EFFECTS OF BRACING SYSTEM ON MULTISTORYED STEEL BUILDING

A DISSERTATION

SUBMITTED IN THE PARTIAL FULFILLMENT OF REQUIREMENTS
FOR THE AWARD OF THE DEGREE
OF
MASTER OF TECHNOLOGY
IN

STRUCTURAL ENGINEERING

Submitted by:

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CANDIDATE'S DECLARATION

I, Rahul Kumar Meena, Roll No 2k16/ste/15 student of M.Tech. (STRUCTURAL ENGINEERING), hereby declare that the project Dissertation titled "EFFECT OF BRACING SYSTEM ON MULTISTOREYD STEEL BUILDING" which is submitted by me to the department of CIVIL ENGINEERING,DELHI TECHNOLOGICAL UNIVERSITY,DELHI in partial fulfillment of the requirement for the award of degree of Master Of Technology is original and not copied from any source without proper citation. This work has not previously formed the basis for the award of any Degree, Diploma Associate ship, Fellowship or other similar title or recognition.

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CERTIFICATE

I hereby certify that the Project Dissertation titled "Effect of bracing system on multi storied steel building " which is submitted by Rahul Kumar Meena, 2K16/STE/15 department of civil engineering(structural engineering), Delhi Technological University, Delhi in partial fulfillment of the requirement for the award of the degree of Master of Technology, is a record of the project work carried out by the student under my supervision. To the best of my knowledge this work has not been submitted in part or full for any Degree or Diploma to this University or elsewhere.

Place: Delhi

Date: 02/07/2018

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ABSTRACT

In recent years, steel construction has shifted from moment-resisting frames to concentric braced frames in regions of highly seismic prone area. Bracing element in structural system by providing more stiffness plays a vital role in structural behaviour to resist earthquake forces. Concentric bracing is one of the most common lateral load resistant systems in building frames due to their manufacturing simplicity and economy.

In this work, different types of bracing (X bracing, Inverted V bracing, K bracing, V bracing, Forward bracing, Backward bracing) have been analysed and comparison has been made on the basis of maximum lateral displacement at each floor level due to seismic and wind loading.

The main parameters considered to compare the seismic performance of buildings were bending moment, shear force, story drift, storey shear and concluded that the braced building of the storey drift decreases as compared to the unbraced building which indicates that the overall response of the building decreases, the displacement of the building decreases depending upon the different bracing system employed and the bracing sizes.

In the present study, a 20 storey steel frame structure is analysed. For this purpose, seven different models were generated by changing the bracing system in steel frame and analysed for wind and seismic forces. It may be concluded from this study that bracing element will have very important effect on structural behaviour under seismic loading. Most suitable bracing system is Backward bracing system.

Lateral displacement at top floor is reduced approximately 50% for Backward braced in frame structure compared to without bracing system.

ACKNOWLEDGEMENT

I express my deepest gratitude to Mr. G. P. Awadhiya for his guidance, support and invaluable suggestions to complete this report on**"EFFECT OF BRACING SYSTEMS ON MULTI-STOREY STEEL BUILDINGS"**.His extreme energy, creativity and excellent skills have always been a constant source of motivation for me. It is because of his excellent guidance, that I could complete this work successfully. He is a great person and one of the best supervisors, I always be thankful to him.

Date
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LIST OF SYMBOLS

P Axial force in member

Δ Global lateral deformation

δ Local deformation

L Length of member

P_d Design strength in axial compression

e Eccentricity

T Natural period of vibration

V Design shear force

P_z Design wind pressure

h Height of building

V_z Design wind velocity

V_b Basic wind speed

A Surface area of structural element or cladding

F Wind load

C_{pe} External pressure coefficient

C_{pi} Internal pressure coefficient

W Seismic weight of building

Z Zone factor

I Importance factor

R Response reduction factor

$\frac{s_a}{g}$ Average response acceleration coefficient

d Base dimension of building at plinth level

Q Design lateral force

CHAPTER 1

INTRODUCTION

1.1 GENERAL

India at present is fast developing country which requires demands in increase of infrastructure facilities along with the growth of population. Due to increased population, the demand and cost of land for housing is increasing day by day. To fulfil the need of the land for housing and other commercial purposes, vertical development that is multi-storey buildings are the only option. This type of development needs safety because these multi-storey buildings are highly susceptible to additional lateral loads due to earthquake and wind. In other countries, as the elevation of building increases, its reaction to lateral loads increases. Multi-storey reinforced concrete buildings are vulnerable to excessive deformation, which necessitate the introduction of special measures to decrease this deformation.

