ADDIS ABABA UNIVERSITY

ADDIS ABABA INSTITUTE OF TECHNOLOGY

SCHOOL OF CIVIL& ENVIRONMENTAL ENGINEERING

POST GRADUATE PROGRAM

STRUCTURAL ENGINEERING STREAM

Seismic Behaviour of Multi Story Shear Wall Frame versus Braced Frame

By

Getachew Asefa

November, 2016

Addis Ababa

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Seismic Behaviour of Multi Story Shear Wall Frame versus Braced Frame

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

By

Getachew Asefa

Advisor: Dr. Shifferaw Taye

November, 2016

Addis Ababa

DECLARATION

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

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POST GRADUATE PROGRAM

This is to certify that the thesis prepared by Getachew Asefa, entitled: *Seismic Behaviour of Multi Story Shear Wall Frame versus Braced Frame* and submitted in partial fulfillment of the requirements for the Degree of Master of Science in Civil Engineering (Structural Engineering) complies with the regulations of the University and meets the accepted standards with respect to originality and quality.

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ABSTRACT

Presently, our country, Ethiopia is growing in the way of multi disciplines, such for example as: constructing for commercial, residential, and mixed used buildings. Headed for, it requires high rise buildings, which pass up scarcity of the land. Due to this aspect there is a need to study the structural system, which resists the lateral loads due to seismic effect. In this paper the seismic behavior of reinforced concrete shear walled frame and braced frame has been studied. A symmetric plan shear walled frame and braced frame has been selected, and compared the seismic response of the structural systems with the help of storey displacement and fundamental period of the building. This study has been conducted by considering different models with the aspect of changing different parameters such as: Varying thickness of shear wall, types of bracing, and compared effectiveness of shear walled frame and braced frame. Because of the structural characteristics of the building, Modal Response Spectrum analysis has been used. All analyses were carried out using finite element software's, ETABs version 15.

When only one bay is shear walled and braced at the corner of the building in both X and y direction, the result showed that, from braced frames inverted V-braced frame showed the maximum reduction in storey displacement and fundamental time period of the frame than X-braced frame and V-braced frame. And from shear walled frames, 300mm thickness shear walled frame showed the maximum reduction in storey displacement and fundamental time period than 200mm and 400mm thickness shear walled frame. This indicates that, Increase shear wall thickness is not always beneficial for earthquake resistance design.

When two bays in a row are shear walled and braced at the corner of the building in both X and y direction, the result showed that, from braced frames X-braced frame showed the maximum reduction in storey displacement and fundamental time period of the frame than inverted V-braced frame and V-braced frame. And from shear walled frames, larger thickness shear walled frame which is 400mm thickness shear walled frame showed the maximum reduction in storey displacement and fundamental time period.

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Besides, I would like to thank Debretabor University which sponsored me to have this great opportunity. And I would like to express my deepest gratitude to my parents and friends for their prayers and critical supports.

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CHAPTER ONE

INTRODUCTION

1.1 Background of the study

A tall building is the demand of present situation. As the height of structure increases, lateral forces due to seismic become predominant. The major portion of these shall be resisted by the structural elements. Out of different structural systems, shear wall frames and braced frames are two principal structural systems used in reinforced concrete buildings to resist earthquake forces.

Reinforced concrete shear-walls are mostly used in buildings due to better- observed performance in recent past. In areas of high seismic risk, RC shear walls have been widely used as main lateral load resisting system in medium & high rise buildings because of their high lateral stiffness.

And also the most effective and practical method of enhancing the seismic resistance is to increase the energy absorption capacity of structures by combining bracing elements in the frame. The braced frame can absorb a greater degree of energy exerted by earthquakes. Braced frame reduces the column and girder bending moments. The shear is primarily absorbed by diagonals and not by girders. The diagonals carry the lateral forces directly in predominantly axial action, providing for nearly pure cantilever behavior. Bracing members are widely used in steel structures to reduce lateral displacements and dissipate energy during strong ground motions. But, recently this concept is extended to concrete frames.

Seismic response of braced frames was compared with that of shear walled frames. The parameters studied for this thesis are bracing patterns and shear walled frames.

The bracing patterns namely Double diagonal bracing(X- bracing), V-bracing, and Chevron bracing (inverted-V bracing).The shear walled frames namely Shear wall with 200mm thickness, Shear wall with 300mm thickness, and Shear wall with 400mm thickness.

Each of the bracings and shear walls provided on the 40 storey frame building with 6m bay width.

Then this building is modeled and analyzed using finite element software's, ETABs version 15.

1.2 Statement of the problem

So far Structural Engineers focused on the need to control the horizontal displacements. The horizontal actions, coming from earthquakes become more sensitive with the height of the buildings. Without specific stiffening members, the dimensions of the columns increase to such an extent that they are no longer reasonable. So, the obvious solution is providing vertical bearing capacity or shear wall. But, in recent years besides using steel bracing for retrofitting of reinforced concrete (RC) frames, it also used in seismic design of new buildings. Therefore there is a gap which type of lateral resisting system is more efficient in resisting seismic load.

1.3 Significance of the thesis

Shear walls and different types of bracings have been analyzed to understand the structural behaviour under seismic action. These can serve as a guideline to consider and analyze the possible lateral load resisting systems during the design phase and finally selecting the best lateral load resisting systems on basis of analysis results.

1.4 Objectives

1.4.1 General objective

To demarcate the seismic behavior between shear walled frame and braced frame, based on bracing pattern, shear wall thickness, single and double bay bracing, and providing shear wall in single bay and double bay.

1.4.2 Specific Objective

- To compare the effectiveness of concentrically braced frame.
- To study the behaviour of braced frame. (When single bay and double bay is braced).
- To study the behaviour of shear wall thickness on shear walled frame system.

- To study the behaviour of shear walled frame. (When single bay and double bay is shear walled).
- To give a clue for structural designers and researchers during design of seismic resistance building.

1.5 Content of the thesis

The study comprises a Reinforced concrete structure of forty story was analyzed with three different bracing type namely; X-bracing, V-bracing, inverted V-bracing (chevron bracing), and shear wall. The general classification of bracings based on their geometrical arrangements or connection styles were grouped in to two (i.e. concentrically bracing and eccentrically bracing). In this study, the bracing considered are concentrically types of bracing.

The story height of the building is assumed to be equal with 3m and for equal treatment of the study which do not alter behavior of bracings and shear wall. For analysis of this RC building, Euro code 8- Design of structures for earthquake resistance is used. These codes are direct similarity to that of the new ESEN 1998: 2015 version.

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

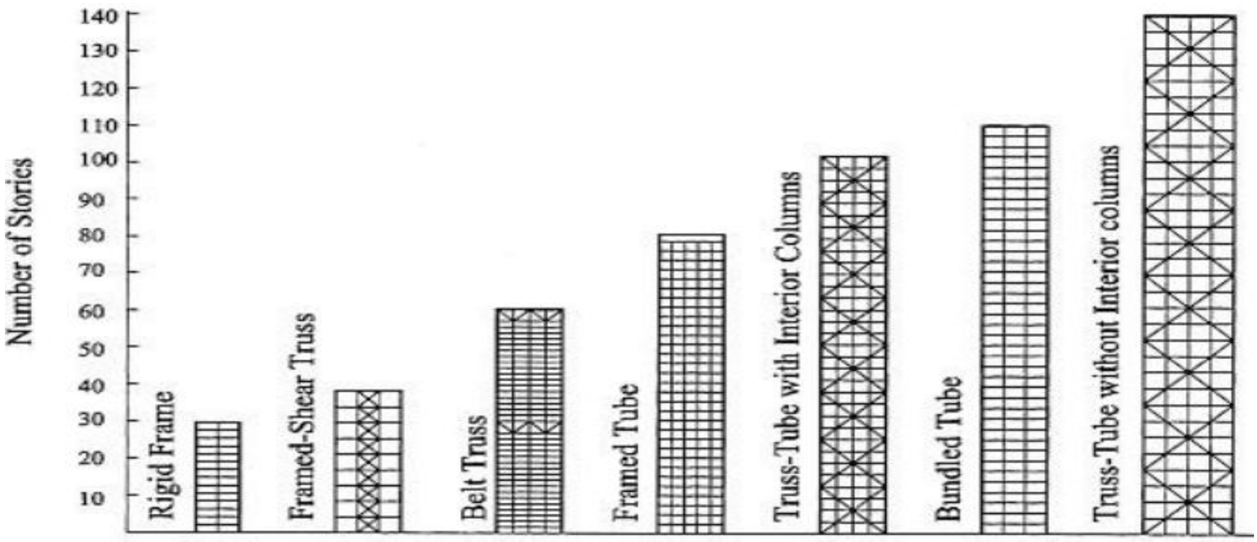
a) Section one deals with an introductory part which include background of the study, statement of the problem, significance of the thesis, objective and contents of the thesis.
b) Section two briefly reviews theoretical background of shear wall and bracing systems, classifications, principles and behaviors are considered.
c) Section three discusses about the methods of analysis.
d) Section four tells about the modeling of structural systems.
e) Section five presents, result and discussion for maximum storey displacement and fundamental period for each of the bracing type and shear wall, and were made using Microsoft excel program with the help of tables and graphs.
f) Finally, conclusions and recommendation were drawn and forwarded respectively to show research areas for the next researchers.

CHAPTER -TWO

STATE OF THE ART

2.1 Classification of tall building structural systems

In 1969 Fazlur Khan classified structural systems for tall buildings relating to their heights with considerations for efficiency in the form of “Heights for Structural Systems” diagrams (Khan, 1969). This marked the beginning of a new era of skyscraper revolution in terms of multiple structural systems. Later, he upgraded these diagrams by way of modifications (Khan, 1972, 1973). He developed these schemes for both steel and concrete as can be seen from Figure 2-1 (Ali, 2001; Ali & Armstrong, 1995; Schueller, 1986). Khan argued that the rigid frame that had dominated tall building design and construction so long was not the only system fitting for tall buildings. Because of a better understanding of the mechanics of material and member behavior, he reasoned that the structure could be treated in a holistic manner, that is, the building could be analyzed in three dimensions, supported by computer simulations, rather than as a series of planar systems in each principal direction. Feasible structural systems, according to him, are rigid frames, shear walls, interactive frame-shear wall combinations, belt trusses, and the various other tubular systems [3].



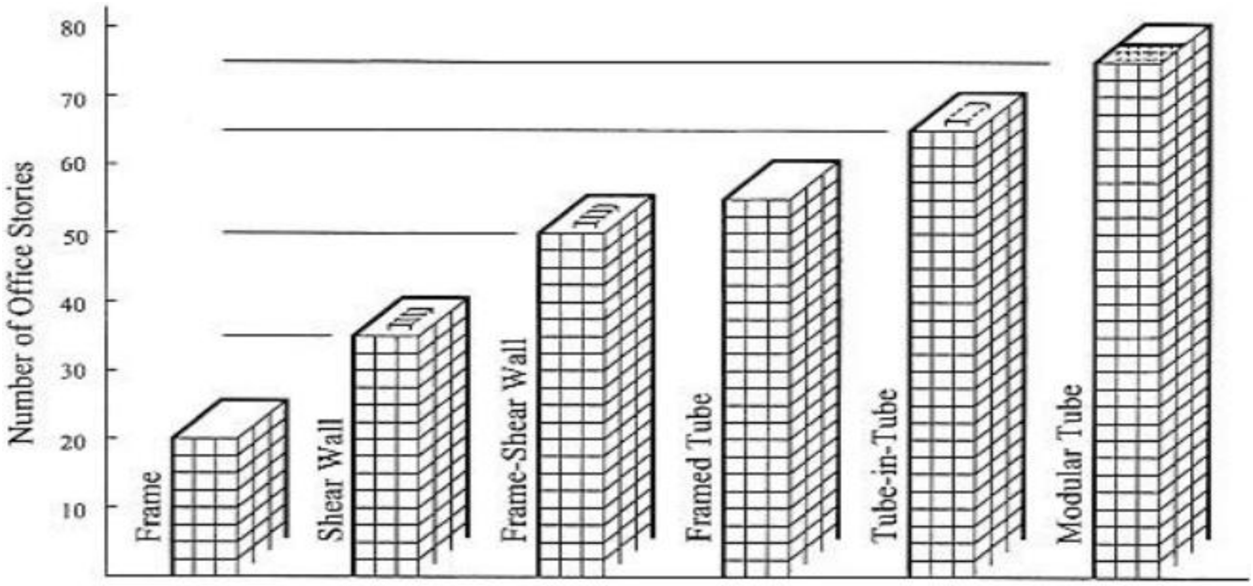


Figure 2-1: Classification of tall building structural systems by Fazlur Khan
(above: steel; below: concrete).

According to the above Fazlur Khan Classification of concrete building, framed-shear wall is used up to 50 stories.

2.2 Efficiency of structural system

Rigid frame systems are not efficient for buildings over 30 stories in height because the shear racking component of deflection caused by the bending of columns and girders causes the building to sway excessively [3]. Ali, M. and Kyoung Sun Moon [3] recommended the followings for rigid frame structure:

i. Rigid frame steel is efficient up to 30 stories.
ii. Rigid frame concrete are efficient up to 20 stories.

Systems composed of steel bracing or shear walls alone, or interacting with the rigid frames can be accepted as an improvement of the rigid frame system.

These systems are stiffer when compared to the rigid frame system, and can be used for buildings over 30 stories, but mostly applicable for buildings about 50 stories in height [10]. The interacting wall-frame combination is appropriate for building in the 40 to 60 stories range [19].

According to the structural efficiency, limiting the drift as well as the cost and stiffness of the structure Shruti Badami and M. R. Suresh [19], recommended the following:

i. Rigid frame concrete are efficient up to 70 m which is around 24 stories with 3m story height.
ii. Rigid frame concrete with shear wall are efficient from 70 m up to 140 m which is around 25 -48 stories with 3m story height.

2.3 Lateral load resisting systems

The primary purpose of all kinds of structural systems used in the building type of structures is to transfer gravity loads effectively. The most common loads resulting from the effect of gravity are dead load, live load and snow load. Besides these vertical loads, buildings are also subjected to lateral loads caused by wind, blasting or earthquake. Lateral loads can develop high stresses, produce sway movement or cause vibration. Therefore, it is very important for the structure to have sufficient strength against vertical loads together with adequate stiffness to resist lateral forces.

Seismic analysis is a major tool in earthquake engineering which is used to understand the response of buildings due to seismic excitations in a simpler manner. In the past the buildings were designed just for gravity loads, and seismic analysis is a recent development. It is a part of structural analysis and a part of structural design where earthquake is prevalent.

Lateral load resisting systems are structural elements which resist seismic, wind and eccentric gravity loads. In tall buildings lateral loads are premier one which will increase rapidly with increase in height. The design takes care of the requirements of strength, rigidity and stability. The design for lateral load will increase the structural cost substantially with increase in number of storey. To achieve economy in tall buildings special systems to resist lateral load should be adopted.

There are a lot of different systems but they can be broken down to three fundamental ones which all other systems are a combination of [6]. They are:

a) Moment resistant frames
b) Braced frames
c) Shear wall structures

2.4 Moment resistant frames

Moment resisting frame, also called moment frame or rigid frame, is made by rigid connections between horizontal and vertical members. Steel, reinforced concrete and steel-concrete composite rigid frames are used. Its resisting of lateral loads includes primarily by the flexural stiffness and strength of members and joints. Number of stories, story height and column spacing has proportional influence on the frame's strength and stiffness. Larger bending moments appears in the lower levels with its maximum in the connections. As building stories increase so does the bending moment both in beams and columns.

When the number of stories exceeds of about 30, huge dimensions of frame beams and complex connections are required that the usage of a moment resisting frame gets uneconomical [7].

The prime advantage of this system is the flexibility regarding windows and doors which is very good. That is one of the reasons why the moment frame is not abandoned, instead quite the opposite, often used combined with other systems in high-rise buildings.

2.5 Bracing systems

Pure rigid frame systems are not efficient for buildings higher than about twenty storey because the deflection produced by bending of columns and girders cause the buildings drift to be too large [6]. A braced frame attempts to improve the efficiency of pure rigid frames by adding truss members such as diagonals between floor systems so that the shear is mainly absorbed by the diagonals.

There are many different types of bracing. While the most common and one of the most effective is the X-bracing, this takes a lot of space in the structure which makes little room left for openings. V,K, diagonal- and knee bracing are other types that is often used, these provide better room for openings but are less effective against horizontal loads. There are also eccentrically braced systems that provides different shapes and openings, they have good ductility for resisting seismic forces but provide less stiffness than the concentric braced frame [7].

Using a completely braced frame system in a high-rise building, the stability is very good. This system has a few major drawbacks however. The weight of the building when completely braced becomes massive, with a lot of different pieces to fit together. Another drawback is the limitations in terms of space for windows and doors, and the ability to form an architectural expression would be less.

If the braced frame, or shear walls, and a rigid frame are combined, it produces a greater amount of lateral stiffness. This is because of the way the two systems react to the horizontal loads. With the moment frames shear deformation and the bracing's bending deformation, the combined deformation is more efficient, as shown in Figure 2.2. Instead of continuing to bend at the top the rigid frame keeps the shear wall or braced frame in place, while at the bottom the bracing, or wall, is restraining the shear deformation of the moment frame. This results in a deflection with an "S" shape [7].

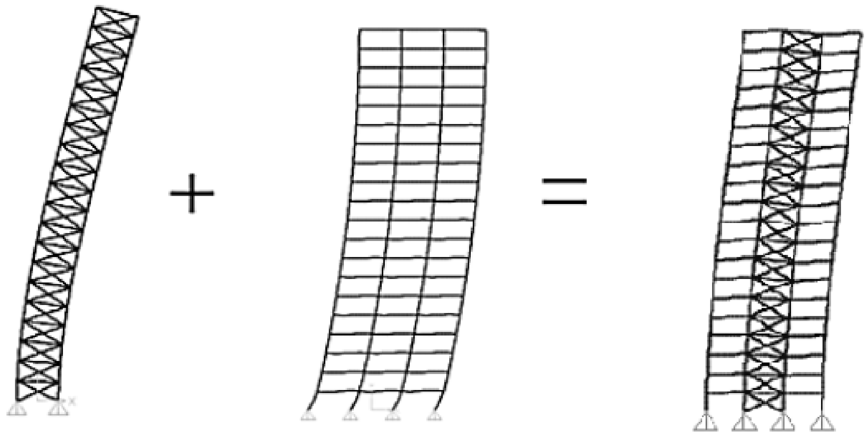


Figure 2.2: A braced frame combined with a rigid frame will decrease a buildings deflection

Instead of experiencing a maximum bending moment at the bottom of the building this now happens in the middle of the structure. This moment is also much smaller than the moment of the rigid frame system alone.

A braced rigid frame structural system is most efficient when between 40-60 stories [7]. Bracing is efficient because the diagonals work in axial stress and therefore call for minimum member sizes in providing stiffness and strength against horizontal shear

2.6 Types of bracing

On a global basis of resisting earthquake loads, shear walls are commonly used in RC framed buildings, whereas, steel bracing is most often used in steel structures. In the last two decades, a number of reports have also indicated the effective use of steel bracing in RC frames [21].

The bracing systems can be grouped according to their location in the reinforced concrete frames as internal or external and according to their connection style as eccentric or concentric bracing system [13].Based on location bracing can be Internal or external, and based on connection style bracing can be concentric or eccentric

2.6.1 Internal bracing

In internal bracing system, bracing members are introduced to the empty space enclosed by columns and beams of the frames. As a result, each unit frame is individually braced from within.

Figure 2.3: Internal bracing

2.6.2 External bracing

In external bracing system, the bracing members are introduced to the exterior frames of the building. Badoux and Jirsa [5] investigated numerically the behaviour of RC frames retrofitted with external bracing. They recommended using cables instead of steel sections for the brace elements to avoid buckling of the brace members, and thus increase the ductility of frames.

Architectural concerns and difficulties in providing appropriate connections between the bracing system and RC frames are two of the shortcomings of this method.

Figure 2.4: External bracing

2.6.3 Concentric braced frame

Concentrically braced frames (CBF) consists of beams and columns and braces which are connected with pinned connections. Thus, the members can be said to be arranged to form a vertical truss.

They resist lateral force by this truss action and columns and develop ductility by inelastic action in braces experiencing tension. They have high elastic stiffness but low ductility as the braces in compression can buckle which is a brittle failure [23].

