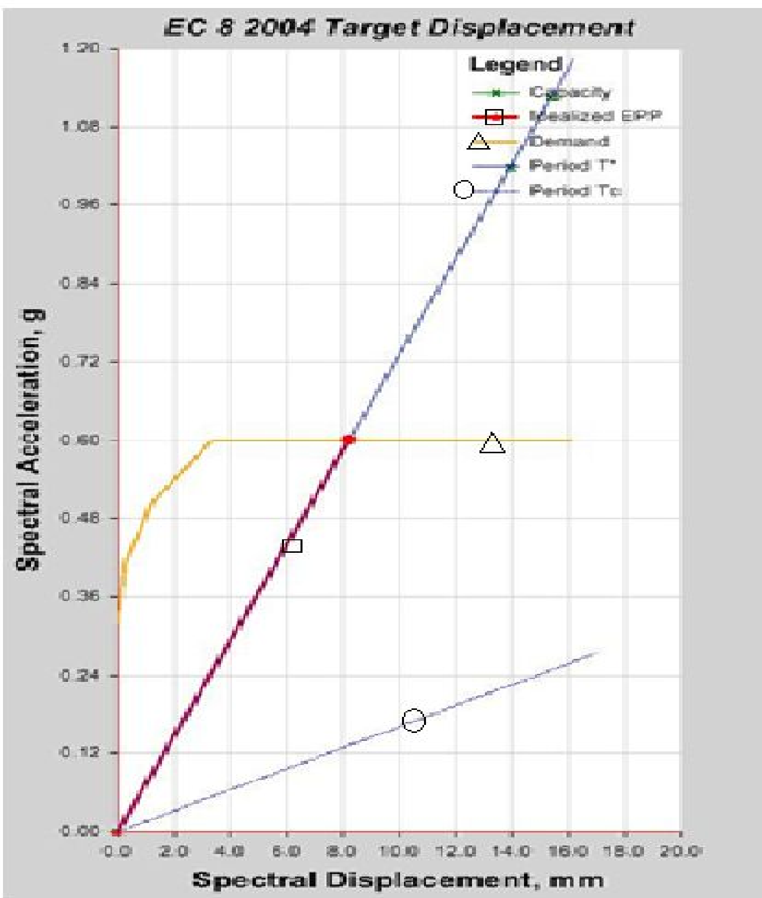


Figure 4.17 Capacity Spectrum

Table 4.24 Equivalent SDOF system target displacement

Calculated parameters	
Calculated Target displacement, dt* (mm)	5.24
Damping ratio	5%
T* (sec)	0.235
S_e(T*).g	0.288
Γ	1.334
E_m (mm)	0.565
Shear at dt* (KN)	2724.12

Where T*=the period of the equivalent reduced SDOF system; Se (T*) =the spectral acceleration at the period T*, Γ= is the modal participation factor used to convert the control node displacement to the spectral displacement. $E_{m=}$ the area under the capacity spectrum curve up to the target displacement of the SDOF (single degree of freedom) system.

CHAPTER FIVE

CONCLUSIONS AND RECOMMENDATIONS

5.1 CONCLUSIONS

Several models of story ten, twelve and sixteen have been modelled, analyzed and designed for a seismic load of artificial acceleration. Frame hinge properties were properly defined and nonlinear time history analysis is performed. The structural response output of a bare frame as well as braced frames were presented in figures and tables. For the economic considerations nonlinear static pushover analysis was carried out because when selecting steel bracing we should consider from the performance of the structure perspective so that no local or global collapse should occur.

This study has demonstrated that performance-based criteria could be implemented not only for retrofitting existing structures but also for design of new buildings. The incorporation of performance-based criteria in the design process allows the designer to design a structure for a specific safety level. By adopting these criteria, the designer has a better control of the project, and can determine the expected structural behavior in the case of other earthquakes, which may occur in the life of the structure.

As it can be seen from the result and discussion cross bracings have reduced the lateral displacement on average of 55% better than as built one. Chevron bracings displayed 45% reduction in the corner bracings and 52.6% in the middle bracings. Eccentric type of bracings are good in dissipating energy at the steel link provided however in this case, they demonstrated lower displacement drop compared to the concentric types of bracings in the ten, twelve as well as the sixteen stories considered for this kind of structural configuration and level of seismic excitations. Hence, from the results obtained the following conclusions have been drawn:

- ✓ Steel inclusion for bracing displayed lateral displacement reduction of 47% and 1.8 stiffness increase
- ✓ Overturning moment resistance increases with inclusion of steel bracing due to increase in the overall stiffness of the structure especially when using cross bracings.