Due to the lateral forces acting on the building storey drift takes place which is more vulnerable as the height of the structure increases. To satisfy strength and serviceability limit, lateral stiffness is a major consideration in the design of tall buildings. The simple parameter that is used to estimate the lateral stiffness of a building is the drift index defined as the ratio of the maximum deflections at the top of the building to the total height. Different structural forms of tall buildings can be used to improve the lateral stiffness and to reduce the drift index. Steel braced frame is one of the lateral load opposing frameworks in multi-storey structures. Steel bracing system enhances the resistance of the structure against horizontal forces by expanding its stiffness and stability. Bracings hold the structure stable by exchanging the horizontal loads, for example, earthquake or wind burdens down to the ground and oppose sidelong loads, in that way keep the influence of the structure. Steel bracing members in RC multi-storey building is conservative, simple to set up, involve less space and give obliged quality and inflexibility. There are various types of bracing systems like X bracing, V bracing, inverted V bracing, K bracing, diagonal bracing and so on.

As compared to reinforced concrete structures, steel has got some important properties like high strength and ductility. We know that steel is ductile so it gives warning before failures. All these properties of steel will play very important role in case of seismic design. In this research study of different types of bracing systems have been investigated for the use in tall building in order to provide lateral stiffness and finally we conclude the best suited option from them.

1.2 SEISMIC FORCE RESISTING SYSTEMS

Several systems can be adopted to provide adequate resistance to seismic lateral forces. The most common systems are: moment resisting frames (though they consist of a three-dimensional space frame, for the purpose of analysis, they may be considered as two-dimensional in most cases), a combined system of moment frames and shear walls, braced frames with horizontal diaphragms, and a combination of the systems (MRF, BRF and SWRF). Of these, moment resisting frames may be economical for buildings with only up to five to ten storeys (the infill walls of non-reinforced masonry also provides some stiffness).

Shear wall and braced systems (which are more rigid than moment resisting frames) are economical up to 20 storeys. When frames and shear walls are combined, the system is called a dual system. A moment resisting frame, when provided with specified details for increasing the ductility and energy absorbing capacity of its components, is called a “special moment resisting frames (SMRF); otherwise it is called an ordinary moment resisting frame (OMRF).