Concentrically braced frames (CBF) Increases the lateral stiffness of the frame thus increases the natural frequency and also usually decreases the lateral storey drift. However, increase in the stiffness may attract a larger inertia force due to earthquake. Further, while the bracings decrease the bending moments and shear forces in columns, they increase the axial compression in the columns to which they are connected. Since reinforced concrete columns are strong in compression, it may not pose a problem to use bracing in RC frame.

Concentrically braced frames consist of diagonal brace members pinned to beam column junctions, such that only axial force is developed in them. The braces impart high elastic stiffness thus allowing designing the structure with smaller drift. The primary mechanism for development of ductility in CBF is axial yielding under tensile force in the bracing members. Since axial ductility is lower than flexural ductility, the ductility level of CBF is lower than that of MRF. However, earthquake induced vibrations are reversible in nature. Thus when a brace is subjected to tensile forces, at the same time, an oppositely placed brace will be subjected to compressive forces. These braces under compression buckle prematurely thus producing a brittle failure in frame [23].

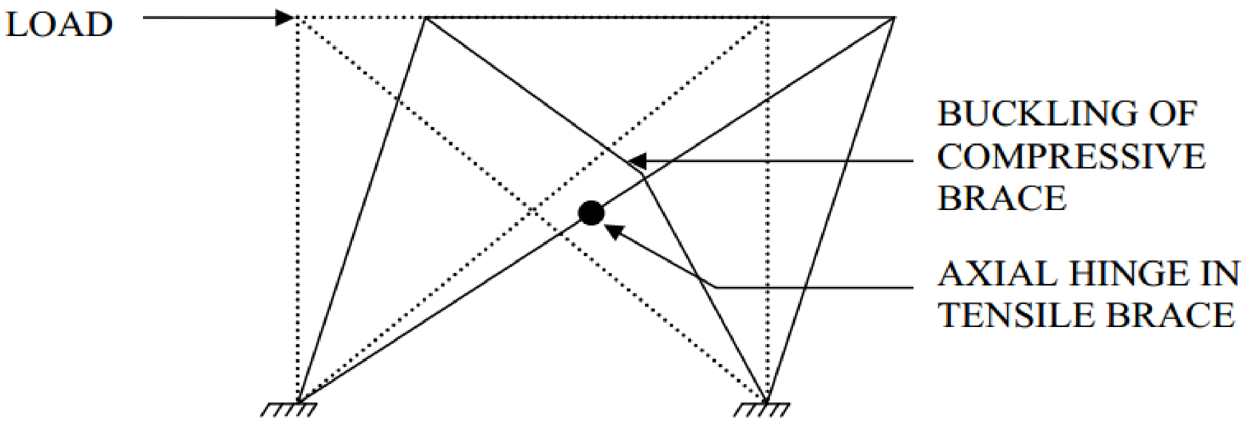


Figure 2.5: deformed shape of CBF

The followings are the types of concentric bracings

i. Diagonal bracing

This type of bracing is preferred when the availability of the opening spaces in a bay of frame are required. Diagonal bracing is obstructive in nature as it blocks the location of opening which ultimately affects the esthetic of the building elevation. It also sometimes hinders the passage for use. Diagonal bracing can be single diagonal or double diagonal as shown in Figure-2.6. If there is no architectural limitation, diagonal bracings are considered to be the most efficient in resisting the lateral forces due to wind and earthquake as these forms a fully triangular vertical truss [20].

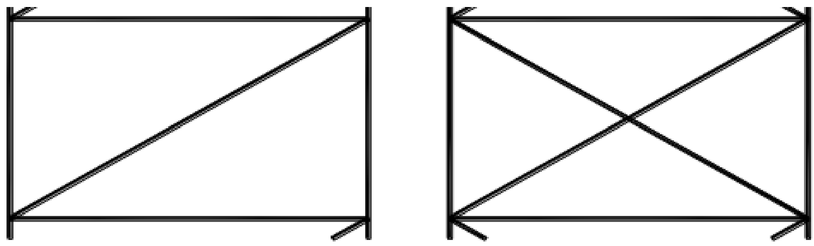


Figure 2.6: Diagonal bracing

ii. V and inverted V (chevron)-bracing

The full diagonal bracing is not used in areas where a passage is required. In such cases, V and inverted V (chevron)-bracings are preferred over diagonal bracing because there is a room to provide opening for doors and windows etc. as shown in Figure 2.7 [20].

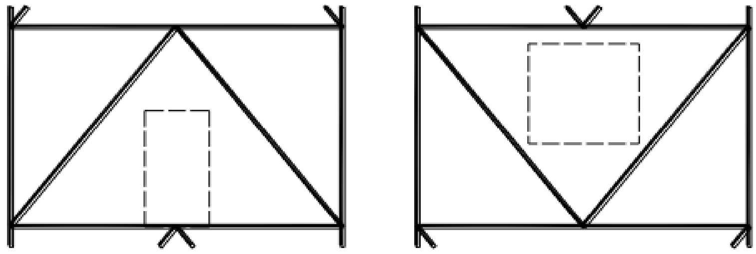


Figure 2.7: V and inverted V (chevron) -bracing

2.6.4 Eccentric braced frame

This type of bracing reduces the lateral stiffness of the system and improves the energy dissipation capacity. The lateral stiffness of the system depends upon the flexural stiffness property of the beams and columns, thus reducing the lateral stiffness of the frame. The vertical component of the bracing forces due to earthquake causes lateral concentrated load on the beams at the point of connection of the eccentric bracings.

Besides V and inverted V (chevron) -bracing, there is another type in which door and window openings can be allowed known as eccentric bracing as shown in Figure 2.8. Such type of bracing arrangement cause the bending of the horizontal members of the web of braced bent. Generally these types of braced bents resist the lateral forces by bending action of beams and columns. These provide less lateral stiffness hence less efficient as compared to diagonal bracing [20].

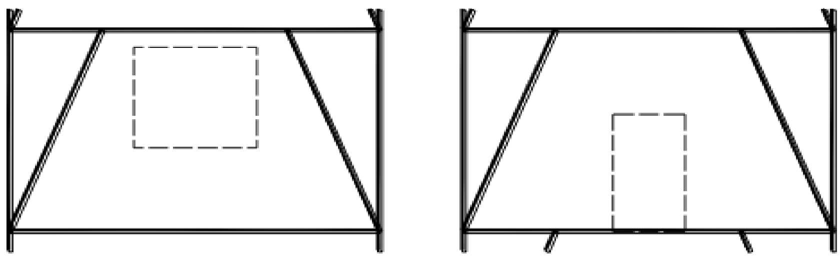


Figure 2.8: Eccentric bracing

Eccentrically Braced Frames (EBFs) are a lateral load resisting systems for steel building that can be considered as a hybrid between conventional Moment Resisting Frame (MRFs) and Concentrically Braced Frames (CBFs).Properly designed EBF display high levels of ductility as by MRF and at the same time possesses high elastic stiffness similar to CBF. Also EBF display better architectural versatility than CBF to provide space for openings. When an EBF is subjected to lateral load, the axial force induced in the brace is transferred in the form of high levels of shear and bending moment in the link.

However, the link is usually subjected to typically low levels of axial force. Consequently, links will normally experience shear and /or flexural yielding during an earthquake and undergo formation shear or flexural plastic hinges. Thus in EBF links, by undergoing plastic deformations, allow dissipating seismic energy and act as a fuse to prevent damage in other parts of the frame. Other members of an EBF, including the braces, the columns and the beams segments outside of the links are intended to remain essentially elastic during an earthquake.

The link undergoing shear yielding when lateral load is applied is can be seen from the rigid body deformed shape for an EBF as presented in Figure 2.9 [23].

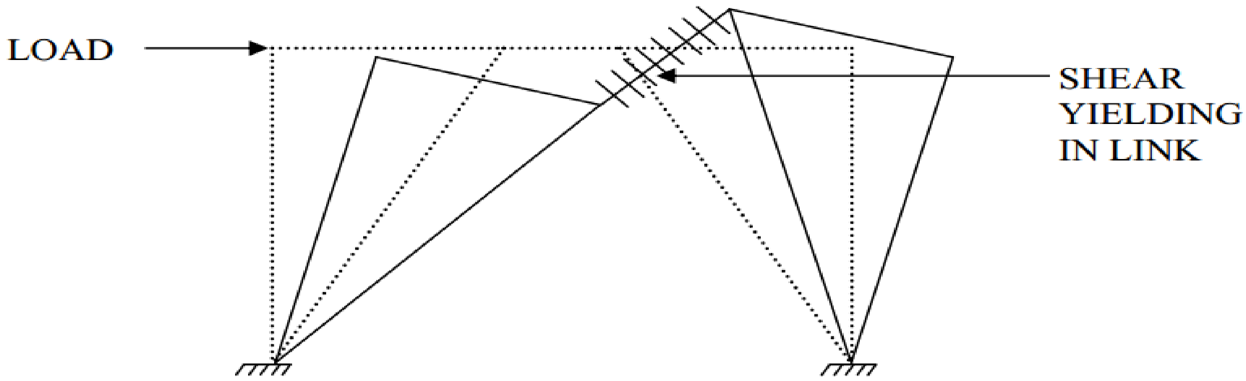


Figure 2.9: deformed shape for EBF

2.7 *Bracing connection for bracing system*

The CBF system was developed with the intention of maximizing the inelastic drift capacity that could be obtained based on brace buckling and yielding while maintaining lateral resistance. As such, connection rupture is to be avoided. This is achieved by requiring connections to be designed for forces corresponding to the expected strength of the brace as it undergoes inelastic axial deformations (yielding in tension and buckling in compression.)

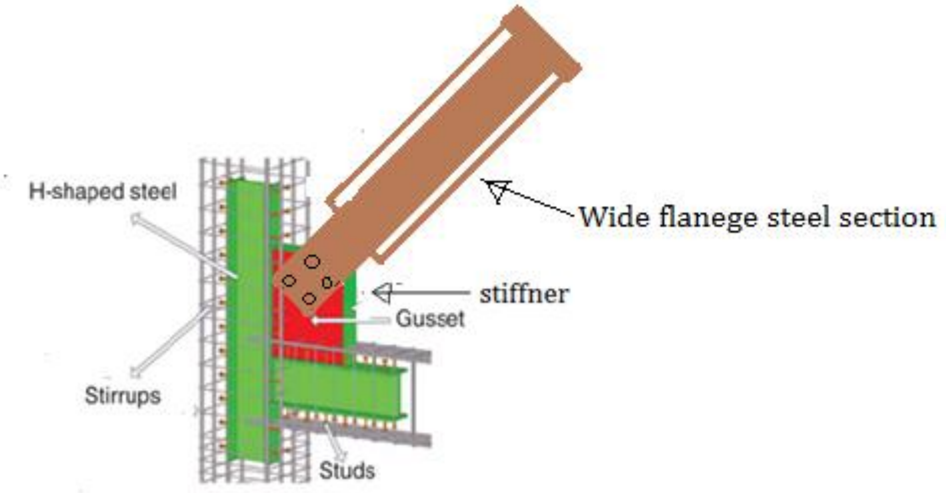


Figure 2.10: Wide flange Steel Bracing connection with RC frame.

Engineers must select brace types from a range of material and shapes. From a design point of view, braces are selected based on their compression strength. As such, HSS tend to have advantages from an economic point of view. However, the fracture life of wide-flange braces and of some other shapes is somewhat greater [16].

An additional advantage of wide-flange A992 material is that, it is currently better controlled in terms of expected strength.
Note that: HSS is Hollow steel section

2.8 Shear wall system

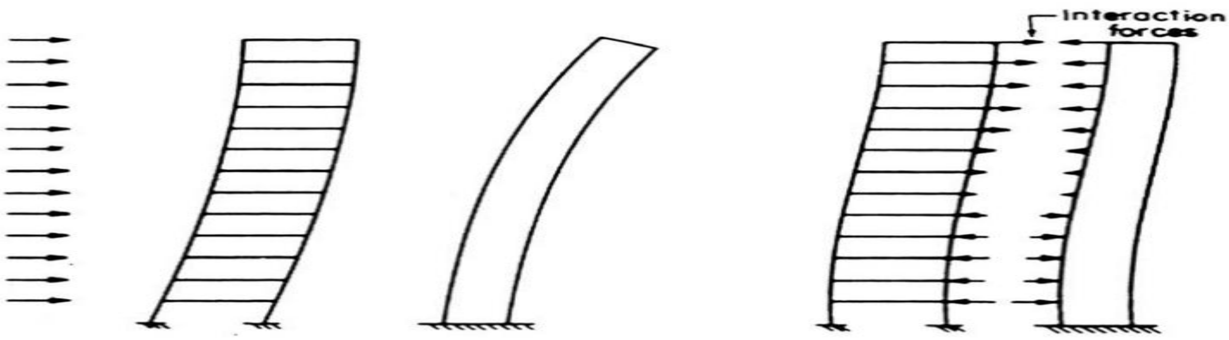
Reinforced concrete (RC) buildings often have vertical plate-like RC walls called Shear Walls in addition to slabs, beams and columns. These walls generally start at foundation level and are continuous throughout the building height.

Their thickness can be as low as 150mm, or as high as 400mm in high rise buildings [11]. Shear walls are like vertically-oriented wide beams that carry earthquake loads downwards to the foundation and usually provided along both length and width of buildings.

The walls provide large strength and stiffness to buildings in the direction of their orientation, mostly due to its large cross-section area that provide great moment of inertia, which significantly reduces lateral sway of the building.

When shear walls are combined with rigid frame the walls, which tend to deflect in a flexural configuration, and the frames, which tend to deflect in shear mode are constrained to adopt a common deflected shape by the horizontal rigidity of the girders and slabs.

And consequences, the walls and frames interact horizontally, especially at the top to produce a stiffer and stronger structure [18].



a) Rigid frame b) shear wall c) Interconnected frame and shear wall

Shear mode deformation bending mode deformation (equal deflection at each story level)

Figure 2.11: Shear wall frame combined with a rigid frame will decrease a buildings deflection

CHAPTER THREE

METHODS OF ANALYSIS

3.1 Methods of analysis

The lateral load analysis of this study is based on new code ESEN: 2015 which is the direct copy of Euro code 8 designs manual. As per new code ESEN: 2015 and Euro code 8, the horizontal design forces are defined from maximum acceleration of the structure, under the expected earthquake, that is represented with the acceleration spectrum of the structure. The starting point is an elastic response spectrum, which is reduced with factors that take into consideration the ability of structure to absorb seismic energy through rigid deformation. In the horizontal plane, the seismic action acts simultaneously and independently in two orthogonal directions that have the same response. Euro code suggests two different design spectrums.

a) Type 1 for High and moderate seismicity regions (distance EQ, $M_S > 5.5$) (southern Europe)
b) Type 2 for Low seismically active regions (local EQs <5.5) (central and northern Europe). and (NDP, recommended: PGA on rock ≤0.08g)

In this study, Type1 design spectrum was selected in order to notice the effect of earthquake on each bracing systems, and shear wall which gives maximum lateral displacement. In addition, there were also different parameters that are considered as an input for analysis. Of which behavior factor (q) is one factor that affect the analysis result.

Table 3.1: Values of the parameters describing the recommended type 1 elastic response spectra.

Ground type	S	TB(s)	TC(s)	TD(s)
A	1.0	0.15	0.4	2.0
B	1.2	0.15	0.5	2.0
C	1.15	0.20	0.6	2.0
D	1.35	0.20	0.8	2.0
E	1.4	0.15	0.5	2.0

Several methods can be used to analyze the response of a structure subjected to an earthquake. The choice of method depends on the structure and on the objectives of the analysis. The followings are the methods used to analyze the response of the structure:

1. Lateral force method of analysis/Equivalent static analysis /linear static
2. Modal response spectrum analysis/linear dynamic analysis
3. Pushover analysis/Non-linear static
4. Time history Analysis/Non-linear dynamic

3.1.1 Lateral force method of analysis/Equivalent static analysis (ESA)

Lateral force method of analysis is applied to buildings whose response is not significantly affected by contributions from modes of vibration higher than the fundamental mode in each principal direction. The response is read from a design response spectrum, given the natural frequency of the structure. This method works well for low to medium-rise buildings without significant coupled lateral–torsional modes, in which only the first mode in each direction is of significance. The above requirement is deemed to be satisfied in buildings which fulfill both of the following two conditions.

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

$$T_1 = \begin{cases} 4T_C \\ 2.0s \end{cases} \qquad \text{3.1}$$

Where, Tc is given in Table 3-1 for type 1 elastic response spectra

b) They meet the criteria for regularity in elevation.

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

$$T_1 = C_t . H^{\frac{3}{4}} \qquad \text{3.2}$$

Where, Ct = 0.085 for moment resistant space steel frames,

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

Ct =0.050 for all other structures; H is the height of the building, in m from basement.

3.1.2 Modal response spectrum analysis (RSA)

The seismic forces strikes the foundation of a structure will move with the ground motion. It shows that structure movement is generally more than the ground motion. The movement of the structure as compared to the ground is refused as the dynamic amplification. It depends on the natural frequency of vibration, damping, type of foundation, method of detailing of the structure. The response “design acceleration spectrum” which refers to the max acceleration called spectral acceleration coefficient Sa/g, as a function of the structure for a specified damping ratio for earthquake excitation at the base for a single degree freedom system.

In this method takes into account the following important engineering properties of the structures.

i. The fundamental natural period of vibration of the building (T in seconds)
ii. The damping properties of the structure
iii. Importance factor of the building
iv. The ductility of the structure represented by response reduction factor.

The sum of the effective modal masses for the modes taken into account amounts to at least 90% of the total mass of the structure. And the minimum number k of modes to be taken into account in a spatial analysis should satisfy both the two following conditions [9]:

$$K \geq 3.\sqrt{n} \quad \text{...3.3}$$

$$\text{And} \quad T_k \leq 0.20\ s \text{..3.4}$$

Where

k is the number of modes taken into account;

n is the number of storey’s above the foundation or the top of a rigid basement;

Tk is the period of vibration of mode k.

Either Modal Response Spectrum Analysis or Seismic Response History procedures are required for structures over 160 feet (48m) in height with specific types of structural irregularities or with long periods. Since the total building height taken in this thesis is 120m, Modal Response spectrum method is used

3.1.3 Pushover analysis

The 'Pushover' analysis is a non-linear static analysis carried out under constant gravity loads and monotonically increasing horizontal loads. It is applied essentially:

i. To verify or revise the over strength ratio values $\frac{\alpha u}{\alpha 1}$
ii. To estimate the expected plastic mechanisms and the distribution of damage
iii. To assess the structural performance of existing or retrofitted buildings

3.1.4 Non-linear time-history analysis (THA)

Non-linear time-history analysis is a dynamic analysis obtained through direct numerical integration of the differential equations of motion. The earthquake action is represented by accelerograms (minimum 3). This type of analysis is used for research and code background studies.

3.2 Behaviour factors according to ESEN 1998: 2015

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

The upper limit value of the behaviour factor q (response reduction factor), to account for energy dissipation capacity, shall be derived for each design direction as follows:

$$q = q_o K_w \geq 1.5 \quad \text{..3.5}$$

Where

q_o = the basic value of the behaviour factor, dependent on the type of the structural system and on its regularity in elevation.

K_w = the factor reflecting the prevailing failure mode in structural systems with walls

For buildings that are regular in elevation, the basic values of q_o for the various structural types are given in the following table

Table 3.2: Basic Value of the Behaviour Factor, q_o, for Systems Regular in Elevation

Lateral-load resisting structural system	DCM	DCH
Inverted pendulum system*	1.5	2
Torsionally flexible structural system **	2	3
Uncoupled wall system(>65% of seismic base shear resisted by walls, more than half by uncoupled walls) not belonging in one of the categories above	3	$4\left(\frac{\alpha u}{\alpha 1}\right)$
Frame system, dual system, coupled wall system and Any structural system other than those above	$3\left(\frac{\alpha u}{\alpha 1}\right)$	$4.5\left(\frac{\alpha u}{\alpha 1}\right)$

*: at least 50%of total mass in upper – third of the height, or with energy dissipation at base of a single element (except one –storey frame W/ all columns connected at the top via beams in both horizontal directions in plan and with max. value of normalized axial load V_d in combination (s) of the design seismic action with the concurrent gravity loads ≤ 0.3)

**: at any floor: radius of gyration of floor mass > torsional radius in one or both main horizontal directions (sensitive to torsional response about vertical).