- ✓ Compared to corner bracings middle bracings have displayed a better reduction in lateral displacement, inter story drift capacity, and high overturning moment resistance capacity.
- ✓ Using cross bracing has a better lateral displacement reduction capacity than the rest bracings studied in this paper.
- ✓ Increasing eccentricity in corner bracing configuration reduce displacement for a while but increase later on. However, eccentricity inclusion is very effective for middle bracing configuration.
- ✓ Performance-based criteria could be implemented not only for retrofitting existing structures but also for design of new buildings

5.2 RECOMMENDATIONS

This paper barely covered a bit of various structural phenomenon's out there concerning steel bracings. The following has been pointed out for future study

1. The seismic behavior of steel bracing for staged constructions using nonlinear analysis should be done in the future.
2. Nonlinear analysis of different types off bracings with various accelerogram records
3. The seismic behavior of irregular type of buildings should be studied in the future.

Appendices

4.4.1.1. Maximum story displacement of corner bracing

Table 4.9 Maximum story displacement in the X direction off corner braced ten story 4x4 bay configuration

Story	Bare frame (mm)	X- Bracing (mm)	V-Bracing (mm)	V100 Eccentric (mm)	V500 Eccentric (mm)	V1000 Eccentric (mm)	V1500 Eccentric (mm)
10	111.497	41.592	53.915	53.989	44.925	40.056	34.843
9	105.379	35.928	48.202	48.325	41.102	37.023	31.862
8	96.752	30.158	42.168	42.262	36.926	33.38	28.136
7	85.713	24.867	36.005	35.884	32.343	29.05	26.039
6	74.847	20.677	29.822	29.329	27.376	24.921	22.785
5	70.517	16.343	23.681	22.827	22.157	21.546	18.676
4	62.723	11.997	17.706	16.669	16.887	17.535	15.172
3	51.717	7.859	12.103	11.432	11.787	12.795	10.954
2	38.248	4.232	7.122	6.928	7.033	7.667	6.365
1	21.872	1.486	3.053	3.044	2.918	2.96	2.285
Base	0	0	0	0	0	0	0

Where V100, V500, V1000 means an eccentricity of 100mm, 500mm and 1000mm is provided at the tip center of the chevron bracings respectively.

4.4.1.2 Maximum story drift of corner bracing

Table 4.10 Maximum story drift in X-Direction for 4x4 bay configuration (Single bay braced in every corner over the height)

Story	Bare frame	X- Bracing	V-Bracing	V100 Eccentric	V500 Eccentric	V1000 Eccentric	V1500 Eccentric
10	0.001042	0.001163	0.00118	0.001179	0.001116	0.000858	0.000846
9	0.001343	0.001188	0.001208	0.001211	0.001179	0.000955	0.001018
8	0.001721	0.001194	0.001205	0.001208	0.001208	0.00102	0.001106
7	0.002046	0.001176	0.001167	0.001167	0.001199	0.00105	0.001108
6	0.002146	0.001135	0.001095	0.001105	0.001153	0.001096	0.001019
5	0.002241	0.001056	0.001025	0.001039	0.001071	0.001076	0.001013
4	0.002128	0.000944	0.000946	0.000966	0.000963	0.001002	0.00093
3	0.002698	0.000783	0.000838	0.000856	0.000847	0.000889	0.000893
2	0.003432	0.000564	0.000669	0.000695	0.000693	0.000706	0.000755
1	0.004571	0.00029	0.000451	0.00046	0.000382	0.00029	0.000239
Base	0	0	0	0	0	0	0

Table 4.11 maximum base overturning moment for eccentric and concentric bracing, corner braced ten story frame, KNm