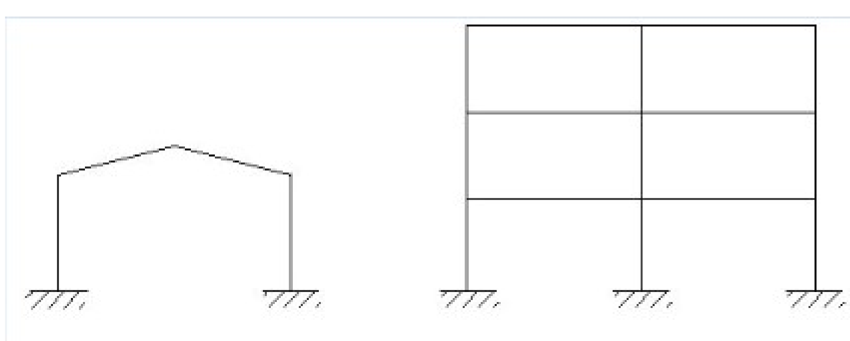

Fig. 1.1 Moment-resisting frames

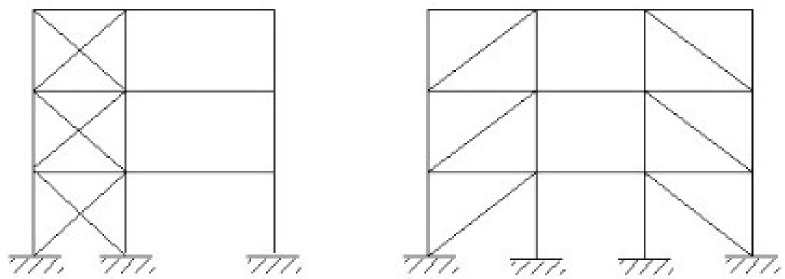

Fig. 1.2 Braced steel frame

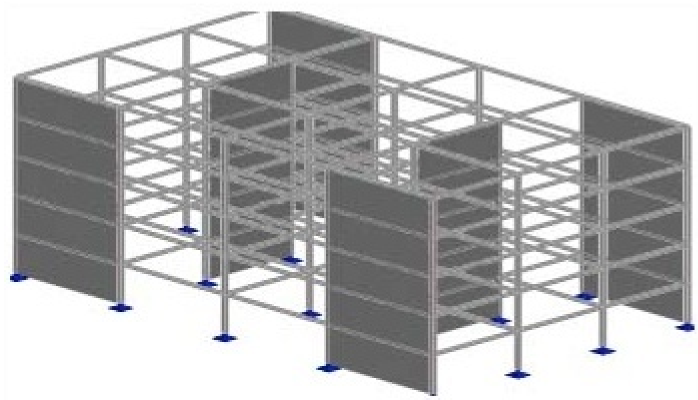

Fig. 1.3 Frames with shear walls

1.3 BRACED FRAMES

Braced frames provide resistance to lateral forces acting on a structure. The members of a braced frame act as a truss system and are subjected primarily to axial stress. Depending on the diagonal force, length, required stiffness, and clearances, the diagonal members can be made of double angles, channels, tees, tubes, or even wide flange shapes. Besides performance, the shape of the diagonal is often based on connection considerations. The braces are often placed around service cores and elevators, where frame diagonals may be enclosed within permanent walls. The braces can also be joined to form a closed or partially closed three-dimensional cell so that torsional loads can be resisted effectively. A height-to-width ratio of 8-10 is considered to form a reasonably effective bracing system.

Braced frames may be grouped into concentrically braced frames (CBFs), and eccentrically braced frames (EBFs), depending on their ductility characteristics. In addition, concentrically braced frames are subdivided into two categories, namely, ordinary concentrically braced frames (OCBFs) and special concentrically braced frames (SCBFs).

The bracing system in a building frame is designed to serve the following important functions:-

1) Resisting lateral loads
2) Counteraction the over turning moment (p-Δ moment) due to gravity loads
3) Preventing frame buckling
4) Improving sway behaviour