When the multiplication factor $\frac{\alpha u}{\alpha 1}$ has not been evaluated through an explicit calculation, for buildings which are regular in plan the following approximate values of $\frac{\alpha u}{\alpha 1}$ may be used.

Table 3.3: Multiplication Factor αu/α1 According to Euro code 8

Frame or frame-equivalent dual systems	$\frac{\alpha u}{\alpha 1}$ EC8
One-storey buildings	1.10
multistory, one-bay frames	1.20
multistory, multi-bay frames or frame-equivalent dual structures	1.30

Table 3-4: Multiplication Factor αu/α1 According to Euro code 8

Wall or wall-equivalent dual systems	$\frac{\alpha u}{\alpha 1}$ EC8
Wall systems with only two uncoupled walls per direction	1.00
Other uncoupled wall systems	1.10
Wall-equivalent dual or coupled wall system	1.20

Normally αu and $\alpha 1$from base shear – top displacement curve from pushover analysis.

- αu:: Seismic action at development of global mechanism.
- $\alpha 1$:: Seismic action at 1st flexural yielding anywhere.

$\frac{\alpha u}{\alpha 1} \leq 1.5$

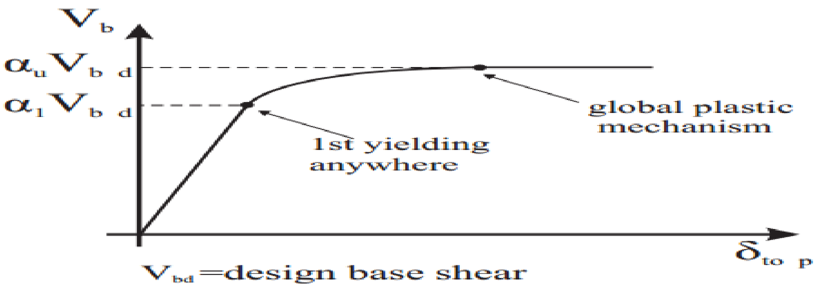


Figure 3.1: Design base shear versus displacement

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

$$K_w = \begin{cases} 1.00, & \text{for frame and frame} - \text{equivalent dual systems} \\ 0.5 \leq \frac{1+\alpha_o}{3} \leq 1, & \text{for wall} - \text{eqquivalent and torsionally flexible systems} \end{cases} \quad \ldots..3.6$$

Where

α_o **:** The prevailing aspect ratio of the walls of the structural system.

$$\alpha_o = (\sum h_{wi}) / \sum l_{wi} \quad \ldots\ldots 3.7$$

Where

h_{wi} is the height of wall i; and

l_{wi} is the length of the section of wall i.

For buildings which are not regular in elevation, the value of q_o should be reduced by 20%.

3.3 Importance classes for buildings according to ESEN 1998: 2015

Buildings are classified in 4 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 3.5: Importance class for buildings

Importance class	Buildings	**γI**
I	Buildings of minor importance for public safety, e.g. Agricultural buildings, etc.	0.8
II	Ordinary buildings, not belonging in the other categories.	1.0
III	Buildings whose seismic resistance is of importance in view of the consequence associated with a collapse, e. g, schools, assembly halls, cultural institutions etc.	1.2
IV	Buildings whose integrity during earthquakes is of vital importance for civil protection, e.g. hospitals, fire stations, power plants, etc.	1.4

3.4 Seismic mass according to ESEN 1998: 2015and ESEN 1990:2015

As the periods T are function of the masses M, a correct evaluation of the masses present in a structure at the time of the earthquake is necessary. A 'seismic mass' is defined, based on a weight W calculated as:

$$W = \sum G_{K,} + \sum \psi_{E,i} \,.\, Q_K \ldots\ldots\ldots\ldots\ldots\ldots 3.8$$

The coefficient $\psi_{E,i}$ is used to estimate a likely value of service loads and to take into account that some masses do not follow perfectly the moves of the structure, because they are not rigidly connected to the structure. $\psi_{E,i}$ is computed as:

$$\psi_E = \varphi . \psi_{2,i} = 0.5 * 0.6 = 0.3 \text{ (Independently occupied storey's)}$$

Therefore,

$$W = \sum G_{K,} + 0.3 * \sum \psi_{E,i}$$

Values of $\psi_{2,i}$ and φ are listed in Table 3-6. It can be noticed that the coefficient $\psi_{E,i}$ which is used to define the mass of the service load present on average over the building height can be much lower than 1.

The seismic mass is used to determine:

i. The global effects due to an earthquake at a given level of the structure, in particular at the foundations
ii. The forces Ptot and Vtot used in the verification of limitation of second order effects
iii. The seismic action effects AEd generated in the structural elements by the earthquake; for the resistance checks of these elements values of AEd are combined to the other action's effects in order to establish the design value of the action effect Ed :

Table 3.6: coefficients $\psi_{2,i}$ and φ

Specific Use	$\psi_{2,i}$	Storey	φ
Cat. A : residence	0.3	Roof	1.0
Cat. B:office	0.3	Storey's with correlated occupancies	0.8
Cat. C : meeting rooms, places where people congregate	0.6	Independently occupied storey's	0.5
Cat. D: shopping area	0.6		1.0
Cat. E: storage, accumulation of goods	0.8		
Cat. F:traffic(vehicle< 30kN)	0.6		

Where:

$G_{K,}$: Characteristics value of permanent action

Q_K: Characteristics value of variable action

$\psi_{E,i}$: Combination coefficient for a variable action I to be used when determining the effects of the design seismic action

$\psi_{2,i}$: Combination coefficient for the quasi-permanent value of a variable action

CHAPTER-FOUR

MODELING OF STRUCTURAL SYSTEMS

4.1 Structural modeling

For the analysis work, the models of high rise reinforced concrete frame building (40) floors were made to know the realistic behavior of building during earthquake. The length of the model building is 48m and width is 36m. Height of typical story is 3 m. Column sizes changes at each 10 story. Generally the following assumptions were taken.

1. Modal damping 5% is considered.
2. Beams and columns are modeled as frame element and joined node to nodes. While shear walls were represented by shell-type element,
3. The effect of soil structure interaction is ignored in analysis. The columns are assumed to be fixed at the ground level.
4. Plan dimension, and beam size, are kept similar to all Storey
5. Bracing is represented by a section of steel
6. Beam column joints are taken as rigid joints
7. The same location of both bracings and shear walls are taken, to have the better seismic performance comparison
8. Shear wall is continues and the same dimension throughout the height of the frames
9. Participating Components: Only the primary structural components are assumed to participate in the overall behavior. The effects of secondary structural components and nonstructural components are assumed to be negligible; these include staircases, partitions, cladding, and openings.
10. AISC 341 allows two beam-to-column connection types for CBFs: simple connections and a moment-resisting connection comparable to those used for Ordinary Moment Frames. For the former, CBFs are usually modeled as trusses with pin connections assumed in both planes.

As such, the stiffness offered by the gusset plates to the girders or columns at the brace connections are largely ignored under a presumption that they will yield relatively early during the seismic excitation. In this paper the connection is assumed to be pin

11. Loading:

i. Gravity Loads: The building self weight and slab weight is considered. But, partition loads are not considered

- ❖ Slab weight

Table 4.1: gravity load data

Components included in the slab weight	Unit weight (γ) (kN/m^3)	Thickness (mm)	Force per area(kN/m^2)
Marble	27	2	0.54
Cement screed	22	3	0.66
Soffit plaster	22	2	0.44
Slab	25	150	3.75
		Total area load	5.39

ii. Live load is taken as 4kN/m² (for shopping areas).

iii. Seismic loading :

Table 4.2: Earthquake data

Earthquake data	
Seismic Zone	V
Bedrock acceleration ratio ($\alpha_o = \frac{a_o}{g}$)(ratio of design bedrock acceleration to acceleration of gravity)	0.2g
Design PGA	γα_o=1*0.2g= 0.2g
Importance factor, I	1
Behavior factor, q	Depends on the structural system
Subsoil class	B

Table 4.3: Material properties used for analysis

Material properties	
Grade of concrete	C-40, as used in practical applications of tall buildings.
Poisons ratio of concrete	0.2
Density of reinforced concrete	$25kN/m^3$
Modulus of elasticity of concrete	35GPa for C-40
Coefficient of thermal expansion of concrete	$10*10^{-6}$ per ^oC
Grade of steel(rebar)	S-420
Grade of steel(wide flange-section for bracing)	S-450
Density of reinforcing steel	7850 kg/m^3
Coefficient of thermal expansion of steel	$10*10^{-6}$ per ^oC
Modulus of elasticity of steel	200GPa
Poisons ratio of steel	0.3

4.2 Studied structural configuration

Following two types of structural configuration is studied.

1. 40 storey reinforced concrete framed structure without bracing and shear wall (MRF)
2. 40 storey reinforced concrete framed structure with shear wall and different bracing patterns such as, X-brace, V-brace, and inverted V (chevron) brace

4.3 Details of the building plan, and member size

4.3.1 Building Plan

Plan of the reinforced concrete building which is used for the study is shown in figure below.

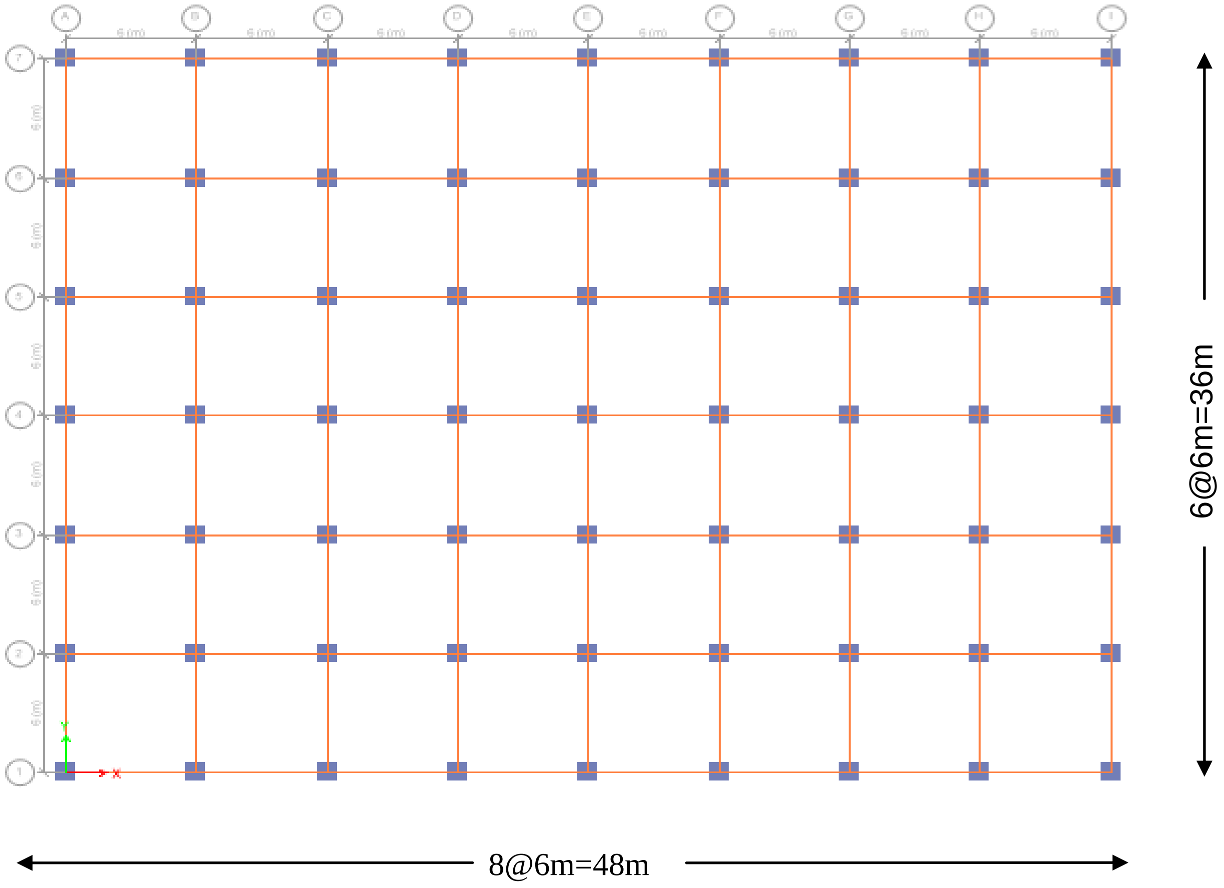


Figure 4.1: plan of the model

4.3.2 Member size of the beams and columns

Member size used for beams, columns and bracing are shown in Table 4.4 and Table 4.5.

Table 4.4: Size of Beams, and Columns

Storey level	Column schedule		Beam schedule	
	Column No	Size	Beam No	Size
1-5	C1	1200*1200	B1	300*600
6-10	C2	1100*1100		
11-15	C3	1000*1000		
16-20	C4	900*900		
21-25	C5	800*800		
26-30	C6	700*700		
31-45	C7	600*600		
36-40	C8	500*500		

4.3.3 Member size of bracing

Table 4.5: Size of Bracings

Storey level	Steel Bracing Schedule	
	Bracing No	Section dimension
1-10	BR1	Wide flange steel section 600*300*15
11-20	BR2	Wide flange steel section550*250*15
21-30	BR3	Wide flange steel section 500*200*15
31-40	BR4	Wide flange steel section 450*150*15

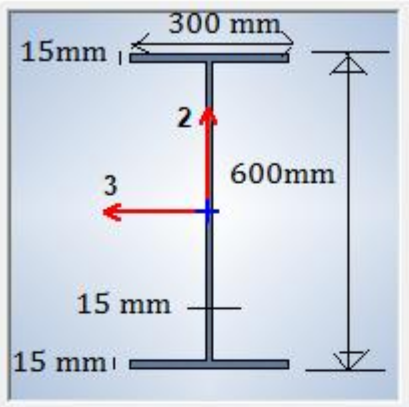
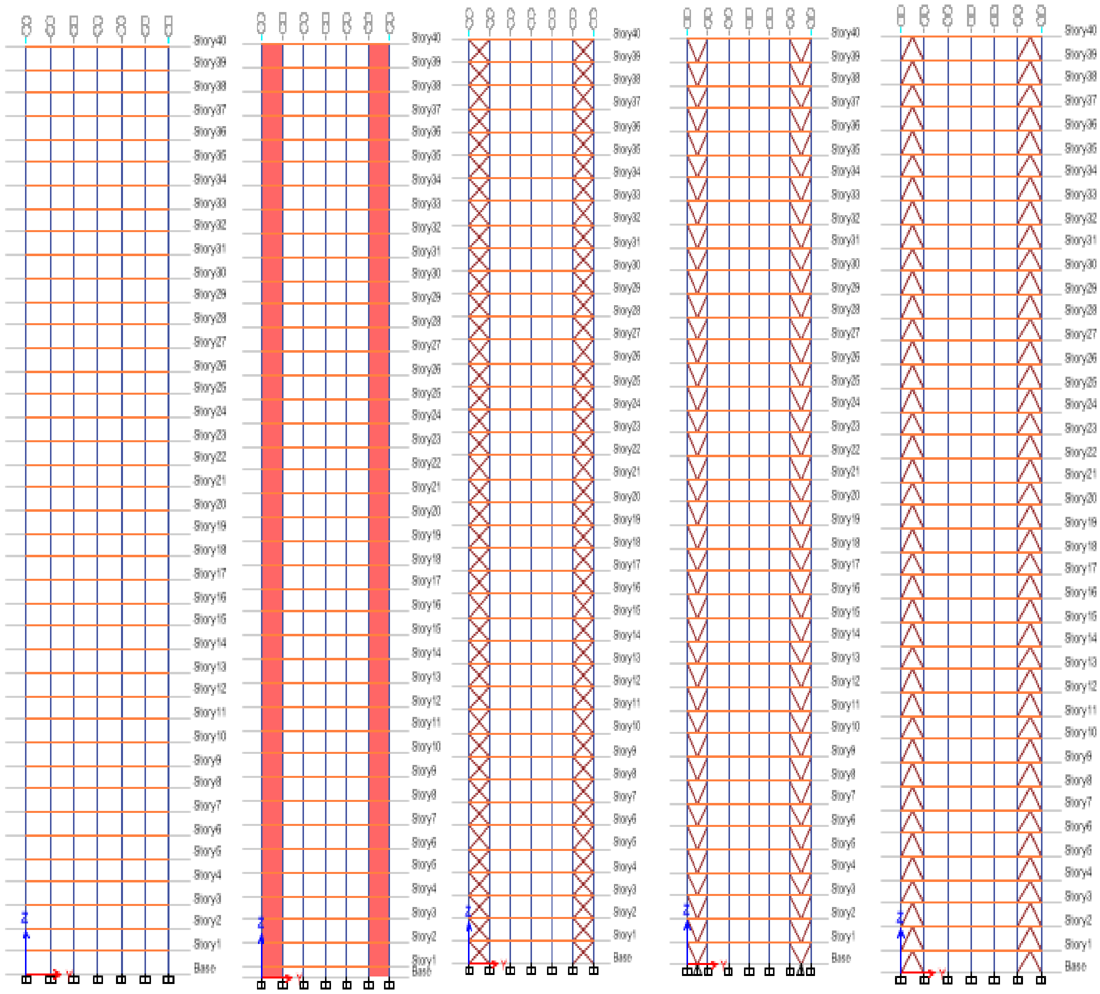


Figure 4.4: Sample wide flange steel section

4.4 Types and location of bracing patterns and shear wall used in the study

Different types of bracing pattern and shear walls used in the study are shown in figure below.