Story	Bare frame	X- Bracing	V-Bracing	V100 Eccentric	V500 Eccentric	V1000 Eccentric	V1500 Eccentric
10	0	0	0	0	0	0	0
9	-1097.41	-3882.44	-2298.7	-1791.45	-2137.89	-1872.86	-1556.52
8	-2860.86	-10726.5	-6366.21	-5250.1	-5844.39	-4933.67	-3566.99
7	-4876.87	-19100.1	-11528.3	-10565.4	-10537.6	-8601.75	-5466.53
6	-7577.48	-27653.3	-17358.3	-17395.6	-15871.2	-12706	-8052.4
5	-10350	-35290.2	-23857.9	-25209.6	-21192.3	-16612.2	-10638
4	-12533.2	-41409.6	-31098.2	-33321.9	-25977.2	-20055	-12965.7
3	-14265.1	-46571.5	-39488.5	-41230.6	-30354.1	-23076.7	-15612.9
2	-15988.8	-52380.6	-48476.8	-48811.4	-34826.9	-25970.3	-17782.1
1	-17945.8	-63640.5	-57585	-56104.3	-42185.8	-29836.6	-20195.4
0	18355.22	75146.38	70714.77	62296.73	51517.1	36673.05	25952.04

4.4. Response of a Middle braced regular frame

4.4.2.1 Maximum story displacement of middle bracing in the ten story

Table 4.12 Maximum story displacement in the X direction off middle braced ten story 4x4 bay configuration

Story	Bare frame (mm)	X- Bracing (mm)	V-Bracing (mm)	V100 Eccentric (mm)	V500 Eccentric(mm)	V1000 Eccentric (mm)	V1500 Eccentric (mm)
10	111.497	34.845	38.572	32.839	53.612	41.686	39.569
9	105.379	30.308	34.261	30.297	48.619	39.001	37.37
8	96.752	25.725	30.126	27.575	42.922	36.544	34.387
7	85.713	21.164	26.454	24.618	37.507	33.448	30.285
6	74.847	16.941	22.586	21.372	32.059	29.596	25.591
5	70.517	12.974	18.562	17.849	26.509	25.016	22.06
4	62.723	9.197	14.448	14.092	21.046	19.698	18.054
3	51.717	5.786	10.337	10.206	15.349	13.868	13.384
2	38.248	2.954	6.404	6.401	9.554	8.368	8.081
1	21.872	0.954	2.962	2.939	4.099	3.302	2.953
Base	0	0	0	0	0	0	0

4.4.2.2 Maximum story drift of middle bracing in the ten story

Table 4.13 Maximum story drift in X-Direction for 4x4 bay configuration (double bay braced in middle face over the height)

Story	Bare frame	X- Bracing	V-Bracing	V100 Eccentric	V500 Eccentric	V1000 Eccentric	V1500 Eccentric
10	0.002175	0.001538	0.001439	0.001152	0.001665	0.001944	0.000859
9	0.002876	0.001551	0.001579	0.001271	0.001899	0.00194	0.001185
8	0.00368	0.001534	0.00165	0.001337	0.002009	0.001777	0.001397
7	0.004204	0.001482	0.001658	0.001354	0.002048	0.001772	0.001703
6	0.00424	0.001402	0.001588	0.00131	0.002018	0.001719	0.001838
5	0.003833	0.00128	0.001447	0.001252	0.001933	0.001688	0.001752
4	0.003897	0.001137	0.00137	0.001295	0.001953	0.001518	0.001682
3	0.00449	0.000944	0.001312	0.001268	0.001932	0.001315	0.001768
2	0.005459	0.000669	0.001162	0.001154	0.001818	0.001101	0.001709
1	0.007291	0.000318	0.000987	0.00098	0.001366	0.001062	0.000984
Base	0	0	0	0	0	0	0

4.4.2.3 Maximum overturning moment in the Y-dir. Middle bracing for ten story frame

Table 4.14 maximum base overturning moment for eccentric and concentric bracing, middle braced ten story frame

Story	Bare frame	X-Bracing	V-Bracing	V100 Eccentric	V500 Eccentric	V1000 Eccentric	V1500 Eccentric
10	0	0	0	0	0	0	0
9	-1097.41	-3784.46	-2853.99	-2517.36	-3080.75	-2132.17	-1394.95
8	-2860.86	-11070.9	-8022.74	-6890.81	-8496.57	-5456.64	-3732.91
7	-4876.87	-20724.1	-14871.5	-12270.8	-15263.1	-9068.87	-6847.82
6	-7577.48	-31621.2	-22756.4	-17960.1	-22846.1	-12939.8	-10281
5	-10350	-42847.7	-30881.5	-24270.3	-30981	-18847.8	-13640
4	-12533.2	-53808.6	-38522.4	-30583.7	-39297.4	-24732.7	-17857.6
3	-14265.1	-64623.7	-45087.7	-36187.8	-47488.4	-30011.7	-21597.9
2	-15988.8	-76859.7	-50738	-40953.1	-56168.8	-34392.6	-25064
1	-17945.8	-93935.3	-56132	-49606.2	-66541.1	-40313.3	-28443.8
0	18355.22	109723.1	69914.48	56542.53	76356.68	54598.5	32989.9