1.4 TYPES OF BRACED FRAMES

1) Concentrically Braced frames
2) Eccentrically Braced Frames

1.4.1 CONCENTRICALLY BRACED FRAMES

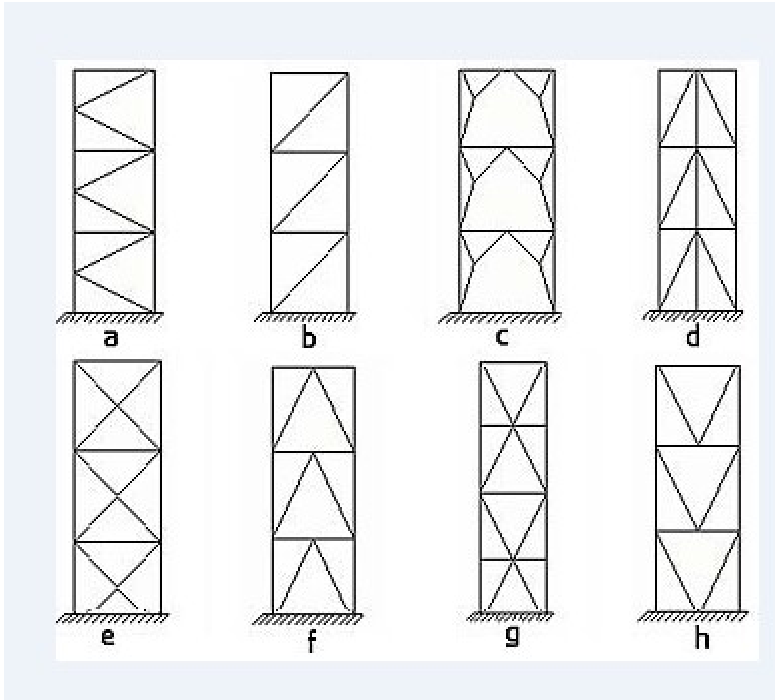

Fig. 1.4 Types of concentrically braced frames

In CBFs, the axes of all members, i.e., columns, beams, and braces, intersect at a common point such that the member forces are axial. The Chevron bracing, cross bracing(X bracing), and diagonal bracing (single, diagonal, or K bracing) are classified as concentrically braced, and are shown in above figure.

These systems resemble a vertical truss where the seismic forces are carried by axial loads produced on the members. The columns are for resisting overturning moments while the braces provide shear resistance. During an earthquake CBFs dissipate energy by yielding and buckling of the brace members. In order to ensure a satisfactory behaviour the columns are designed to remain elastic during a seismic event.

Seismic Design code specifies two types of CBFs; Ordinary Concentric Braced Frames (OCBFs) and Special Concentric Braced Frames (SCBFs). Generally, OCBFs are used in regions with smaller seismic design forces and where wind may be the controlling lateral design load. In, addition, the connection detailing requirements are not as stringent for OCBFs. On the other hand, SCBFs are used in high seismic regions and have strict connection detailing requirements.

Ordinary Concentrically Braced Frames should be designed to withstand inelastic deformation corresponding to a joint rotation of 0.02 radians without degradation in strength and stiffness, below the yield value. The slenderness of the bracing members should not exceed 120 and the required compressive strength should not exceed $0.8P_d$, where $P_d$is the design strength in axial compression. The tension braces will resist between 30-70% of the load.

Special Concentrically Braced Frames should be designed to withstand inelastic deformation corresponding to a joint rotation of 0.04 radians without degradation in strength and stiffness below the full yield value. The slenderness of the bracing members should not exceed 160 and the required compressive strength should not exceed the design strength in axial compression,P_d.