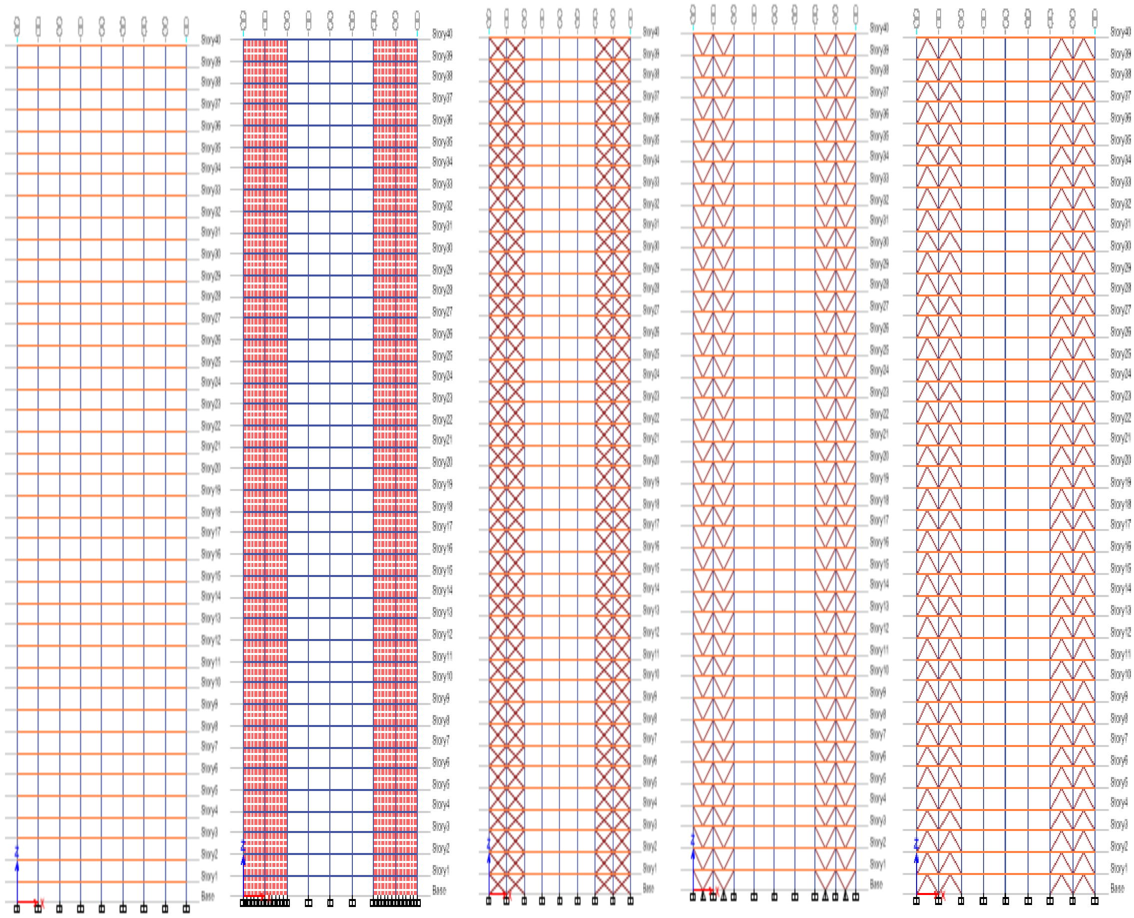
i. when the shear wall and bracing is provided in a single bay



A) Bare frame B) shear wall C) X- brace C) V-brace D) ʌ- brace

Figure 4.5: Concrete Framed Model of Building with bare frame, shear wall, X- brace, V-brace and, ʌ- brace for the whole modeling, when provided in a single bay

ii. when the shear wall and bracing is provided in a double bay



A) Bare frame B) shear wall C) X- brace C) V-brace D) ʌ- brace

Figure 4.6: Concrete Framed Model of Building with bare frame, shear wall, X- brace, V-brace and, ʌ- brace for the whole modeling, when provided in a double bay

4.4.1 Location of bracing patterns and shear wall used in the study

For equal treatment of the study, the location of all bracing as well as the shear wall is at the same place, which is at the corner of the frame.

i. When only single bay is braced and shear walled

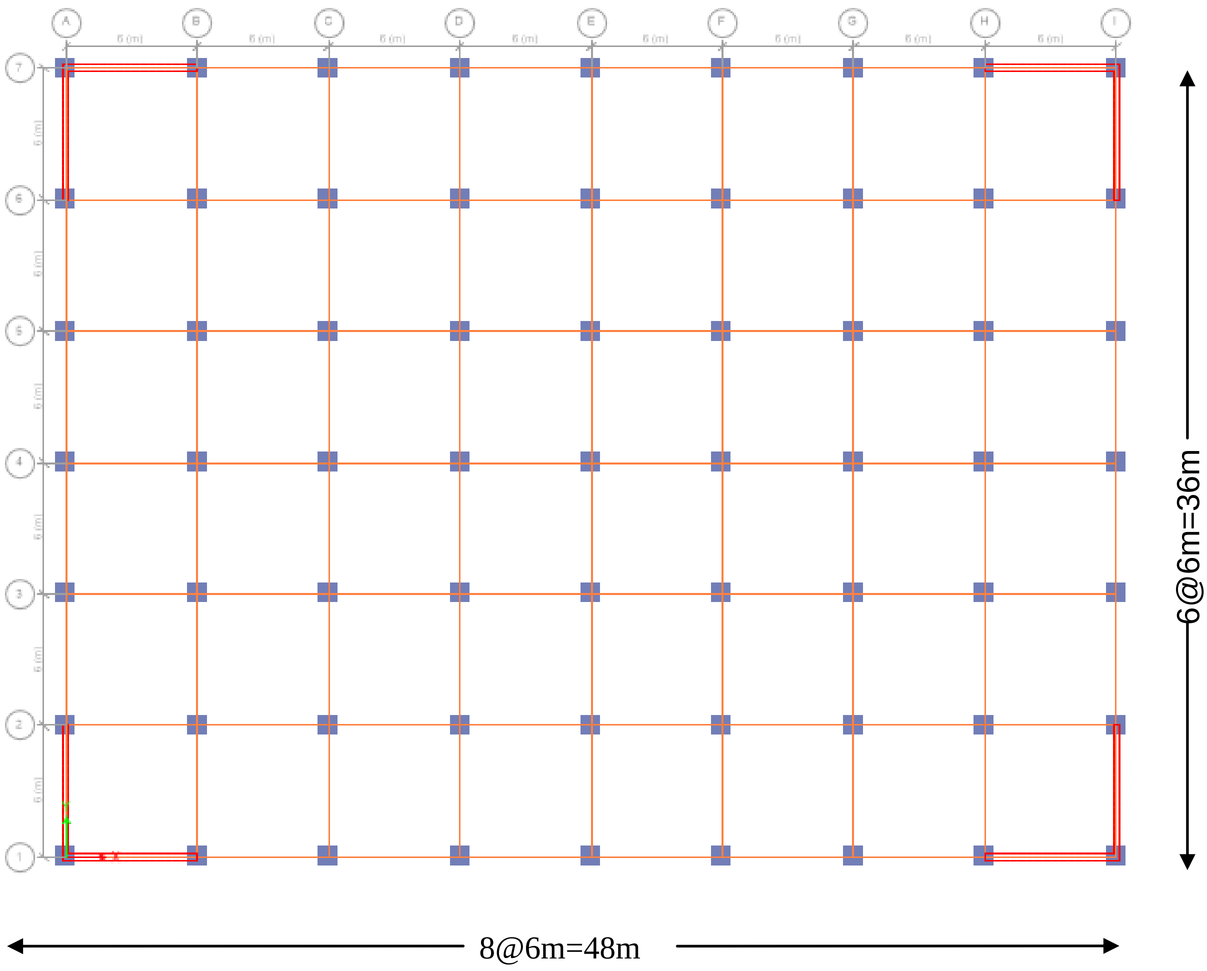


Figure 4.7: location of bracings and shear wall, when provided in a single bay

ii. When successive two bay is braced and shear walled

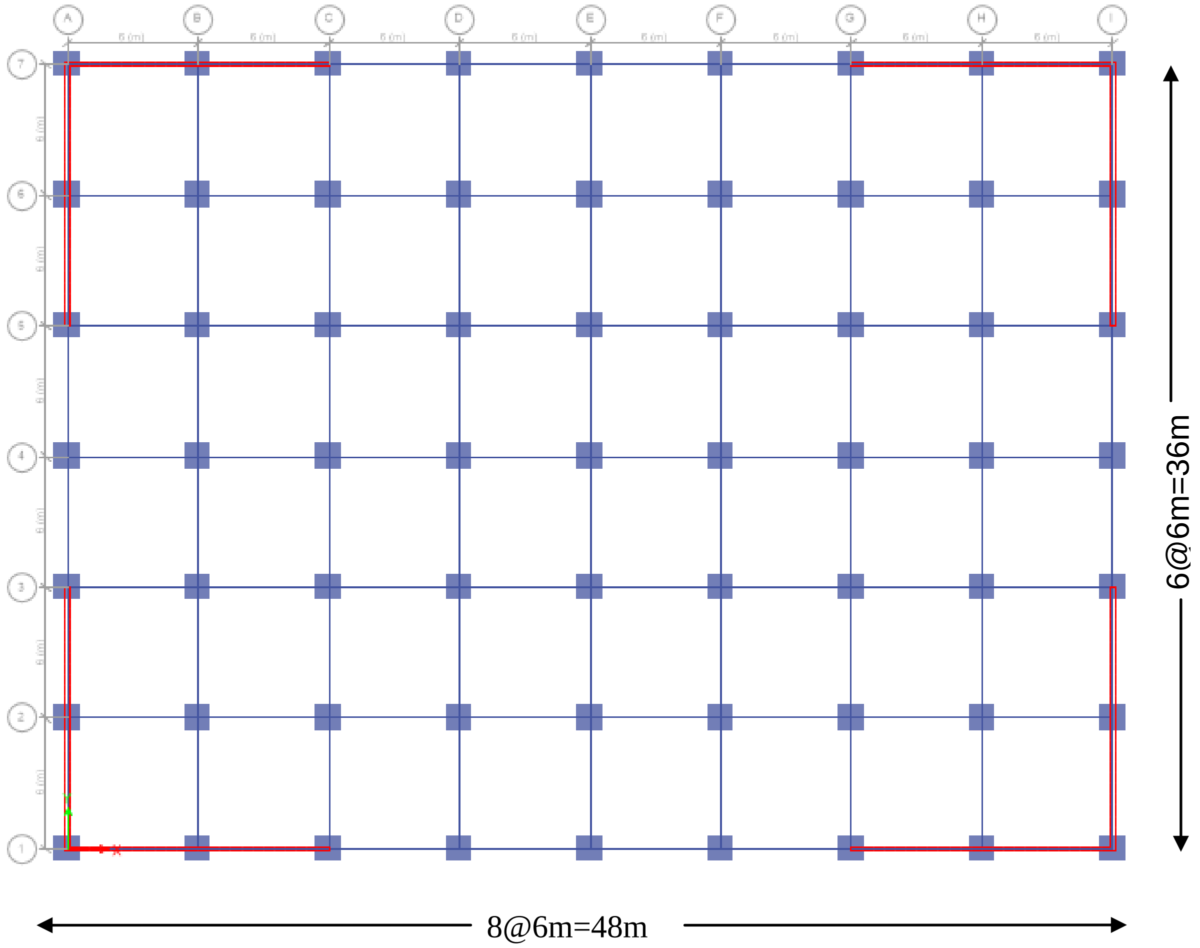


Figure 4.8: location of bracings and shear wall, when provided in a double bay

CHAPTER-FIVE

RESULTS AND DISCUSSION

Analyses were conducted to evaluate the performance of concrete structures under seismic loading with different bracing type and shear wall. Results of Response Spectrum Analysis have been used to observe and compare floor response of all the models in terms of the following parameters.

a) Modal time period
b) Storey displacements

The comparison of results in terms of the above parameters was given in terms of tables & graphs in the coming paragraph.

5.1 Modal time period

The time required to complete one complete cycle of vibration is called time period. Under free vibration the structure always vibrates in single mode called its fundamental mode and the corresponding time period is called fundamental period of the structure. The fundamental period is the longest period of the structure.

Table 5.1 Fundamental time Period when the shear wall and bracing is provided in a single bay

Types of structural system	Period (T) in second	Reduction in % with reference to bare frame
Bare frame	2.8	
X-bracing	2.51	10.36
V-bracing	2.53	9.64
Inverted V-bracing	2.508	10.43
Shear wall with thickness= 400mm	2.415	13.75
Shear wall with thickness= 300mm	2.41	13.93
Shear wall with thickness= 200mm	2.41	13.93

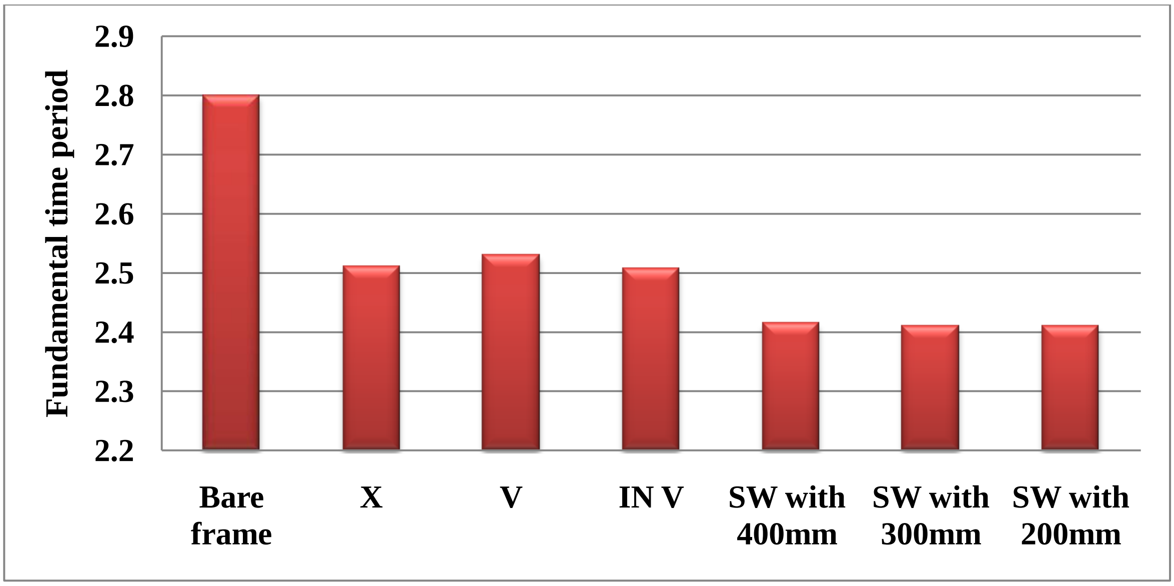


Figure 5.1: fundamental time period versus Types of structural system when the shear wall and bracing is provided in a single bay.

Table 5.2 Fundamental time Period when the shear wall and bracing is provided in a double bay.

Types of structural system	Period (T) in second	Reduction in % with reference to bare frame
Bare frame	2.8	
X-bracing	2.171	22.46
V-bracing	2.202	21.35
Inverted V-bracing	2.179	22.17
Shear wall with thickness= 400mm	1.871	33.17
Shear wall with thickness= 300mm	1.898	32.21
Shear wall with thickness= 200mm	1.932	31

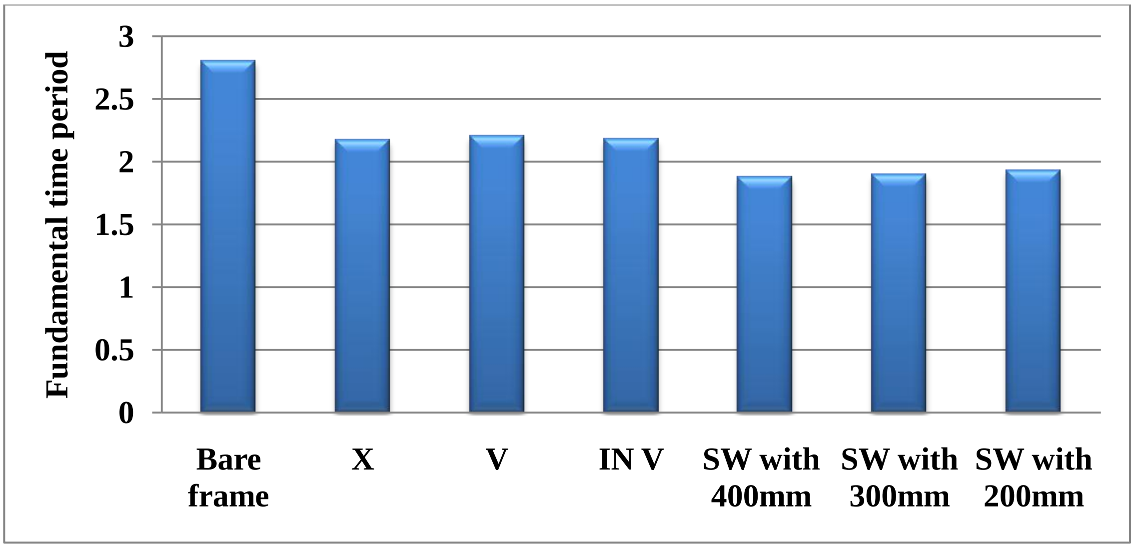


Figure 5.2: fundamental time period versus Types of structural system, when the shear wall and bracing is provided in a double bay.

From Table 5.2 with the addition of shear wall and bracing members the fundamental time period of the building decreases. From the building with, 200mm, 300mm and 400mm thickness shear walled frame, building with 400 mm thickness shear wall was given minimum fundamental time period of 1.871 seconds as compared to 2.8 seconds of bare frame building.

And from bracing type, building with the addition of X- bracing was given minimum time period of 2.171 seconds as compared to 2.8 seconds of bare frame building.

5.2 Storey displacements

Storey displacements depend upon the value of Storey shear at that Storey. Greater the value of Storey shear greater the value of Storey displacement & vice versa.

5.2.1 Storey displacement for shear walled frame and braced frame

i. when the shear wall and bracing is provided in a single bay

Table 5.3 Storey Displacement in X- direction with, 200,300, and 400 mm thickness shear walled frame and with X, V and inverted V bracing types. (When single bay is braced and shear walled).

		Bracing			Shear wall with different thickness		
Storey	Bare frame	X-bracing	V-bracing	IN V-bracing	SW with t=400mm	SW with t=300mm	SW with t=200mm
40	103.5	85.4	86.6	85.2	85.6	84.4	83.4
39	103	85	86.1	84.7	83.5	82.5	81.7
38	102.3	84.3	85.4	84.1	81.5	81	80.9
37	101.4	83.4	84.6	83.2	80.2	79.9	79.9
36	100.2	82.3	83.5	82.2	78.8	78.6	78.7
35	98.9	81.1	82.2	80.9	77.3	77.2	77.2
34	97.5	79.8	81	79.7	75.8	75.7	75.8
33	96	78.4	79.6	78.3	74.1	74.1	74.3
32	94.4	76.8	78	76.7	72.3	72.3	72.6
31	92.6	75.1	76.3	75.1	70.4	70.5	70.8
30	90.6	73.3	74.5	73.2	68.4	68.5	68.8
29	88.6	71.5	72.7	71.4	66.4	66.6	67
28	86.5	69.5	70.7	69.5	64.4	64.6	65
27	84.3	67.5	68.7	67.5	62.3	62.6	63
26	81.9	65.4	66.5	65.4	60.2	60.5	60.9
25	79.4	63.1	64.3	63.1	58	58.3	58.8
24	76.9	60.9	62	60.9	55.8	56.2	56.7
23	74.2	58.6	59.6	58.6	53.6	54	54.5
22	71.5	56.2	57.2	56.2	51.4	51.8	52.3
21	68.6	53.7	54.7	53.7	49.1	49.5	50.1
20	65.6	51.1	52.1	51.1	46.8	47.2	47.8
19	62.6	48.5	49.5	48.5	44.5	45	45.5
18	59.4	45.9	46.8	45.9	42.2	42.6	43.1
17	56.2	43.2	44.1	43.2	39.9	40.3	40.8
16	52.8	40.5	41.3	40.5	37.5	37.9	38.3
15	49.4	37.7	38.4	37.6	35.1	35.4	35.9
14	45.9	34.8	35.5	34.8	32.6	33	33.4
13	42.3	32	32.6	32	30.1	30.5	30.9
12	38.7	29.1	29.7	29.1	27.6	27.9	28.3
11	34.9	26.2	26.7	26.2	25	25.3	25.6
10	31.1	23.3	23.7	23.2	22.4	22.6	22.9
9	27.3	20.3	20.7	20.3	19.7	19.9	20.2
8	23.4	17.4	17.7	17.3	17	17.2	17.4
7	19.6	14.5	14.7	14.4	14.3	14.5	14.6
6	15.8	11.6	11.8	11.6	11.6	11.7	11.9
5	12.1	8.9	9	8.8	8.9	9	9.1
4	8.6	6.3	6.4	6.3	6.4	6.4	6.5
3	5.4	4	4	4	4	4.1	4.1
2	2.7	2	2	2	2	2.1	2.1
1	0.8	0.6	0.7	0.6	0.6	0.6	0.6
Base	0	0	0	0	0	0	0

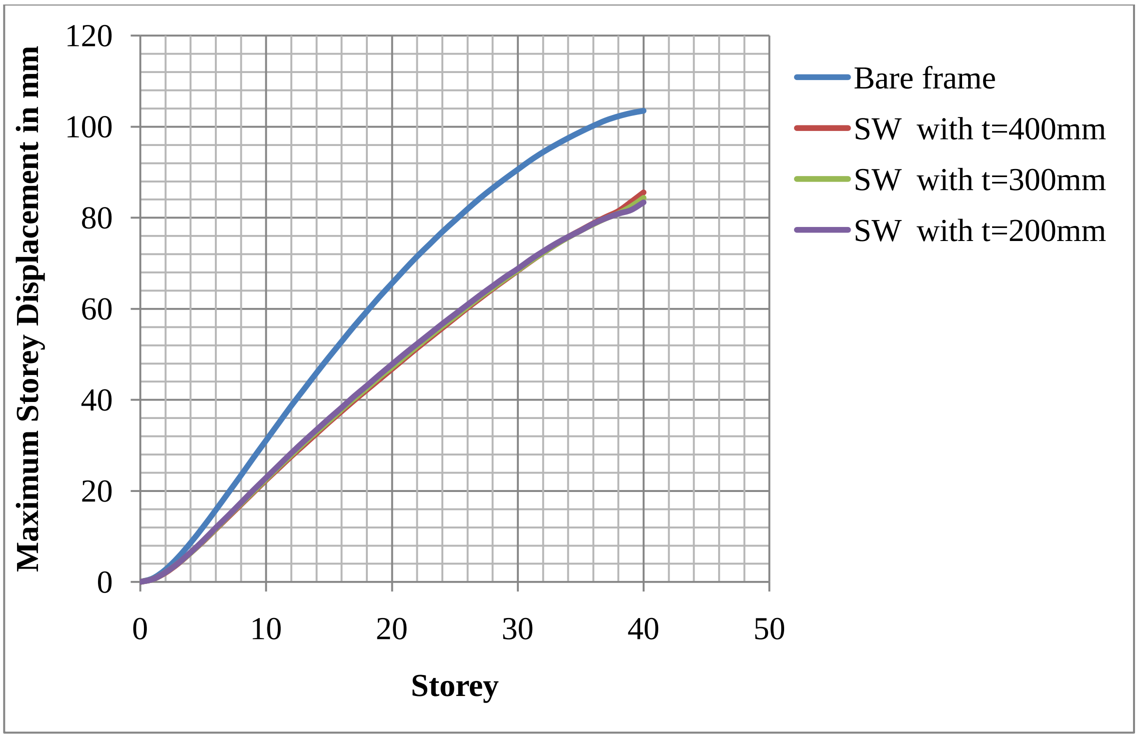


Figure 5.3: Maximum Storey Displacement in X- direction with 200,300, and 400 mm thickness shear walled frame with reference to bare frame. (When single bay is shear walled).