4.6.2 Maximum story displacement of corner braced twelve story frame

Table 4.15 Maximum story displacement in the X direction off corner braced twelve story 4x4 bay configuration

Story	Bare frame (mm)	X- Bracing (mm)	V-Bracing (mm)	V100 Eccentric (mm)	V500 Eccentric (mm)	V1000 Eccentric (mm)	V1500 Eccentric (mm)
12	44.828	28.101	26.557	26.552	27.258	26.084	26.179
11	43.716	25.527	24.207	24.279	25.241	24.06	24.264
10	41.951	22.83	21.732	21.884	23.083	21.755	21.97
9	39.394	20.045	19.181	19.406	20.808	19.265	19.437
8	36.053	17.198	16.58	16.866	18.414	16.779	16.756
7	32.491	14.332	13.971	14.297	15.913	14.342	14.592
6	29.213	11.506	11.398	11.741	13.331	12.026	12.325
5	25.752	8.79	8.911	9.243	10.71	9.919	9.962
4	22.026	6.263	6.565	6.859	8.107	7.664	7.598
3	17.93	4.018	4.424	4.657	5.597	5.354	5.247
2	13.322	2.154	2.56	2.714	3.277	3.094	2.945
1	7.808	0.788	1.071	1.138	1.31	1.116	0.976
Base	0	0	0	0	0	0	0

4.6.3 Maximum story drift of corner braced twelve story frame

Table 4.16 Maximum story drift in X-Direction for 4x4 bay configuration (double bay braced at the corner face over the height)

Story	Bare frame	X- Bracing	V-Bracing	V100 Eccentric	V500 Eccentric	V1000 Eccentric	V1500 Eccentric
12	0.000477	0.000858	0.000784	0.000758	0.000675	0.000675	0.000638
11	0.000691	0.000899	0.000825	0.000801	0.000728	0.000768	0.000765
10	0.000866	0.000928	0.00085	0.00083	0.000763	0.00083	0.000845
9	0.001114	0.000949	0.000867	0.000849	0.000798	0.000864	0.000894
8	0.00133	0.000955	0.00087	0.000856	0.000834	0.000871	0.000921
7	0.00147	0.000942	0.000858	0.000852	0.000861	0.000856	0.000919
6	0.001523	0.000905	0.000829	0.000833	0.000874	0.000826	0.000884
5	0.001501	0.000842	0.000782	0.000795	0.000868	0.000785	0.000818
4	0.001436	0.000748	0.000714	0.000734	0.000836	0.00077	0.000784
3	0.001536	0.000621	0.000621	0.000648	0.000773	0.000753	0.000767
2	0.001838	0.000455	0.000496	0.000525	0.000656	0.000659	0.000656
1	0.002603	0.000263	0.000357	0.000379	0.000437	0.000372	0.000325
Base	0	0	0	0	0	0	0

4.6.2.1 Maximum story displacement of middle braced twelve story frame

Table 4.17 Maximum story displacement in the X direction off middle braced twelve story 4x4 bay configuration

Story	Bare frame (mm)	X- Bracing (mm)	V-Bracing (mm)	V100 Eccentric (mm)	V500 Eccentric (mm)	V1000 Eccentric (mm)	V1500 Eccentric (mm)
12	44.82	21.62	23.412	23.212	23.598	25.217	24.395
11	43.76	19.70	21.457	21.274	21.829	23.801	23.116
10	41.95	17.672	19.371	19.199	19.92	22.166	21.67
9	39.39	15.564	17.2	17.036	17.89	20.349	20.51
8	36.05	13.461	14.959	14.804	15.75	18.347	19.052
7	32.49	11.352	12.681	12.538	13.528	16.163	17.238
6	29.21	9.245	10.406	10.279	11.26	13.81	15.054
5	25.75	7.191	8.183	8.078	8.99	11.314	12.528
4	22.02	5.246	6.068	5.987	6.772	8.718	9.733
3	17.93	3.479	4.124	4.067	4.664	6.085	6.773
2	13.32	1.963	2.419	2.381	2.731	3.496	3.78
1	7.81	0.787	1.036	1.014	1.086	1.209	1.169
Base	0	0	0	0	0	0	0