1.4.2 ECCENTRICALLY BRACED FRAMES

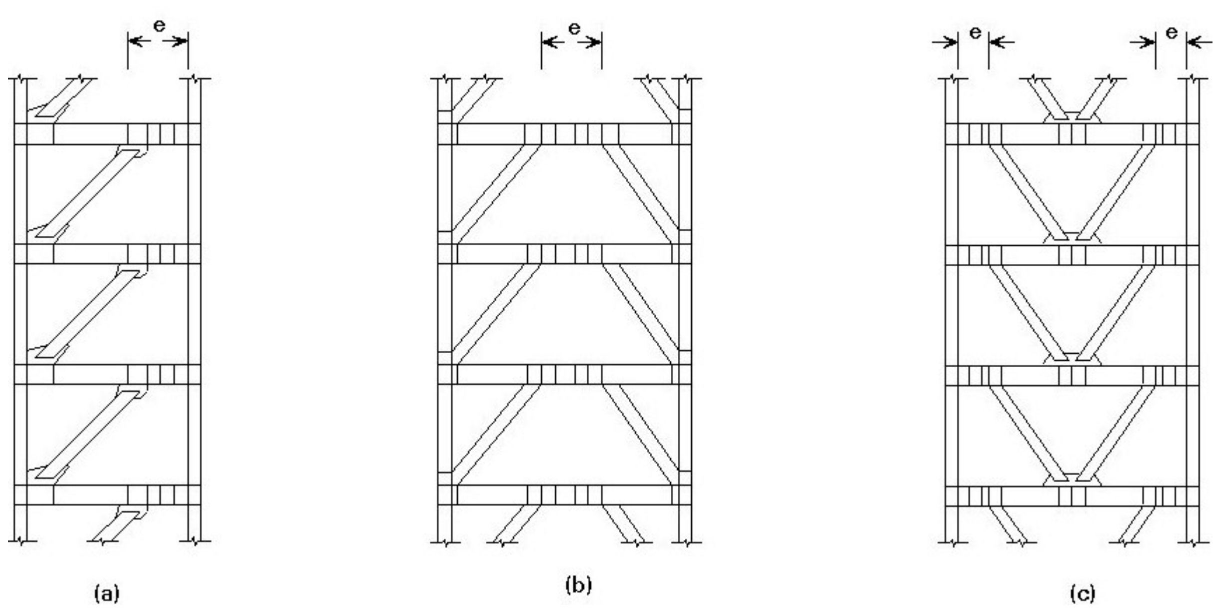

Fig. 1.5 Different types of Eccentrically Braced Frames

Eccentric Braced Frame has more ductility than concentric braced one. Therefore, the ability to absorb and dissipate energy during an earthquake in eccentric braced system is increased. In these braces, ductility is caused due to yielding the beam between two braces or the beam between the brace and the column. This part of the beam is called the link beam. These beams are experienced very large displacement, due to nonlinear behaviour of link beam sunder the applied load of diagonal braces. On the basis of present study, the EBF increases ductility but the CBF increases lateral strength.

The link beam acts as a fuse to prevent buckling of the brace due to large overloads that may occur during major earthquakes. After the elastic capacity of the system is exceeded, shear or flexural yielding of the link provides a ductile response in contrast to that obtained in a special moment resisting frame. In addition, eccentrically braced frames may be designed to control frame deformations and minimize damage to architectural finishes during seismic loading (Williams 2004). The connection between the column and beam are moment connected to achieve brace action. The web buckling is prevented by providing adequate stiffness in the link. Links longer than twice the depth of the beam tend to develop plastic hinges, while shorter links tend to yield in shear.

1.5 PROPOSED PERFORMANCE-BASED DESIGN PROCEDURE (EBF)

1.5.1 DESIGN LATERAL FORCES

Unlike in the current design codes, the design lateral force distribution in the proposed method is determined by using a shear distribution factor, βi, which is obtained from and calibrated by extensive nonlinear time-history analyses of EBF. This lateral force distribution accounts for inelastic behaviour of EBF when subjected to major earthquakes and can be expressed as (Lee et al., 2004; Chao and Goel, 2005):

$$\beta_i = \frac{V_i}{V_n} = \left(\frac{\sum_{i}^{n} w_i h_i}{w_n h_n} \right)^{0.75T^{-0.2}}$$

$$F_n = V \left(\frac{w_n h_n}{\sum_{j=1}^{n} w_j h_j} \right)^{0.75T^{-.2}}$$

$$F_i = \left(\beta_i - \beta_{i+1} \right) F_n$$

When $i = n,\ \beta_{n+1} = 0$

Where β_i = shear distribution factor at level i

V_i, V_n = story shear forces at level *i* and at the top (*n*th) level, respectively

w_i, w_j = seismic weights at level *i* and *j*, respectively

$h_{i,} h_j$ = heights of levels *i* and *j* from the ground, respectively

w_n = seismic weight of the structure at the top level

h_n = height of roof level from ground

T= fundamental structure period obtained by code specified methods or elastic dynamic analysis

F_i, F_n = lateral forces applied at level *i* and top level *n*, respectively

V= Design shear force

CHAPTER 2

LITERATURE REVIEW

2.1 BACKGROUND

Considerable research has been conducted on the behaviour of steel bracing elements and on connections and sub assemblages from concentrically braced steel-frame structures under loading representative of strong earthquake ground shaking. Because of the rapid evolution of codes, much of this research is not necessarily consistent with modern construction detailing; however, many of fundamental observations from these investigations are germane to an assessment of modern design and analysis procedure. Moreover, experimental data are critical to the validation and calibration of analytical models used for carrying out simulations to predict the performance of braced-frame structures. The available body of literature extends over several decades and is rapidly growing. As such, it cannot be adequately summarized in a brief chapter. Instead, an overview of major references is provided here along with useful citations to previous works that contain detailed reviews of related literature.