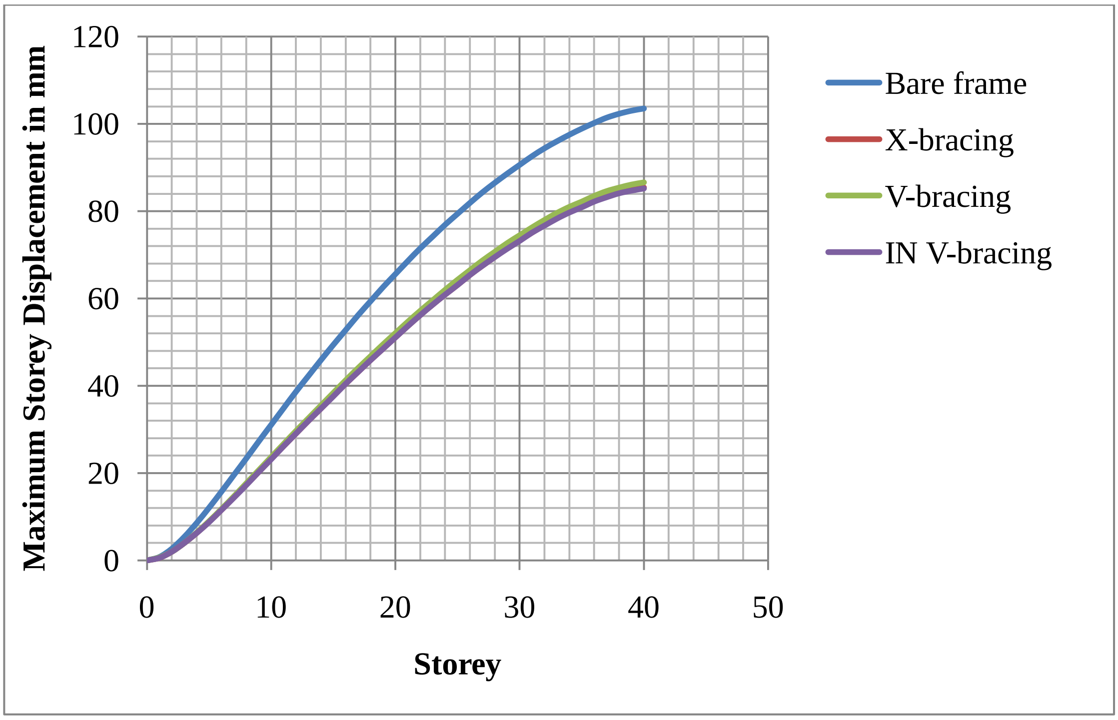


Figure 5.4: Maximum Storey Displacement in X- direction with X, V and inverted V bracing type. (When single bay is braced).

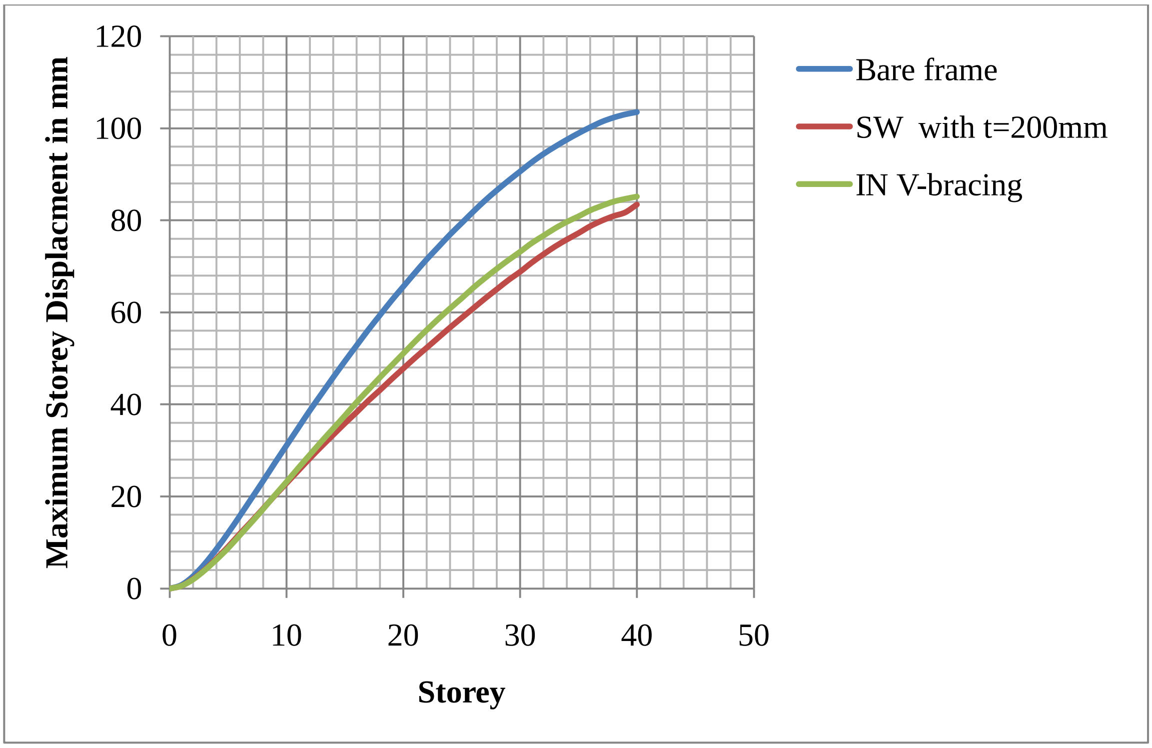


Figure 5.5: Maximum Storey displacement in X- direction for 200 mm shear walled frame and inverted V- braced frame. (When single bay is braced and shear walled).

As shown in Table 5.3, the storey displacement was higher when no shear wall and bracing members were provided, but when shear wall and bracing member were provided then Storey displacement decreases. By comparison, the shear wall with thickness 300mm showed better reduction in storey displacement as compared to 200mm, and 400mm thickness shear wall, but the last three storey, 200mm thickness shear walled frame showed insignificantly higher reduction in storey displacement. The maximum reduction in storey displacement of 84.4 mm was observed for building with 300mm thickness shear wall when compared with 103.5mm for bare frame at the 40th storey. So there was 18.45% reduction for this case.

Similarly from braced frame, inverted-V braced frame showed the maximum reduction in storey displacement as compared to X and V- braced frame. The maximum storey displacement of 85.2 mm was observed for building with inverted V bracing when compared with 103.5mm for bare frame. So there was 17.68% reduction for this case.

Therefore, from braced frame inverted V - braced frame showed the maximum reduction in storey displacement as compared to X and V- braced frame, and from shear walled frame 300 mm thickness shear walled frame showed the maximum reduction in storey displacement as compared to 200mm, and 400mm thickness shear walled frame, therefore when inverted V- braced frame was compared to 300 mm thickness shear walled frame, 300 mm thickness shear walled frame showed the maximum reduction in storey displacement than inverted V- braced frame with the whole stories.

Generally the following table elaborates the reduction of storey displacement in percentage.

Table 5.4 Reduction of top Storey displacement in % in X-direction. (When single bay is braced and shear walled).

Model	Reduction of maximum Storey displacement in % at the 40^{th} storey
X-bracing	17.48
V-bracing	16.33
In V-bracing	17.68
SW with t=400mm	17.29
SW with t=300mm	18.45
SW with t=200mm	19.42

Table 5.5 Storey Displacement in Y- direction with 200,300, and 400 mm thickness shear walled frame and with X, V and inverted V bracing types. (When single bay is braced and shear walled).

		Bracing			Shear wall with different thickness		
Storey	Bare frame	X-bracing	V-bracing	IN V-bracing	SW with t=400mm	SW with t=300mm	SW with t=200mm
40	109.6	91.3	92.3	91	89.8	88.4	88.4
39	109	90.7	91.8	90.5	87.9	87.7	87.8
38	108.2	90	91	89.7	87	86.9	87
37	107.1	89	90.1	88.8	85.9	85.8	85.9
36	105.8	87.9	89	87.7	84.6	84.5	84.7
35	104.3	86.5	87.6	86.4	83.1	83	83.2
34	102.8	85.2	86.3	85	81.5	81.6	81.8
33	101.2	83.7	84.8	83.6	79.9	80	80.2
32	99.3	82.1	83.2	82	78.1	78.2	78.5
31	97.4	80.3	81.4	80.2	76.2	76.4	76.7
30	95.2	78.4	79.5	78.3	74.2	74.4	74.7
29	93.1	76.5	77.6	76.4	72.3	72.5	72.8
28	90.8	74.5	75.5	74.4	70.3	70.5	70.9
27	88.4	72.4	73.4	72.3	68.2	68.4	68.8
26	85.9	70.1	71.1	70.1	66	66.3	66.7
25	83.2	67.8	68.8	67.7	63.8	64.1	64.5
24	80.5	65.4	66.4	65.4	61.5	61.8	62.3
23	77.7	63	63.9	62.9	59.2	59.6	60
22	74.7	60.4	61.3	60.4	56.9	57.2	57.7
21	71.7	57.8	58.7	57.8	54.5	54.8	55.3
20	68.5	55.1	55.9	55	52	52.4	52.8
19	65.2	52.4	53.1	52.3	49.5	49.9	50.3
18	61.9	49.5	50.3	49.5	46.9	47.3	47.8
17	58.5	46.7	47.4	46.6	44.3	44.7	45.2
16	55	43.7	44.4	43.7	41.6	42	42.5
15	51.3	40.7	41.3	40.7	38.9	39.3	39.7
14	47.7	37.7	38.2	37.6	36.1	36.5	36.9
13	43.9	34.6	35.1	34.5	33.3	33.7	34.1
12	40.1	31.5	31.9	31.4	30.4	30.8	31.2
11	36.2	28.3	28.7	28.2	27.5	27.8	28.2
10	32.2	25.1	25.5	25.1	24.5	24.8	25.2
9	28.2	21.9	22.2	21.9	21.5	21.8	22.1
8	24.2	18.8	19	18.7	18.5	18.8	19
7	20.2	15.6	15.8	15.5	15.5	15.7	15.9
6	16.2	12.5	12.7	12.4	12.5	12.7	12.8
5	12.4	9.5	9.6	9.5	9.6	9.7	9.8
4	8.8	6.8	6.8	6.7	6.8	6.9	7
3	5.6	4.2	4.3	4.2	4.3	4.4	4.4
2	2.8	2.1	2.2	2.1	2.2	2.2	2.2
1	0.8	0.6	0.7	0.6	0.6	0.6	0.6
Base	0	0	0	0	0	0	0

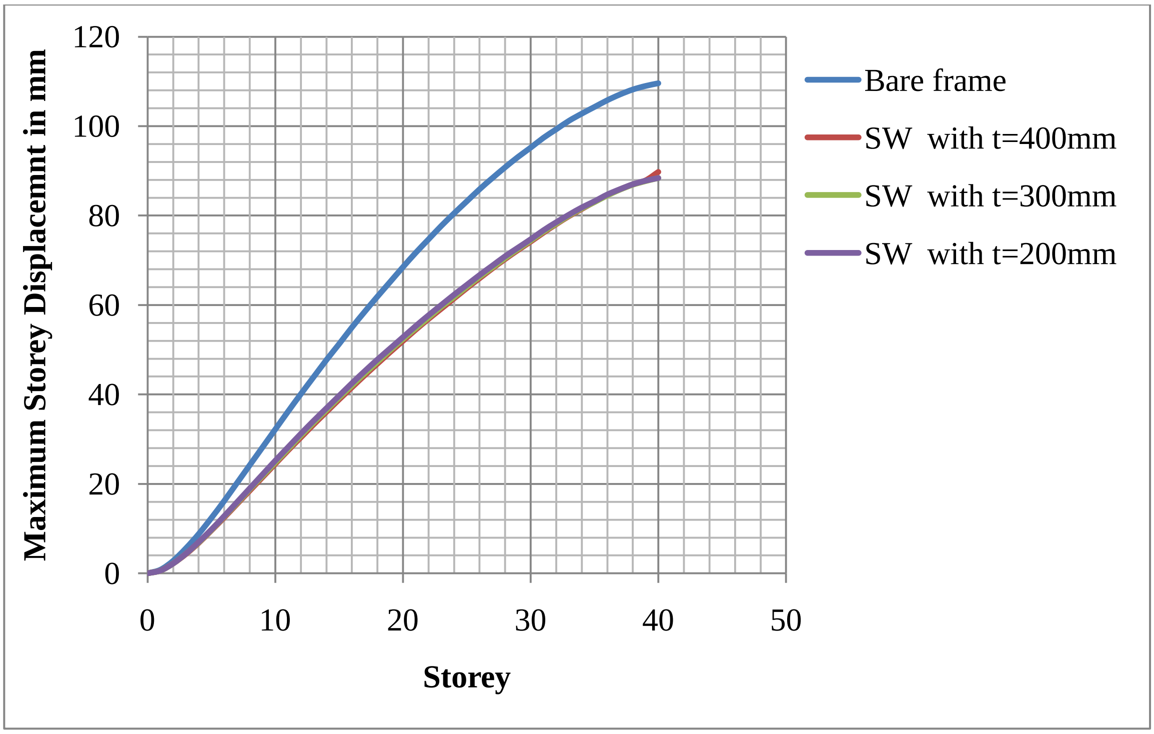


Figure 5.6: Maximum Storey Displacement in Y- direction with 200,300, and 400 mm thickness shear walled frame with reference to bare frame. (When single bay is shear walled).

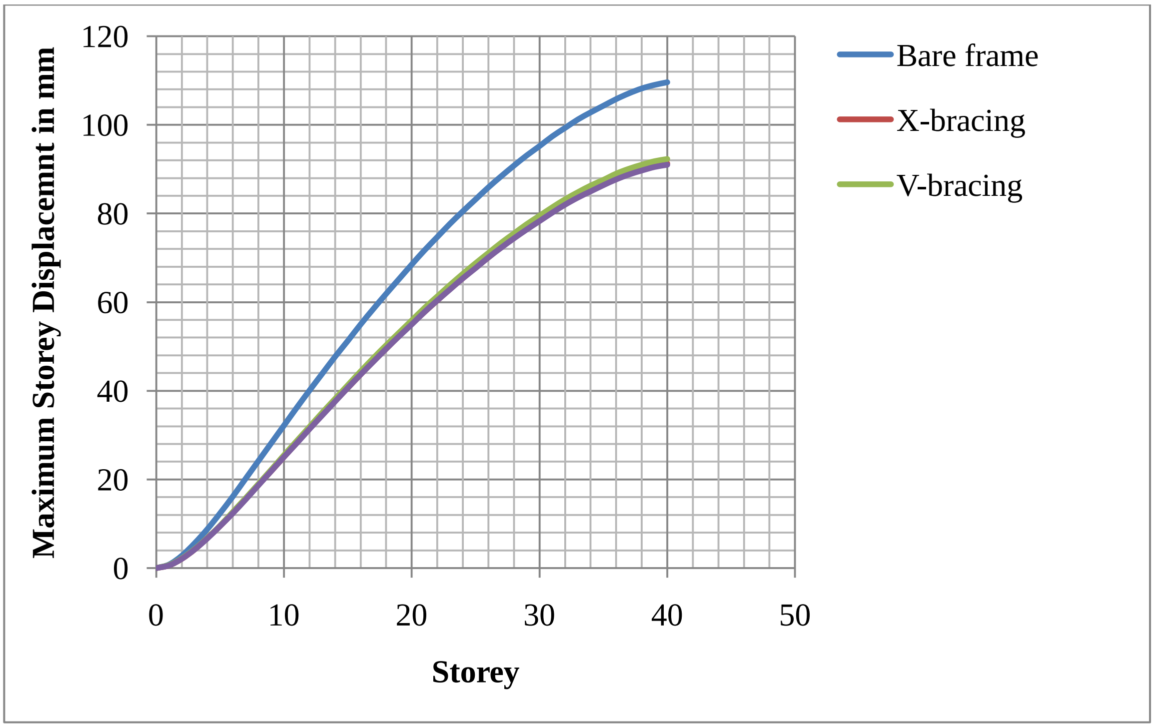


Figure 5.7: Storey Displacement in Y- direction with X, V and inverted V bracing type. (When single bay is braced).

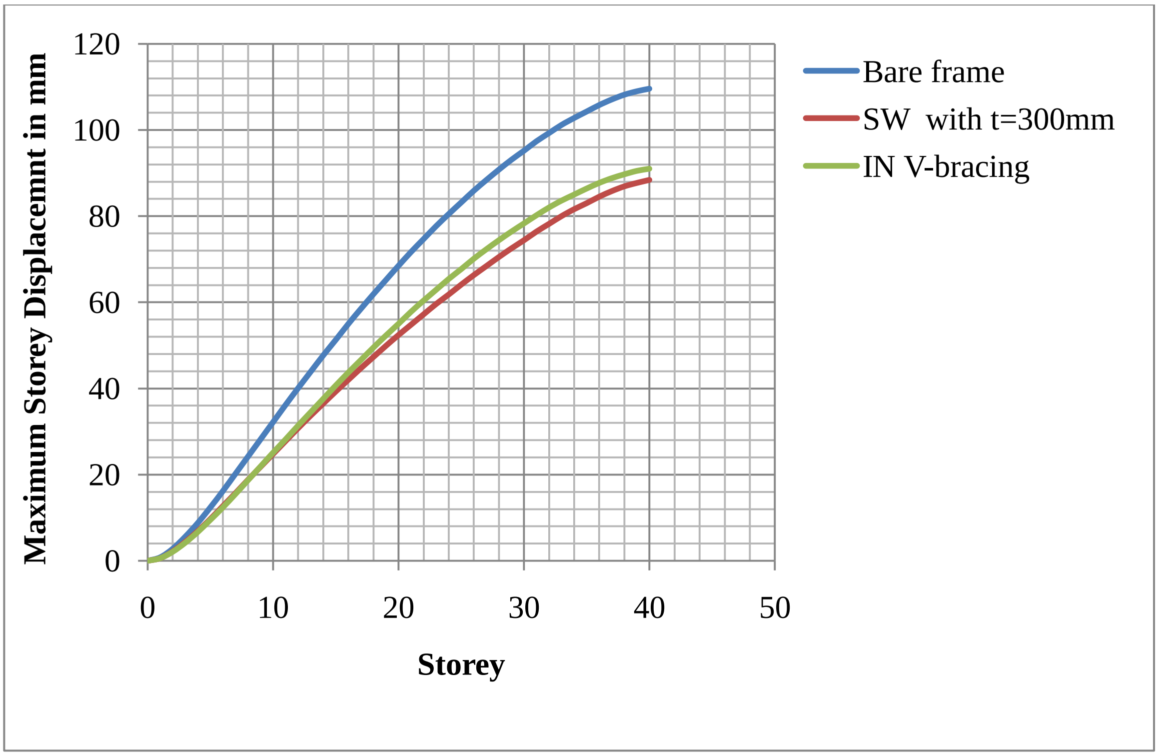


Figure 5.8: Storey displacement in Y- direction for 300 mm shear walled frame and inverted V- braced frame. (When single bay is braced and shear walled).