4.6.2.2 Maximum story drift of middle braced twelve story frame

Table 4.18 Maximum story drift in X-Direction for 4x4 bay configuration (double bay braced in middle face over the height)

Story	Bare frame	X- Bracing	V-Bracing	V100 Eccentric	V500 Eccentric	V1000 Eccentric	V1500 Eccentric
12	0.000477	0.000638	0.000652	0.000646	0.00059	0.000525	0.000426
11	0.000691	0.000677	0.000696	0.000692	0.000648	0.000621	0.000539
10	0.000866	0.000703	0.000724	0.000721	0.000691	0.000692	0.000625
9	0.001114	0.000721	0.000747	0.000744	0.000728	0.000747	0.000687
8	0.00133	0.000727	0.000759	0.000755	0.000756	0.000782	0.000731
7	0.00147	0.000718	0.000758	0.000753	0.000769	0.000796	0.000781
6	0.001523	0.000691	0.000741	0.000734	0.000765	0.000832	0.000842
5	0.001501	0.000648	0.000705	0.000697	0.00074	0.000865	0.000932
4	0.001436	0.000589	0.000648	0.00064	0.000703	0.000878	0.000987
3	0.001536	0.000505	0.000568	0.000562	0.000644	0.000863	0.000997
2	0.001838	0.000392	0.000461	0.000457	0.000548	0.000762	0.000871
1	0.002603	0.000262	0.000345	0.000338	0.000362	0.000403	0.00039
Base	0	0	0	0	0	0	0

4.8 Response of a Sixteen Storied Regular Building

4.8.1 Response of Corner Braced regular building

4.8.1.1 Maximum story displacement

Table 4.19 Maximum story displacement in the X direction off corner braced twelve story 4x4 bay configuration

Story	Bare frame (mm)	X- Bracing (mm)	V-Bracing (mm)	V100 Eccentric (mm)	V500 Eccentric (mm)	V1000 Eccentric (mm)	V1500 Eccentric (mm)
16	55.435	26.896	29.782	30.615	36.285	42.279	47.416
15	53.656	24.441	27.836	28.584	33.481	39.548	45.015
14	51.472	21.915	25.854	26.503	30.538	36.626	42.372
13	49.133	19.495	23.876	24.369	27.514	33.663	39.661
12	46.599	17.769	21.83	22.174	25.252	30.601	36.797
11	43.563	16.618	19.71	19.915	23.263	27.519	33.873
10	40.358	15.412	17.519	17.603	21.334	24.506	30.967
9	37.32	14.05	15.274	15.259	19.302	21.539	28.137
8	34.007	12.526	13.002	12.913	17.297	18.649	25.306
7	30.287	10.848	10.744	10.607	15.104	15.734	22.393
6	26.345	9.047	8.55	8.407	12.737	12.903	19.331
5	22.343	7.172	6.476	6.405	10.393	10.137	15.971
4	19.748	5.296	4.862	4.789	7.973	7.525	12.35
3	16.586	3.517	3.338	3.271	5.559	5.055	8.588
2	12.591	1.95	1.961	1.916	3.28	2.79	4.748
1	7.255	0.739	0.826	0.8	1.295	0.938	1.454
Base	0	0	0	0	0	0	0

4.8.1.2 Maximum Story drift for corner braced sixteen story frame

Table 4.20 Maximum story drift in X-Direction for 4x4 bay configuration (double bay braced at the corner face over the height)