This section is a brief summary of past work carried on braced frames, specifically concentric braced frames. In this section, analytical studies focusing on steel braces and steel retrofit of existing structures surveyed within the scope of this thesis will be briefly summarized.

Badoux and Jirsa (1990) investigated the behaviour of braced frames both analytically and experimentally. Retrofitted frame was prepared to have deep beams and short columns and tested under lateral cyclic loading. An analytical study was conducted by simulating an interior column in a braced frame loaded laterally. In addition to that, a parametric study was conducted to understand the effect of slenderness ratio of braces on the response of retrofitted frame. Their studies showed that-

- Steel frame and the bracing system could be taken as independent systems and designer strength and stiffness by changing brace sections.
- To have acceptable seismic behaviour, brace sections should be designed to remain elastic because of the unpredictable nature of exposed seismic loading that can trigger buckling.
- Reducing slenderness ratio would help to prevent inelastic buckling effects on the brace sections.

Schierele and Ho (1990) published a journal paper Effect of configuration and lateral drift on high-rise space frames. Excessive lateral drift in high-rise frames can damage secondary systems, such as partitions walls and cause discomfort to building occupants. Damage to secondary system can be controlled by reducing drift. The P-δ effect is most severe in moment resisting frames. The Uniform Building Code allows smaller seismic drift for moment resisting frames (0.3% storey drift). Design for wind or seismic forces are usually based on objective to minimize lateral drift.

To reduce lateral drift of high-rise building is an important design consideration in areas of high wind or seismic activity. The research presented here shows that selecting the most appropriate bracing system can substantially reduce drift with only minor cost differences. The reductions ranged from a minimum of 1% for the 20 storey K-braced frame, to a maximum of 7.6% for the same X-braced frame. Seismic forces tend to increase with the stiffness of a building.

Hines and Jacob (1996) discussed related to the seismic performance of low ductility steel system for moderate seismic regions. Performance assessment results of eccentric braced frame on the basis of storey drift capacity, response to higher mode effects and frame overturning forces were presented. Their results show that there is no improvement in storey drift on a low ductility CBF in moderate region. Section at the top of the building experienced excessive drift due to higher mode effects, but at the same time higher mode effects benefit column design criteria by reducing building overturning forces. It is therefore advisable to study the relationship between frame ductility and column over strength demand with the aim of minimizing lateral drift.

Tremblay *et al.* (2004) performs an experimental study on the seismic performance of concentrically braced steel frames with cold-formed rectangular tubular bracing system. Analysis is performed on X- bracing and single diagonal bracing system. One of the loading sequences used is a displacement history obtained from non-linear dynamic analysis of typical braced steel frames. Results were obtained for different cyclic loading and were used to characterize the hysteretic response, including energy dissipation capabilities of the frame.

The ductile behaviour of the braces under different earthquake ground loading are studied and used for design applying the codal procedures. Simplified models were obtained to predict plastic hinge failure and local buckling failure of bracing as a ductility failure mode. Finally, inelastic deformation capabilities are obtained before failure of moment resisting frame and bracing members.

Mahmoud Miri*et al.* (2009) studied on the effects of asymmetric bracing on steel structures under seismic Loads. The irregular distribution of stiffness and the mass of the structure was also might be asymmetric as an asymmetric bracing in plan and both the condition lead to eccentricity and torsion in the construction.

Due to the defect of ordinary code to evaluate the performance of steel structures against earthquake has been caused designing based on performance level to be used. The author mentioned that it is possible to design a structure and its behaviour against different earthquakes was anticipating. A five storey building with different percentage of asymmetric which is because of stiffness changes have been designed in this paper.

Deulkar*et al.* (2010) used five different configurations for their study shown below on the BRB system to help with vibration control. The projects compared the reduction in roof displacements obtained from analyses of different bracing configurations and found that the inverted V-bracing has the least roof displacement of the tested configurations.