As shown in Table 5.5, the storey displacement was higher when no shear wall and bracing members were provided, but when shear wall and bracing member were provided then Storey displacement decreases. By comparison, the shear wall with thickness 300mm showed better reduction in storey displacement as compared to 200mm, and 400mm thickness shear wall. The maximum reduction in storey displacement of 88.4 mm was observed for building with 300mm thickness shear wall when compared with 109.6mm for bare frame at the 40th storey. So there was 19.34% reduction for this case.

Similarly from braced frame, inverted-V braced frame showed the maximum reduction in storey displacement as compared to X and V- braced frame. The maximum storey displacement of 91 mm was observed for building with inverted V bracing when compared with 109.6mm for bare frame. So there was 16.97% reduction for this case.

Therefore, from braced frame inverted V - braced frame showed the maximum reduction in storey displacement as compared to X and V- braced frame, and from shear walled frame 300 mm thickness shear walled frame showed the maximum reduction in storey displacement as compared to 200mm, and 400mm thickness shear walled frame, therefore when inverted V- braced frame was compared to 300 mm thickness shear walled frame, 300 mm thickness shear walled frame showed the maximum reduction in storey displacement than inverted V- braced frame with the whole stories.

Generally the following table elaborates the reduction of storey displacement in percentage.

Table 5.6 Reduction of top Storey displacement in % in Y-direction. (When single bay is braced and shear walled).

Model	Reduction of maximum Storey displacement in % at the 40^{th} storey
X-bracing	16.69
V-bracing	15.78
In V-bracing	16.97
SW with t=400mm	18.06
SW with t=300mm	19.34
SW with t=200mm	19.34

ii. *when the shear wall and bracing is provided in a double bay*

Table 5.7 Storey Displacement in X direction with 200,300, and 400 mm thickness shear walled frame and with X, V and inverted V bracing types. (When a double bay is braced and shear walled).

		Bracing			Shear wall with different thickness		
Storey	Bare frame	X-bracing	V-bracing	IN V-bracing	SW with t=400mm	SW with t=300mm	SW with t=200mm
40	103.5	66.1	67.7	66.3	72.5	73.1	73.7
39	103	65.6	67.3	65.8	71.9	72.5	73.1
38	102.3	65	66.7	65.2	71.1	71.6	72.2
37	101.4	64.2	65.9	64.5	70	70.6	71.2
36	100.2	63.3	65	63.6	68.7	69.3	70
35	98.9	62.2	63.9	62.5	67.2	67.9	68.6
34	97.5	61.1	62.8	61.5	65.8	66.5	67.2
33	96	60	61.6	60.3	64.2	65	65.7
32	94.4	58.7	60.4	59.1	62.6	63.3	64.1
31	92.6	57.3	59	57.7	60.8	61.6	62.4
30	90.6	55.8	57.5	56.2	59	59.8	60.6
29	88.6	54.4	56	54.8	57.2	58.1	58.9
28	86.5	52.8	54.4	53.3	55.4	56.3	57.1
27	84.3	51.2	52.8	51.7	53.5	54.4	55.3
26	81.9	49.6	51.1	50	51.6	52.6	53.5
25	79.4	47.8	49.3	48.3	49.7	50.7	51.6
24	76.9	46.1	47.5	46.5	47.9	48.9	49.8
23	74.2	44.3	45.7	44.7	46.1	47.1	48
22	71.5	42.5	43.8	42.9	44.3	45.3	46.2
21	68.6	40.6	41.9	41	42.6	43.5	44.3
20	65.6	38.7	39.9	39	40.8	41.6	42.5
19	62.6	36.7	37.9	37.1	39	39.8	40.6
18	59.4	34.7	35.8	35.1	37.2	38	38.7
17	56.2	32.7	33.7	33	35.5	36.2	36.8
16	52.8	30.7	31.6	30.9	33.6	34.3	34.9
15	49.4	28.6	29.4	28.8	31.7	32.3	32.8
14	45.9	26.5	27.3	26.7	29.8	30.3	30.8
13	42.3	24.4	25.1	24.5	27.8	28.2	28.6
12	38.7	22.2	22.8	22.3	25.7	26.1	26.4
11	34.9	20	20.6	20.1	23.6	23.9	24.1
10	31.1	17.8	18.3	17.9	21.3	21.5	21.7
9	27.3	15.6	16	15.6	18.9	19.1	19.3
8	23.4	13.4	13.7	13.4	16.5	16.7	16.8
7	19.6	11.1	11.4	11.2	14	14.1	14.2
6	15.8	9	9.2	9	11.4	11.5	11.5
5	12.1	6.9	7	6.9	8.9	8.9	8.9
4	8.6	4.9	5	4.9	6.4	6.4	6.4
3	5.4	3.1	3.1	3.1	4.1	4.1	4.1
2	2.7	1.5	1.6	1.5	2.1	2.1	2.1
1	0.8	0.5	0.6	0.5	0.6	0.6	0.6
Base	0	0	0	0	0	0	0

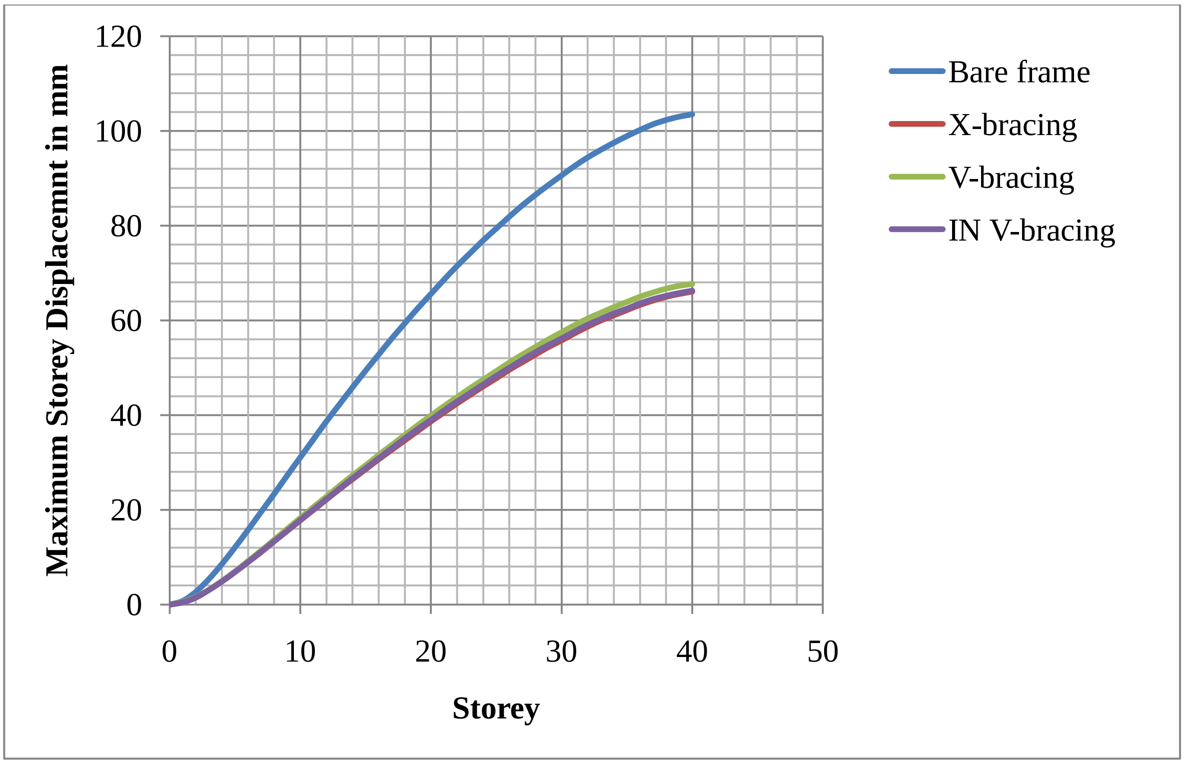


Figure 5.9: Double bay bracing Storey Displacement in X- direction with X, V and inverted V bracing type. (When a double bay is braced).

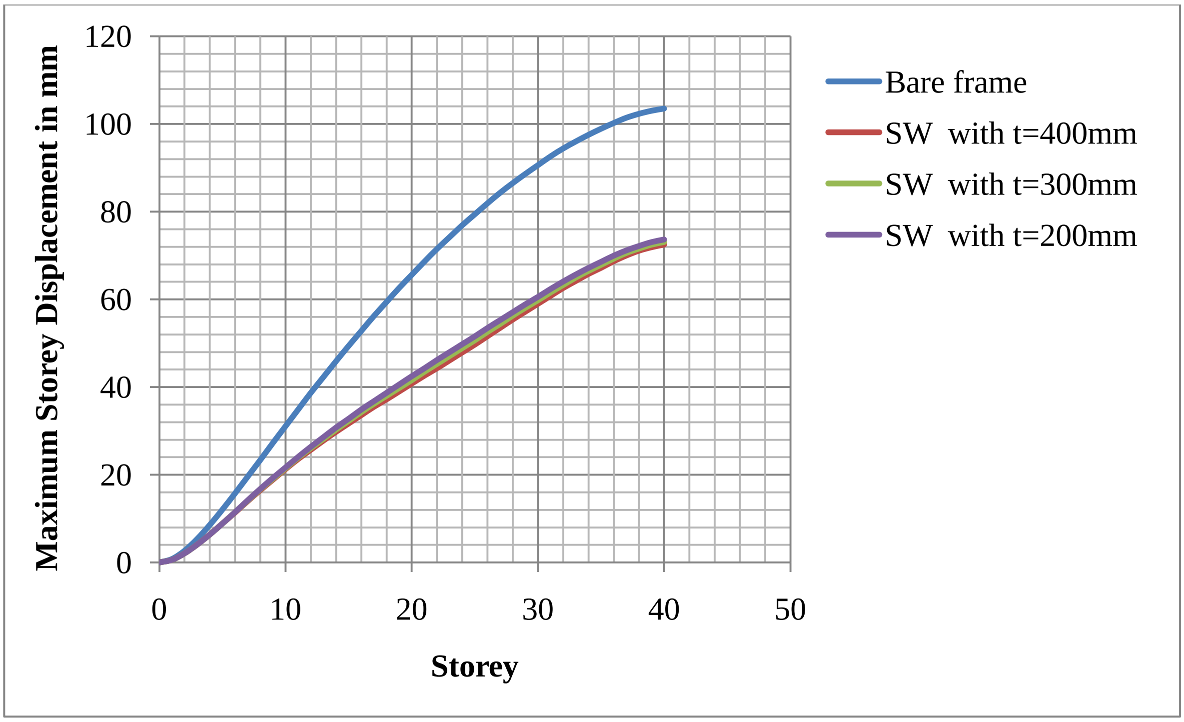


Figure 5.10: Double bay bracing Storey Displacement in X- direction with 200,300, and 400 mm thickness shear walled frame with reference to bare frame. (When a double bay is shear walled).

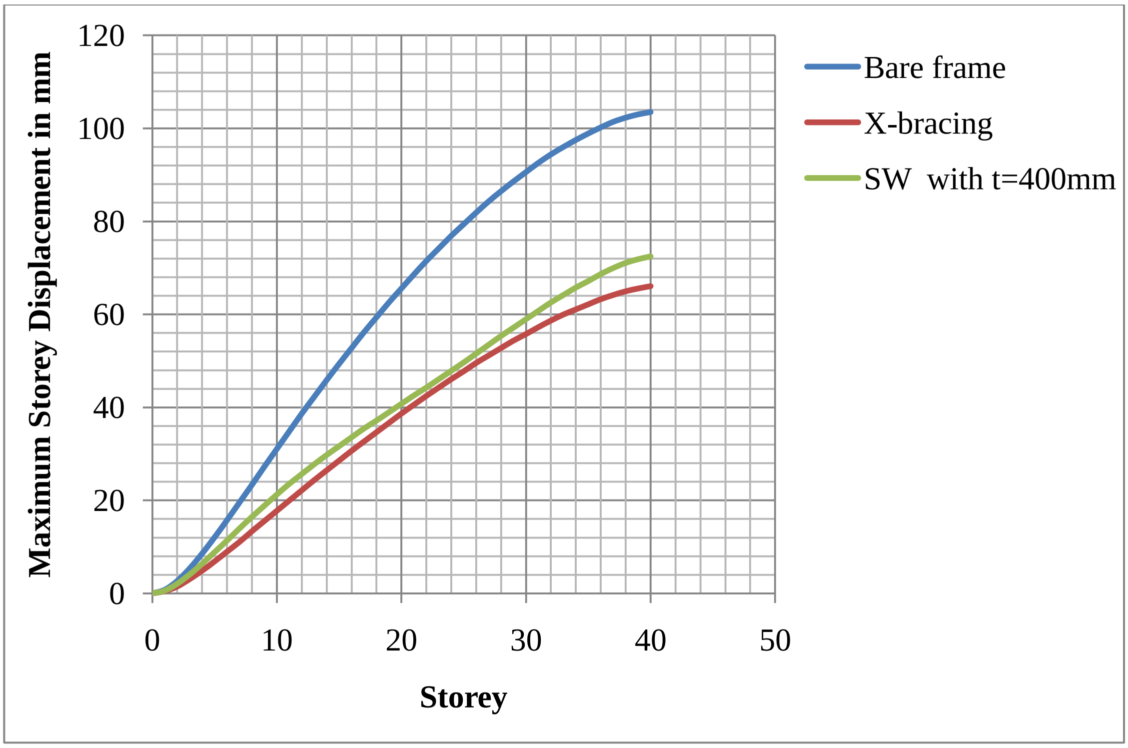


Figure 5.11: Double bay bracing Storey displacement in X- direction for 400 mm shear walled frame and X- braced frame. (When a double bay is braced and shear walled).

As shown in Table 5.7, the storey displacement was higher when no shear wall and bracing members were provided, but when shear wall and bracing member were provided then Storey displacement decreases. By comparison, the shear wall with thickness 400mm showed better reduction in storey displacement as compared to 200mm, and 300mm thickness shear wall. The maximum reduction in storey displacement of 72.5 mm was observed for building with 400mm thickness shear wall when compared with 103.5mm for bare frame at the 40th storey. So there was 29.95% reduction for this case.

Similarly from braced frame, X- braced frame showed the maximum reduction in storey displacement as compared to V and inverted V- braced frame. The maximum storey displacement of 66.1 mm was observed for building with X- bracing when compared with 103.5mm for bare frame. So there was 36.13% reduction for this case.

Therefore, from braced frame X - braced frame showed the maximum reduction in storey displacement as compared to V and inverted V - braced frame, and from shear walled frame 400 mm thickness shear walled frame showed the maximum reduction in storey displacement as compared to 200mm, and 300mm thickness shear walled frame, therefore when X- braced frame was compared to 400 mm thickness shear walled frame, X- braced frame showed the maximum reduction in storey displacement than 400 mm thickness shear walled frame with the whole stories.

Generally the following table elaborates the reduction of storey displacement in percentage.

Table 5.8 Reduction of top Storey displacement in % in X- direction. (When a double bay is braced and shear walled).

Model	Reduction of maximum Storey displacement in % at the 40th storey
X-bracing	36.13
V-bracing	34.59
In V-bracing	35.94
SW with t=400mm	29.95
SW with t=300mm	29.37
SW with t=200mm	28.79

Table 5.9 Storey Displacement in Y- direction with 200,300, and 400 mm thickness shear walled frame and with X, V and inverted V bracing types. (When a double bay is braced and shear walled).

		Bracing			Shear wall with different thickness		
Storey	Bare frame	X-bracing	V-bracing	IN V-bracing	SW with t=400mm	SW with t=300mm	SW with t=200mm
40	109.6	72.4	73.9	72.6	79.9	80.2	80.4
39	109	71.9	73.5	72.1	79.4	79.7	79.8
38	108.2	71.3	72.8	71.5	78.5	78.9	79
37	107.1	70.5	72	70.7	77.5	77.8	78
36	105.8	69.6	71.1	69.8	76.3	76.6	76.8
35	104.3	68.5	70	68.7	74.9	75.3	75.5
34	102.8	67.4	68.9	67.6	73.5	73.9	74.1
33	101.2	66.2	67.7	66.5	72	72.4	72.7
32	99.3	64.9	66.4	65.2	70.4	70.9	71.2
31	97.4	63.5	64.9	63.8	68.6	69.2	69.5
30	95.2	61.9	63.4	62.2	66.8	67.4	67.7
29	93.1	60.4	61.8	60.7	65	65.6	66
28	90.8	58.8	60.2	59.1	63.2	63.8	64.2
27	88.4	57.1	58.5	57.5	61.2	61.9	62.4
26	85.9	55.4	56.7	55.7	59.3	60	60.5
25	83.2	53.5	54.8	53.8	57.3	58	58.5
24	80.5	51.7	52.9	52	55.3	56	56.6
23	77.7	49.8	50.9	50	53.3	54	54.6
22	74.7	47.8	48.9	48.1	51.3	52	52.5
21	71.7	45.7	46.8	46	49.2	49.9	50.5
20	68.5	43.6	44.6	43.9	47.1	47.8	48.3
19	65.2	41.5	42.4	41.7	45	45.7	46.2
18	61.9	39.3	40.2	39.5	42.9	43.5	44
17	58.5	37	37.9	37.2	40.7	41.3	41.8
16	55	34.8	35.5	34.9	38.5	39	39.5
15	51.3	32.4	33.1	32.6	36.1	36.7	37.1
14	47.7	30	30.7	30.2	33.8	34.3	34.7
13	43.9	27.6	28.2	27.7	31.4	31.8	32.2
12	40.1	25.2	25.7	25.3	28.9	29.3	29.6
11	36.2	22.7	23.1	22.7	26.3	26.7	27
10	32.2	20.2	20.5	20.2	23.7	24	24.2
9	28.2	17.6	17.9	17.7	20.9	21.2	21.4
8	24.2	15.1	15.4	15.1	18.1	18.4	18.6
7	20.2	12.6	12.8	12.6	15.3	15.5	15.6
6	16.2	10.1	10.3	10.1	12.4	12.6	12.7
5	12.4	7.7	7.8	7.7	9.6	9.7	9.8
4	8.8	5.5	5.6	5.5	6.9	6.9	7
3	5.6	3.5	3.5	3.4	4.3	4.4	4.4
2	2.8	1.7	1.8	1.7	2.2	2.2	2.2
1	0.8	0.5	0.6	0.5	0.6	0.6	0.6
Base	0	0	0	0	0	0	0

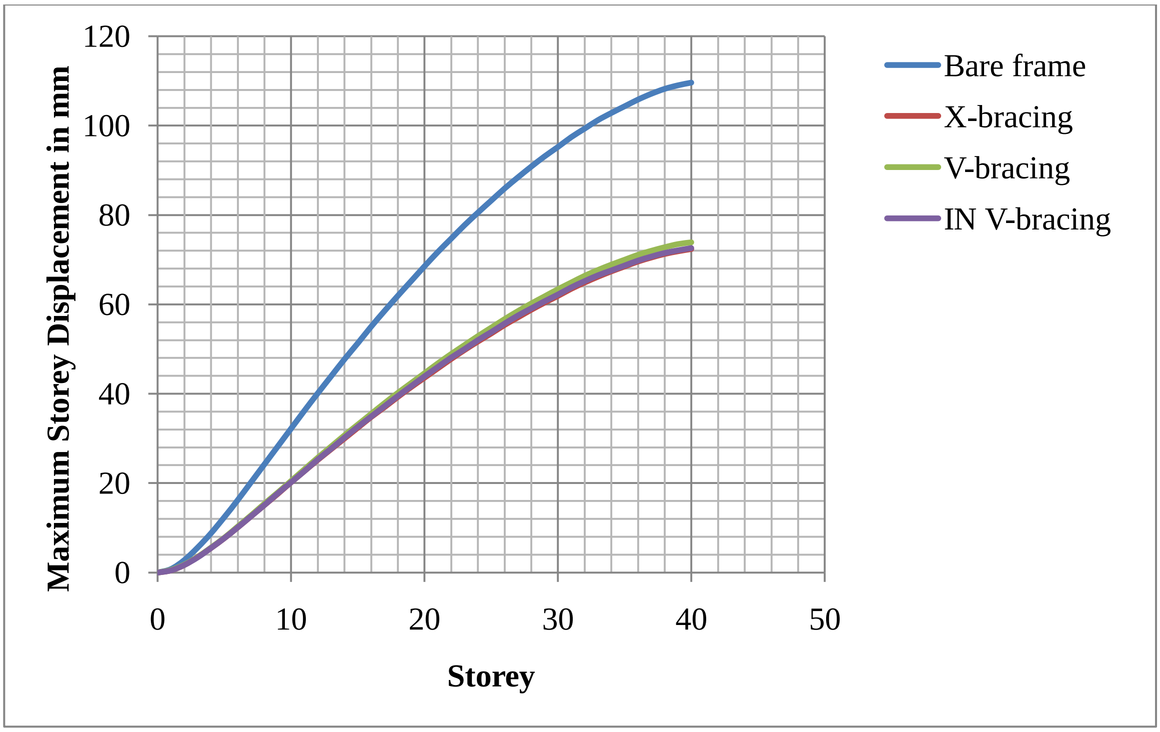


Figure 5.12: Double bay bracing Storey Displacement in Y- direction with X, V and inverted V bracing type. (When a double bay is braced).