Story	Bare frame	X- Bracing	V-Bracing	V100 Eccentric	V500 Eccentric	V1000 Eccentric	V1500 Eccentric
16	0.000595	0.00083	0.000771	0.000791	0.001035	0.000922	0.000891
15	0.000768	0.000855	0.000795	0.000819	0.001097	0.000999	0.001018
14	0.000902	0.000863	0.000807	0.00083	0.001122	0.001062	0.001097
13	0.000941	0.000853	0.000806	0.000827	0.001116	0.001107	0.001118
12	0.001047	0.000824	0.000793	0.000811	0.001079	0.001136	0.001192
11	0.001211	0.000784	0.000774	0.000791	0.001033	0.001141	0.001295
10	0.001285	0.000737	0.000756	0.000782	0.001001	0.00112	0.001345
9	0.001281	0.000691	0.000757	0.000783	0.00095	0.001075	0.001336
8	0.001259	0.000641	0.000753	0.00077	0.000887	0.001021	0.001295
7	0.001329	0.000606	0.000731	0.000739	0.000862	0.000993	0.001247
6	0.001347	0.000625	0.000691	0.000692	0.000855	0.000949	0.001221
5	0.001434	0.000625	0.000631	0.000625	0.000843	0.000896	0.001244
4	0.00146	0.000593	0.00055	0.00054	0.000818	0.000829	0.001279
3	0.001446	0.000522	0.000459	0.000453	0.000767	0.000755	0.00128
2	0.001789	0.000404	0.000378	0.000372	0.000664	0.000617	0.001098
1	0.002418	0.000246	0.000275	0.000267	0.000432	0.000313	0.000485
Base	0	0	0	0	0	0	0

4.8.2 Response of Middle braced sixteen story frame

4.8.2.1 Maximum story displacement

Table 4.21 Maximum story displacement in the X direction off middle braced twelve story 4x4 bay configuration

Story	Bare frame (mm)	X- Bracing (mm)	V-Bracing (mm)	V100 Eccentric (mm)	V500 Eccentric (mm)	V1000 Eccentric (mm)	V1500 Eccentric (mm)
16	55.435	24.31	23.8	23.776	23.49	26.844	39.745
15	53.656	22.593	22.155	22.142	21.836	24.999	37.537
14	51.472	20.873	20.642	20.597	20.375	23.043	35.04
13	49.133	19.162	19.255	19.237	19.222	21.354	32.586
12	46.599	17.485	17.833	17.913	18.099	20.317	30.126
11	43.563	15.821	16.414	16.528	16.897	19.224	27.754
10	40.358	14.118	14.904	15.085	15.579	18.018	26.079
9	37.32	12.376	13.293	13.518	14.147	16.752	24.252
8	34.007	10.603	11.586	11.831	12.674	15.355	22.07
7	30.287	8.818	9.807	10.051	11.068	13.796	19.441
6	26.345	7.056	8.002	8.225	9.339	12.04	16.513
5	22.343	5.365	6.226	6.417	7.521	10.053	13.411
4	19.748	3.85	4.546	4.698	5.675	7.84	10.287
3	16.586	2.499	3.026	3.155	3.884	5.464	7.071
2	12.591	1.368	1.736	1.824	2.234	3.053	3.875
1	7.255	0.526	0.723	0.756	0.852	0.962	1.083
Base	0	0	0	0	0	0	0

4.8.2.2 Maximum Story drift

Table 4.22 Maximum story drift in X-Direction for 4x4 bay configuration (double bay braced in middle face over the height)

Story	Bare frame	X- Bracing	V-Bracing	V100 Eccentric	V500 Eccentric	V1000 Eccentric	V1500 Eccentric
16	0.000595	0.000601	0.000616	0.000617	0.000654	0.000637	0.00076
15	0.000768	0.000619	0.000639	0.000642	0.000702	0.000719	0.000912
14	0.000902	0.00063	0.000640	0.000652	0.000724	0.000763	0.001031
13	0.000941	0.000637	0.000651	0.000658	0.000725	0.000783	0.00109
12	0.001047	0.000638	0.000646	0.000657	0.00071	0.000788	0.001106
11	0.001211	0.000633	0.000634	0.000645	0.00068	0.000765	0.001075
10	0.001285	0.000621	0.00061	0.00062	0.000634	0.000726	0.001021
9	0.001281	0.000601	0.000577	0.000584	0.000583	0.000691	0.000991
8	0.001259	0.000599	0.000593	0.000593	0.000589	0.000676	0.000966
7	0.001329	0.000587	0.000602	0.000609	0.000592	0.000668	0.001043
6	0.001347	0.000564	0.000592	0.000603	0.000606	0.000682	0.001123
5	0.001434	0.000521	0.00056	0.000573	0.000615	0.000738	0.001142
4	0.00146	0.000458	0.000507	0.000521	0.000597	0.000792	0.001119
3	0.001446	0.000377	0.000433	0.000447	0.000551	0.000804	0.001089
2	0.001789	0.000282	0.000339	0.000356	0.000463	0.000699	0.000931
1	0.002418	0.000175	0.000241	0.000252	0.000284	0.000321	0.000361
Base	0	0	0	0	0	0	0

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