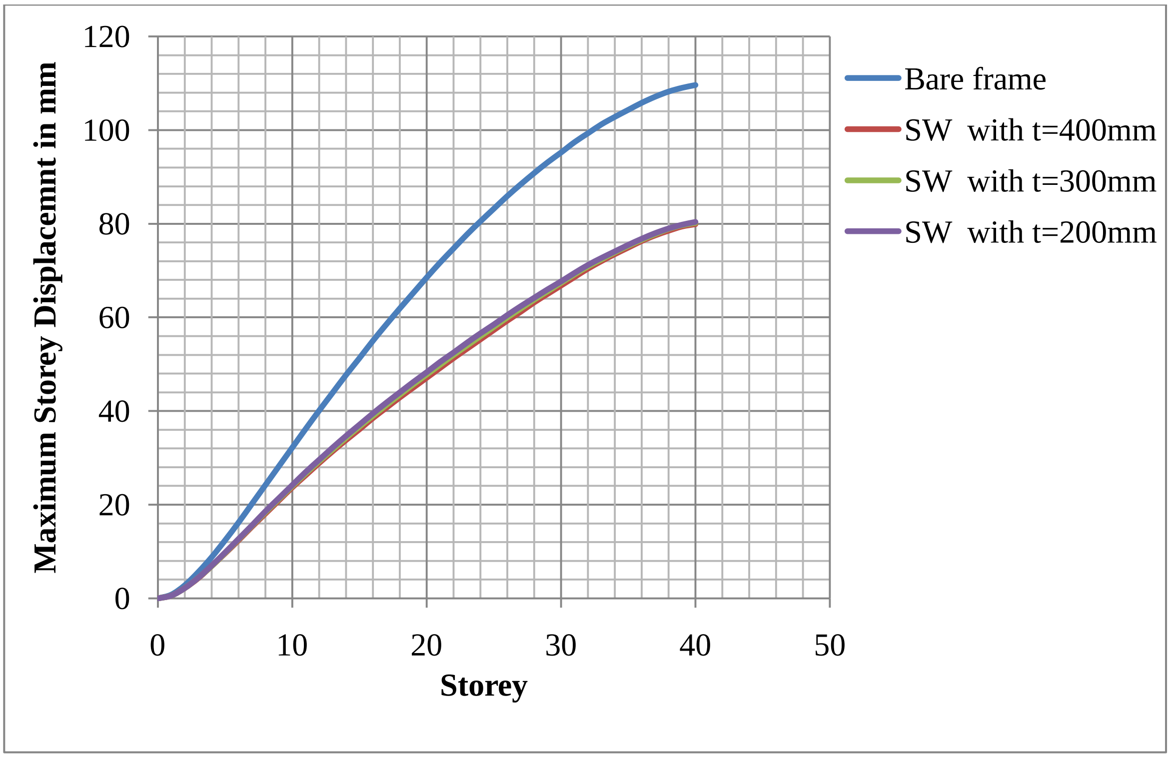


Figure 5.13: Double bay bracing Storey Displacement in Y- direction with 200,300, and 400 mm thickness shear walled frame. (When a double bay is shear walled).

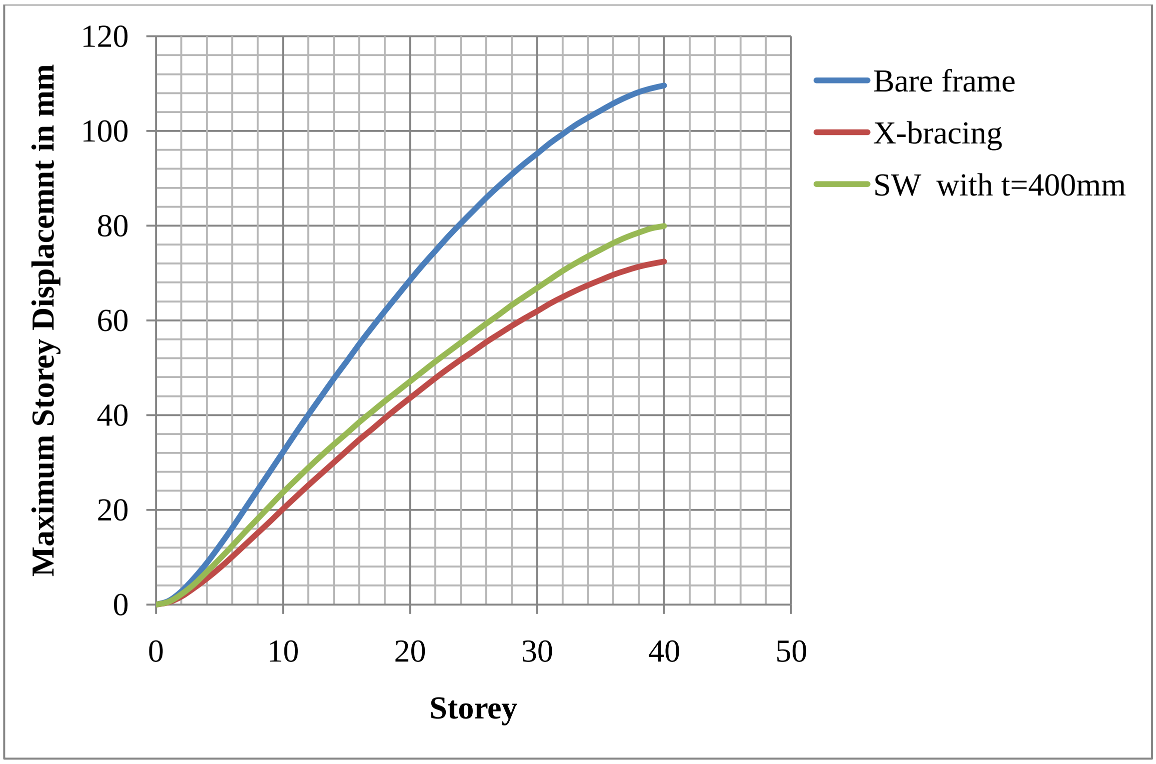


Figure 5.14: Double bay bracing Storey displacement in Y- direction for 400 mm shear walled frame and X- braced frame. (When a double bay is braced and shear walled).

As shown in Table 5.9, the storey displacement was higher when no shear wall and bracing members were provided, but when shear wall and bracing member were provided then Storey displacement decreases. By comparison, the shear wall with thickness 400mm showed better reduction in storey displacement as compared to 200mm, and 300mm thickness shear wall. The maximum reduction in storey displacement of 79.9 mm was observed for building with 400mm thickness shear wall when compared with 109.6mm for bare frame at the 40th storey. So there was 27.1% reduction for this case.

Similarly from braced frame, X- braced frame showed the maximum reduction in storey displacement as compared to V and inverted V- braced frame. The maximum storey displacement of 72.4 mm was observed for building with X- bracing when compared with 109.6mm for bare frame. So there was 33.94% reduction for this case.

Therefore, from braced frame X - braced frame showed the maximum reduction in storey displacement as compared to V and inverted V- braced frame, and from shear walled frame 400 mm thickness shear walled frame showed the maximum reduction in storey displacement as compared to 200mm, and 300mm thickness shear walled frame, therefore when X- braced frame was compared to 400 mm thickness shear walled frame, X- braced frame showed the maximum reduction in storey displacement than 400 mm thickness shear walled frame with the whole stories.

Generally the following table elaborates the reduction of storey displacement in percentage.

Table 5.10 Reduction of top Storey displacement in % in y- direction. (When a double bay is braced and shear walled).

Model	Reduction of maximum Storey displacement in % at the 40th storey
X-bracing	33.94
V-bracing	32.57
In V-bracing	33.76
SW with t=400mm	27.1
SW with t=300mm	26.82
SW with t=200mm	26.64

CHAPTER SIX

CONCLUSIONS AND RECOMMENDATIONS

6.1 Conclusions

The behaviour of seismic load resisting elements, which includes concentric braced frame model, and shear wall model were studied. Models with different parameters were created and response spectrum analysis method was performed. The plot of maximum storey displacements at each story level and fundamental period of the system is done. And from the results obtained, the following conclusions are drawn:

i. When shear wall and bracing provided in a single bay

1. In both X and Y direction, Inverted V- bracing or chevron bracing showed the maximum reduction in fundamental time period and storey displacement than the other types of bracing systems, which are X-bracing and V-bracing.
2. In both X and Y direction, 300mm thickness shear walled frame showed the maximum reduction in fundamental time period and storey displacement than 400mm and 200mm thickness shear walled frame.
3. It is noticed that from comparison plots of each graph, Inverted V- braced frame and 300mm thickness shear walled frame has almost equal values of maximum storey displacement and fundamental time period. The percentage difference is less than 2.4% and 3.7% respectively, that is the maximum storey displacement and fundamental time period found from 300mm thickness shear walled frame is higher by 2.4% and 3.7% respectively, compared to the values obtained in Inverted V- braced frame.
4. Even if, by comparison 300mm thickness shear walled frame is better than 200mm, 400mm thickness shear walled frame, their difference in reduction of maximum storey displacement and fundamental time period is almost the same, which is less than 2%.This is also true for bracing systems, their difference in reduction of maximum storey displacement and fundamental time period is almost the same, which is less than 2%.
5. Increase shear wall thickness was not always beneficial for earthquake resistance design. As it is seen from comparison plots, 300mm thickness shear walled frame

showed the maximum reduction in storey displacement and fundamental time period.

ii. When shear wall and bracing provided in a double bay

6. In both X and Y direction, X- bracing showed the maximum reduction in fundamental time period and storey displacement than the other types of bracing systems, which are inverted V-bracing and V-bracing.
7. In both X and Y direction, 400mm thickness shear walled frame showed the maximum reduction in fundamental time period and storey displacement than 300mm and 200mm thickness shear walled frame.
8. from comparison plots of X- braced frame and 400mm thickness shear walled frame, the difference in reduction of maximum storey displacement and fundamental time period between 400mm thickness shear walled frame and X- braced frame is 6.84% and 10.71% respectively, that is the maximum storey displacement found from X- braced frame is higher by 6.84%, compared to the values obtained in 400mm thickness shear walled frame. And the fundamental time period found from 400mm thickness shear walled frame is higher by 10.71% compared to the values obtained in X- braced frame.
9. Even if, by comparison 400mm thickness shear walled frame is better than 200mm, 300mm thickness shear walled frame, their difference in reduction of maximum storey displacement and fundamental time period is almost the same, which is less than 2%.This is also true for bracing systems, their difference in reduction of maximum storey displacement and fundamental time period is almost the same, which is less than 2%.
10. Finally as the number of braced bay increase the effectiveness of bracing is higher than shear wall.

6.2 Recommendations

This thesis work is an inch towards the complex phenomena to select the performance of various bracing types, and different thickness shear walls. Under this study the sample of the bracing type that was considered as classified under concentrically bracing. Among the possibilities for future study, the following are the main points that deserve attention.

1. In this study it is only considered the seismic behaviour of concentrically type of bracing and shear walled frame. A study for the seismic behaviour of eccentrically braced frame and shear walled frame under similar criteria of comparison is left for future investigation.
2. The analysis takes place by selecting a wide flange steel section for bracing cross section. A study for another cross section like angle section, tubular section, etc is left for future investigation.
3. It is also considered that the structure here considered fulfils plan and elevation regularity, the behaviors for irregular structures under those bracing type can be considered for future study.
4. In this paper cost comparison is not included to compare shear walled frame and braced frame, for future study it is also better to study by considering the cost of the systems.
5. The effect of wind on the seismic behaviour of braced frame and shear walled frame is left for future study.

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APPENDIX-A: Behaviour factor calculation for bare frame and shear wall

For buildings that are regular in elevation, the values of behaviour factor q for the various structural types are calculated.

1. When the length of the shear wall is 6m(one bay)
   i. For bare frame

Since it is a frame system, the behaviour factor is calculated below

From the table 3.2 qo=$3\left(\frac{\alpha u}{\alpha 1}\right)$ for Ductility Class Medium (DCM)

Again from table 3.3, $\left(\frac{\alpha u}{\alpha 1}\right)$ =1.3.

And Kw = 1.00.

$$q_o = 3 * 1.3 = 3.9$$

$$q = q_o K_w \geq 1.5$$

$$q = 3.9 * 1 = 3.9$$

   ii. For shear walled frame

The shear resistance of the, 200mm, and 300mm thickness shear walls at the building base is in between 50% and 65% of the total seismic resistance of the whole structural system, therefore the system is wall-equivalent dual system. And the behaviour factor is calculated below.

From the table 3.2 qo=$3\left(\frac{\alpha u}{\alpha 1}\right)$ for Ductility Class Medium (DCM)

For wall-equivalent dual systems, from table 3, $\left(\frac{\alpha u}{\alpha 1}\right)$ =1.2.

And Kw, is calculated below

$$0.5 \leq \frac{1+\alpha_o}{3} \leq 1 \text{ for wall} - \text{eqquivalent systems}$$

$$\alpha_o = (\sum h_{wi}) / \sum l_{wi}$$

$$\alpha_o = \frac{8*118.5}{8*6} = 19.75$$

$$Kw = \frac{1+19.75}{3} = 6.91$$

$$qo = 3 * 1.2 = 3.6$$

$$q = q_o K_w \geq 1.5$$

$$q = 3.6 * 1 = 3.6$$

Table A.1 Behaviour factor for shear walled frame with single bay shear walled

q=3.6	With thickness of shear wall = 200mm		
	VX	VY	VX+VY
V total	11501	11295.09	22796.09
V shear wall	6198.76	5656	11854.76
(V shear wall/V total)*100	52%		

q=3.6	With thickness of shear wall = 300mm		
	VX	VY	VX+VY
V total	11972	11745	23716
V shear wall	6906	6315	13221
(V shear wall/V total)*100	55.7%		

q=3.6	With thickness of shear wall = 400mm		
	VX	VY	VX+VY
V total	12468	12211	24679
V shear wall	7584	6956	14540
(V shear wall/V total)*100	58.9%		

2. When the length of the shear wall is 12m(two bay)

The shear resistance of the 200mm, 300mm, and 400mm thickness shear walls at the building base is higher than 65% of the total seismic resistance of the whole structural system; therefore the system is wall system. And the behaviour factor is calculated below.

For wall system, from table 3 $\left(\frac{\alpha u}{\alpha 1}\right)$ = 1.0

And Kw, is calculated below

$$0.5 \leq \frac{1+\alpha_o}{3} \leq 1 \text{ for wall} - \text{eqquivalent systems}$$

$$\alpha_o = (\sum h_{wi}) / \sum l_{wi}$$

$$\alpha_o = \frac{8*118.5}{8*12} = 9.875$$

$$Kw = \frac{1+9.875}{3} = 3.625>1$$

$$qo = 3 * 1 = 3$$

$$q = q_o K_w \geq 1.5$$

$$q = 3 * 1 = 3$$

Table A.2 Behaviour factor for shear walled frame with double bay shear walled

q=3.6	With thickness of shear wall = 200mm		
	VX	VY	VX+VY
V total	15833.79	14896.33	30730.13
V shear wall	11679.6	10486.64	22166.24
(V shear wall/V total)*100	72.13%		

q=3.6	With thickness of shear wall = 300mm		
	VX	VY	VX+VY
V total	17129.62	16192.28	33321.9
V shear wall	13243.28	11983.4	25226.7
(V shear wall/V total)*100	75.7%		

q=3.6	With thickness of shear wall = 400mm		
	VX	VY	VX+VY
V total	18479.59	17557.85	36037.44
V shear wall	14824.12	13533.12	28357.24
(V shear wall/V total)*100	78.68%		

Where, Vx = Base shear in X direction

Vy = Base shear in Y direction

V total = The sum of frame and shear wall base shear

V shear wall = Shear wall base shear

APPENDIX-B: Behaviour factor for bracing

In forced-based seismic design procedures, behaviour factor, *q* is a force reduction factor used to reduce the linear elastic response spectra to the inelastic response spectra. The behaviour factor, *q*, therefore accounts for the inherent ductility and over strength of a structure and the difference in the level of stresses considered in its design [14].

Since no code-specified q factor for steel-braced RC frames are known, storey displacement and period of the building are determined using q=7 for X-braced frame and q=8 for chevron and V-braced frame. There are two reasons for this choice; firstly, the results of two related works showed a behaviour factor (q) ranging from 6.5 to 8 for X-braced and from 7 to 9 for Chevron-braced intermediate RC frame dual systems [2].

Determination of behaviour factor for RC building with steel bracing

In forced-based seismic design procedures, behaviour factor, q is a force reduction factor used to reduce the linear elastic response spectra to the inelastic response spectra. The behaviour factor, q, therefore accounts for the inherent ductility and over strength of a structure and the difference in the level of stresses considered in its design. In another study carried out by Maheri and Akbari the seismic behaviour factor (q) was evaluated for steel X-braced RC buildings. The q factor components including ductility reduction factor and over strength factor were extracted from inelastic pushover analyses of brace-frame systems of different heights and configurations. In that study 4-storey, 8-storey and 12-storey frames were considered. These are typical numbers of storey’s used by some other investigators to cover low-rise to medium-rise framed buildings. All frames were three-bay wide with the central bay braced in the braced dual systems. DRAIN-2DX program was utilized to carry out nonlinear pushover analysis of each system. Inelastic pushover analysis of the multi-storey systems under investigation was carried out at horizontal load steps equal to 2% of the design capacity. A constant gravity load equal to total dead load plus 20% live load was also applied to each frame. Using the above methods of analysis and using other two related works showed a behaviour factor (q) ranging from 6.5 to 8 for X-braced and from 7 to 9 for Chevron-braced intermediate RC frame dual systems [2].

APPENDIX-C: Bracing sections

The following wide flange inverted V steel sections were tested to select the appropriate section.

Table C.1 Trial bracing steel sections for braced frame.

Section	1	2	3	4	5	6	7
1-10	600*300*15	500*300*15	400*300*15	400*250*15	300*200*15	250*150*15	200*150*15
11-20	550*250*15	450*250*15	350*250*15	300*200*15	250*150*15	200*150*15	200*100*15
21-30	500*200*15	400*200*15	300*200*15	250*150*15	200*100*15	150*150*15	150*100*15
31-40	450*150*15	350*150*15	250*150*15	200*100*15	150*100*15	150*100*15	100*100*15

Note: the dimensions are in mm

Table C.2. Maximum Storey Displacement for selected bracing steel section.

	Inverted V-bracing													
	1		2		3		4		5		6		7	
Story	X-Dir	Y-Dir	X-Dir	Y-Dir	X-Dir	Y-Dir	X-Dir	Y-Dir	X-Dir	Y-Dir	X-Dir	Y-Dir	X-Dir	Y-Dir
	mm	mm	mm	mm	mm	Mm	mm	mm	mm	mm	mm	mm	mm	mm
40	85.2	91	85.6	91.4	86.1	91.9	86.8	92.6	88.2	94	89.3	95	90.1	95.9
39	84.7	90.5	85.1	90.9	85.6	91.3	86.3	92	87.8	93.4	88.8	94.4	89.7	95.3
38	84.1	89.7	84.5	90.1	85	90.6	85.7	91.3	87.1	92.7	88.1	93.7	89	94.5
37	83.2	88.8	83.6	89.2	84.1	89.7	84.8	90.3	86.2	91.7	87.3	92.7	88.2	93.6
36	82.2	87.7	82.6	88.1	83.1	88.5	83.8	89.2	85.2	90.6	86.2	91.6	87.1	92.5
35	80.9	86.4	81.3	86.7	81.8	87.2	82.5	87.9	83.9	89.3	85	90.2	85.9	91.1
34	79.7	85	80.1	85.4	80.6	85.9	81.3	86.5	82.7	87.9	83.7	88.9	84.6	89.8
33	78.3	83.6	78.7	84	79.2	84.4	79.9	85.1	81.3	86.4	82.3	87.4	83.2	88.3
32	76.7	82	77.1	82.3	77.6	82.8	78.3	83.5	79.8	84.8	80.8	85.8	81.7	86.6
31	75.1	80.2	75.5	80.6	75.9	81	76.6	81.7	78.1	83	79.1	84	80	84.8
30	73.2	78.3	73.7	78.7	74.1	79.1	74.8	79.8	76.2	81.1	77.3	82.1	78.1	82.9
29	71.4	76.4	71.8	76.8	72.3	77.2	73	77.9	74.4	79.2	75.4	80.1	76.3	80.9
28	69.5	74.4	69.9	74.8	70.4	75.2	71.1	75.8	72.5	77.1	73.5	78.1	74.4	78.9
27	67.5	72.3	67.9	72.7	68.4	73.1	69	73.7	70.4	75	71.5	75.9	72.3	76.7
26	65.4	70.1	65.8	70.4	66.2	70.8	66.9	71.4	68.3	72.7	69.3	73.6	70.1	74.4
25	63.1	67.7	63.5	68.1	64	68.5	64.6	69.1	66	70.3	67	71.2	67.8	72
24	60.9	65.4	61.3	65.7	61.7	66.1	62.4	66.7	63.7	67.9	64.7	68.8	65.5	69.5
23	58.6	62.9	59	63.3	59.4	63.6	60	64.2	61.3	65.3	62.3	66.2	63.1	66.9
22	56.2	60.4	56.5	60.7	57	61.1	57.6	61.6	58.8	62.7	59.8	63.6	60.6	64.3
21	53.7	57.8	54	58.1	54.5	58.4	55	59	56.3	60	57.3	60.9	58	61.5
20	51.1	55	51.5	55.3	51.9	55.7	52.4	56.2	53.6	57.2	54.6	58.1	55.3	58.7
19	48.5	52.3	48.9	52.6	49.3	52.9	49.8	53.4	50.9	54.4	51.9	55.2	52.5	55.8
18	45.9	49.5	46.2	49.8	46.6	50.1	47.1	50.5	48.2	51.5	49.1	52.3	49.8	52.8
17	43.2	46.6	43.5	46.9	43.9	47.2	44.4	47.6	45.4	48.5	46.3	49.3	46.9	49.8
16	40.5	43.7	40.8	43.9	41.1	44.2	41.6	44.6	42.5	45.5	43.4	46.2	43.9	46.7
15	37.6	40.7	37.9	40.9	38.3	41.2	38.7	41.5	39.6	42.3	40.4	43	40.9	43.5
14	34.8	37.6	35.1	37.8	35.4	38.1	35.8	38.4	36.7	39.2	37.4	39.8	37.9	40.2
13	32	34.5	32.2	34.7	32.5	35	32.9	35.3	33.7	36	34.4	36.6	34.8	37

12	29.1	31.4	29.3	31.6	29.6	31.8	29.9	32.1	30.6	32.7	31.3	33.3	31.7	33.6
11	26.2	28.2	26.4	28.4	26.6	28.6	26.9	28.9	27.6	29.4	28.2	29.9	28.5	30.2
10	23.2	25.1	23.4	25.2	23.6	25.4	23.9	25.6	24.5	26.1	25	26.5	25.3	26.8
9	20.3	21.9	20.4	22	20.6	22.1	20.9	22.4	21.4	22.8	21.9	23.2	22.1	23.4
8	17.3	18.7	17.5	18.8	17.6	18.9	17.8	19.1	18.3	19.5	18.7	19.8	18.9	20
7	14.4	15.5	14.5	15.6	14.7	15.7	14.8	15.9	15.2	16.2	15.6	16.5	15.8	16.6
6	11.6	12.4	11.7	12.5	11.8	12.6	11.9	12.7	12.2	13	12.5	13.2	12.6	13.3
5	8.8	9.5	8.9	9.5	9	9.6	9.1	9.7	9.3	9.9	9.5	10.1	9.7	10.2
4	6.3	6.7	6.3	6.8	6.4	6.8	6.5	6.9	6.6	7	6.8	7.1	6.9	7.2
3	4	4.2	4	4.2	4	4.3	4.1	4.3	4.2	4.4	4.3	4.5	4.3	4.5
2	2	2.1	2	2.1	2	2.1	2	2.2	2.1	2.2	2.1	2.2	2.2	2.3
1	0.6	0.6	0.6	0.6	0.6	0.6	0.6	0.6	0.6	0.6	0.6	0.6	0.7	0.7
0	0	0	0	0	0	0	0	0	0	0	0	0	0	0

Note: the dimensions are in mm

The above table indicates that, increasing the section of steel doesn't mean reducing the storey displacement, even though insignificant reductions observed. For the comparison purpose trial section -1 is used

Section Adequacy

1. Trial bracing section-7 Adequacy checking

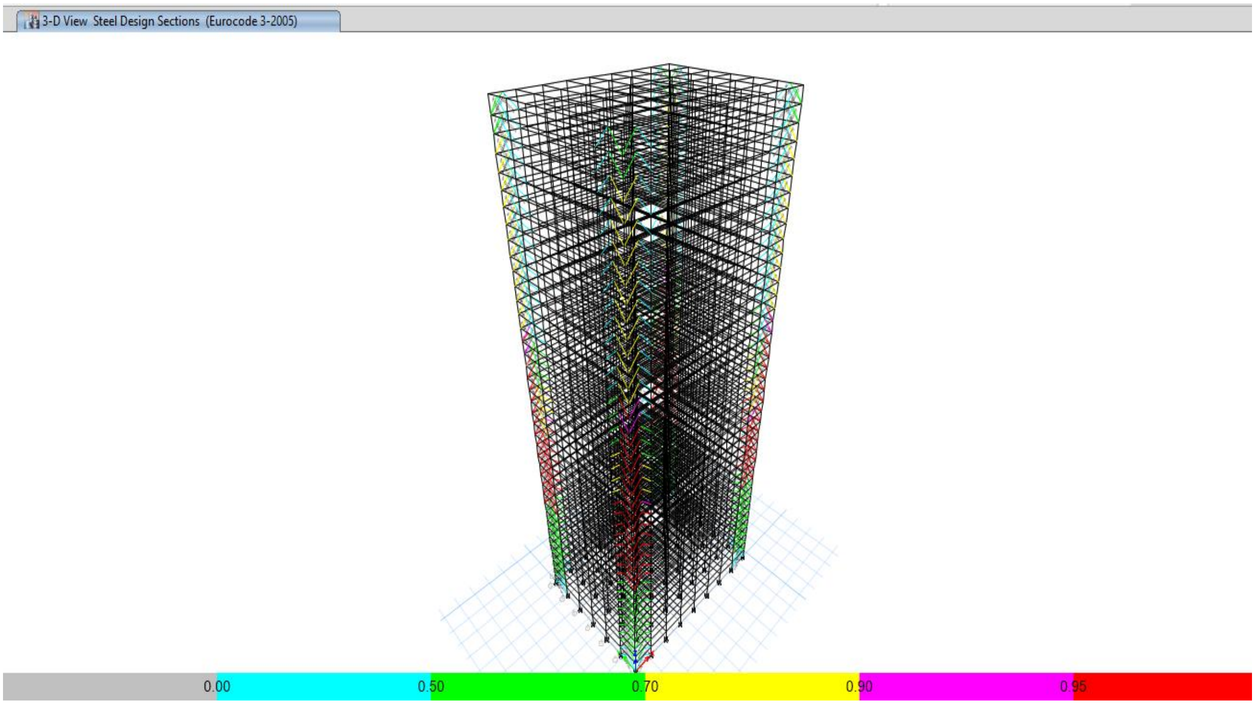

Figure C1. Adequacy checking for bracing section-7

According to the figure above the demand capacity ratio of some section is greater than 0.95(According to Euro code 3-2005), hence which is not safe to use.

2. Trial bracing section-1 Adequacy checking

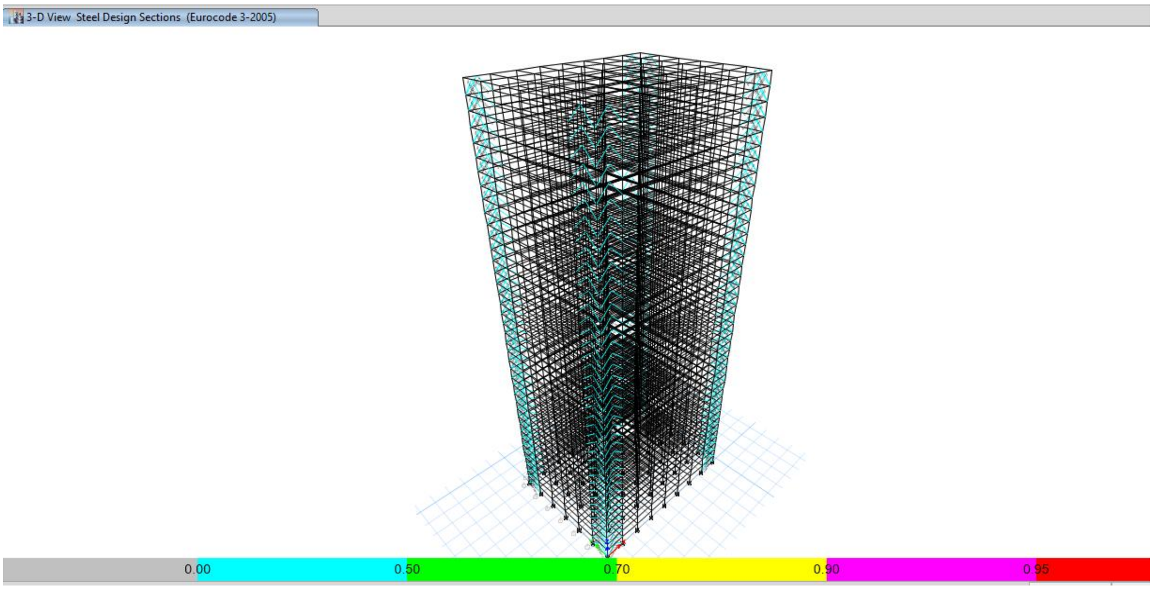

Figure C2. Adequacy checking for bracing section-1

According to the figure above the demand capacity ratio is less than 0.95(according to euro code), hence which is safe to use. And also section-2, 3, 4, 5, and 6 is safe. But, for better seismic response section-1 is used.

3. The Column section is also checked according to demand capacity ratio (D/C) < 1

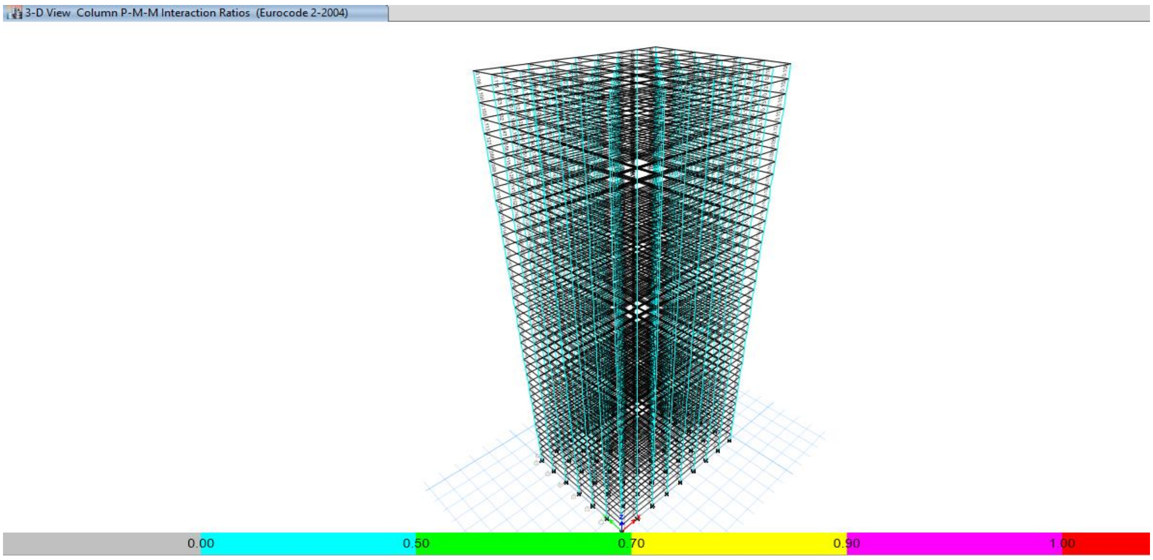

Figure C3. Adequacy checking for column cross-section.

According to the figure above the demand capacity ratio is less than 1 (according to euro code), hence which is safe to use.

APPENDIX-D Second Order effects

The criterion for taking into account the second order effect is based on the inter storey drift sensitivity coefficient θ, which is define with equation (EN 1998-11, cl.4.4.2.2 (2)).

$$\Theta = \frac{Ptot * dr}{Vtot * h} \text{.......................................} \quad (D-1)$$

dr: is the inter storey drift,

h: is the storey height,

Vtot: is the total seismic storey shear

Ptot: is the total gravity load at and above storey considered in the seismic design situation (G+0.3Q).

Table D-1: Consequences of value of P-Δ coefficient θ on the analysis

$\Theta \leq 0.1$	No need to consider P-Δ effects
$0.1 \leq \Theta \leq 0.2$	P-Δ effects may be taken into account approximately by amplifying the effects of the seismic actions by $\frac{1}{1-\theta}$
$0.2 \leq \Theta \leq 0.3$	P-Δ effects must be accounted for by an analysis including second order effects explicitly
$\Theta \geq 0.3$	Not permitted

Second order checking in X- direction

Table D-2: Second order check in X-direction

Storey	dr-x	P total	Vtot	H	Ptot*dr-x/Vtot*h	Remark
40	0.00023	13217.67	568.4997	3	0.0017	No need of P-Δ
39	0.00035	26435.34	1154.401	3	0.0026	No need of P-Δ
38	0.00046	39653.01	1674.141	3	0.0036	No need of P-Δ
37	0.00056	52870.68	2122.632	3	0.0047	No need of P-Δ
36	0.00065	66088.35	2510.046	3	0.0057	No need of P-Δ
35	0.00062	79776.27	2863.219	3	0.0057	No need of P-Δ
34	0.00066	93464.19	3203.293	3	0.0064	No need of P-Δ
33	0.00072	107152.1	3518.038	3	0.0072	No need of P-Δ
32	0.00076	120840	3809.293	3	0.0081	No need of P-Δ
31	0.00082	134528	4081.64	3	0.0089	No need of P-Δ
30	0.00079	148780.6	4351.684	3	0.009	No need of P-Δ
29	0.00083	163033.3	4628.6	3	0.0097	No need of P-Δ
28	0.00087	177286	4899.758	3	0.01	No need of P-Δ
27	0.0009	191538.6	5163.204	3	0.011	No need of P-Δ
26	0.00095	205791.3	5418.509	3	0.012	No need of P-Δ
25	0.00094	220703.2	5678.076	3	0.012	No need of P-Δ
24	0.00098	235615.1	5946.356	3	0.013	No need of P-Δ
23	0.001	250527.1	6209.42	3	0.014	No need of P-Δ
22	0.001	265439	6464.043	3	0.014	No need of P-Δ
21	0.0011	280350.9	6707.936	3	0.015	No need of P-Δ
20	0.0011	296025.5	6952.226	3	0.015	No need of P-Δ
19	0.0011	311691.2	7199.574	3	0.016	No need of P-Δ
18	0.0011	327356.8	7438.093	3	0.017	No need of P-Δ
17	0.0012	343022.5	7665.57	3	0.017	No need of P-Δ
16	0.0012	358688.2	7879.341	3	0.018	No need of P-Δ
15	0.0012	375202.1	8088.25	3	0.019	No need of P-Δ
14	0.0012	391716	8295.292	3	0.019	No need of P-Δ
13	0.0013	408229.9	8493.294	3	0.020	No need of P-Δ
12	0.0013	424743.9	8684.202	3	0.021	No need of P-Δ
11	0.0013	441257.8	8867.724	3	0.021	No need of P-Δ
10	0.0013	458714.4	9051.276	3	0.022	No need of P-Δ
9	0.0013	476171.1	9236.121	3	0.022	No need of P-Δ
8	0.0013	493627.8	9415.665	3	0.022	No need of P-Δ
7	0.0013	511084.5	9592.787	3	0.022	No need of P-Δ
6	0.0012	528541.1	9766.854	3	0.022	No need of P-Δ
5	0.0011	547035	9938.535	3	0.0213	No need of P-Δ
4	0.0011	565529	10093.31	3	0.019	No need of P-Δ
3	0.0009	584022.9	10206.62	3	0.017	No need of P-Δ
2	0.0006	602516.8	10269.88	3	0.0126	No need of P-Δ
1	0.00026	621010.7	10289.29	3	0.0052	No need of P-Δ

Second order checking in y- direction

Table D-3: second order check in Y-direction

Storey	dr-y	P total	Vtot	H	Ptot*dr-x/Vtot*h	Remark
40	0.00026	13217.67	568.9905	3	0.002	No need of P-Δ
39	0.00038	26435.34	1153.53	3	0.003	No need of P-Δ
38	0.0005	39653.01	1669.972	3	0.0039	No need of P-Δ
37	0.0006	52870.68	2113.968	3	0.005	No need of P-Δ
36	0.0007	66088.35	2496.811	3	0.006	No need of P-Δ
35	0.0007	79776.27	2846.218	3	0.0062	No need of P-Δ
34	0.0007	93464.19	3183.291	3	0.007	No need of P-Δ
33	0.0007	107152.1	3495.987	3	0.0078	No need of P-Δ
32	0.0008	120840	3786.061	3	0.0087	No need of P-Δ
31	0.0008	134528	4058.057	3	0.0096	No need of P-Δ
30	0.0008	148780.6	4328.54	3	0.0097	No need of P-Δ
29	0.0009	163033.3	4606.354	3	0.01	No need of P-Δ
28	0.0009	177286	4878.595	3	0.011	No need of P-Δ
27	0.0009	191538.6	5143.089	3	0.012	No need of P-Δ
26	0.001	205791.3	5399.283	3	0.013	No need of P-Δ
25	0.001	220703.2	5659.561	3	0.013	No need of P-Δ
24	0.001	235615.1	5928.19	3	0.0137	No need of P-Δ
23	0.0011	250527.1	6191.157	3	0.0144	No need of P-Δ
22	0.0011	265439	6445.298	3	0.015	No need of P-Δ
21	0.0011	280350.9	6688.47	3	0.016	No need of P-Δ
20	0.0011	296025.5	6931.964	3	0.016	No need of P-Δ
19	0.0011	311691.2	7178.427	3	0.017	No need of P-Δ
18	0.0012	327356.8	7416.036	3	0.017	No need of P-Δ
17	0.0012	343022.5	7642.575	3	0.018	No need of P-Δ
16	0.0012	358688.2	7855.39	3	0.0193	No need of P-Δ
15	0.0013	375202.1	8063.316	3	0.019	No need of P-Δ
14	0.0013	391716	8269.312	3	0.02	No need of P-Δ
13	0.0013	408229.9	8466.196	3	0.02	No need of P-Δ
12	0.0013	424743.9	8655.812	3	0.022	No need of P-Δ
11	0.0013	441257.8	8837.735	3	0.022	No need of P-Δ
10	0.0013	458714.4	9019.199	3	0.022	No need of P-Δ
9	0.0013	476171.1	9201.373	3	0.023	No need of P-Δ
8	0.0013	493627.8	9377.859	3	0.024	No need of P-Δ
7	0.0013	511084.5	9551.716	3	0.024	No need of P-Δ
6	0.0013	528541.1	9722.515	3	0.023	No need of P-Δ
5	0.0012	547035	9890.989	3	0.022	No need of P-Δ
4	0.0011	565529	10042.86	3	0.021	No need of P-Δ
3	0.0009	584022.9	10153.99	3	0.017	No need of P-Δ
2	0.0006	602516.8	10215.97	3	0.013	No need of P-Δ
1	0.0002	621010.7	10234.96	3	0.0054	No need of P-Δ

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