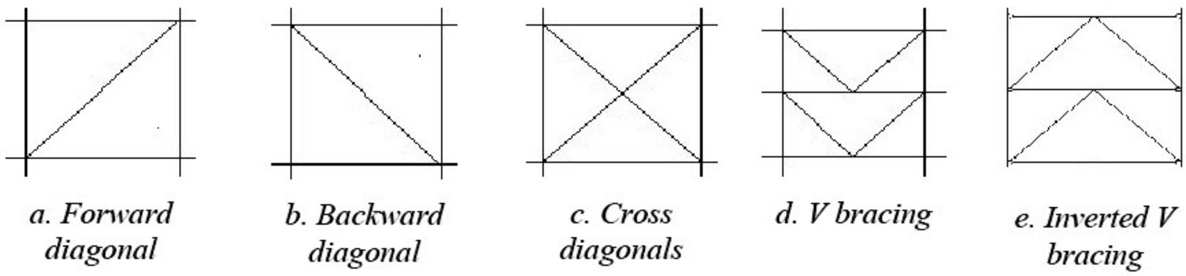

Fig. 2.1 Bracing systems

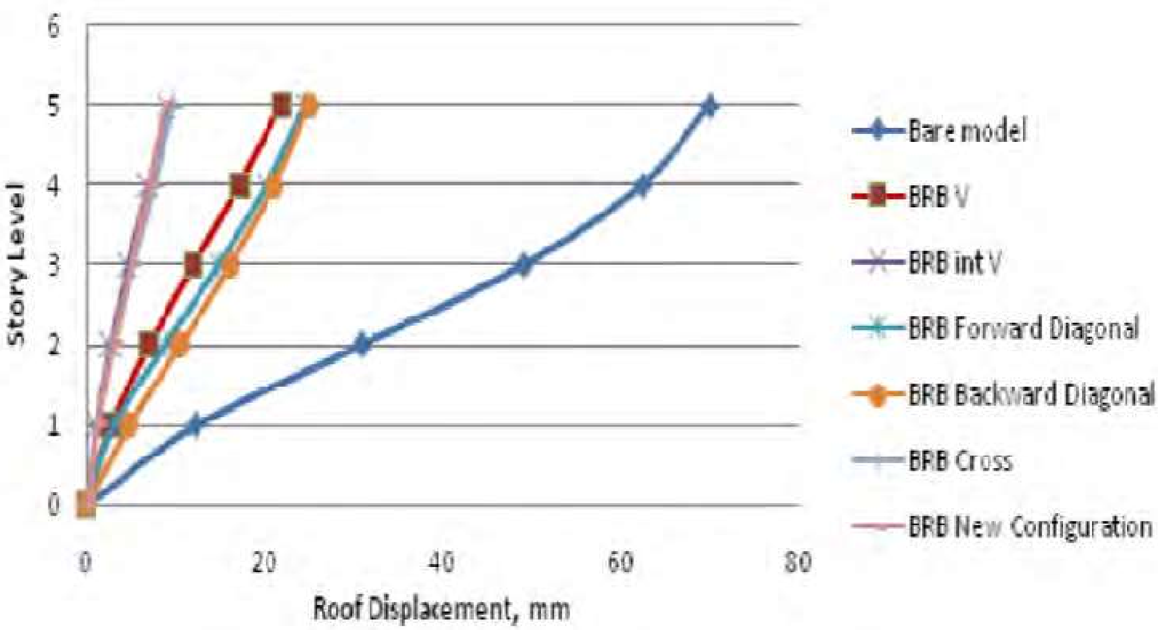

Fig. 2.2 Comparison of displacement between different types of BRB bracing configurations (Deulkar et. al., 2010)

Eghtesadi*et al.* (2011)has considered four types of bracing systems including X-bracing, Diagonal bracing, Inverted chevron CBF and Inverted chevron EBF, in four different height levels, were modelled and analysed. These models were compared in different aspects, such as economical viewpoint with evaluating the weight of the structure, the maximum top story displacement under seismic loading and the energy absorption and concluded that Inverted chevron CBF system has the high energy absorption capacity, the amount of steel used per unit area of the frame and the total weight of the structure was less than other types of bracing systems so applying the inverted chevron concentric bracing system may be proper and economical for the steel braced frames.

Manish *et al.* (2012) has done equivalent static analysis for steel moment resisting building frame having (G+9) storey situated in zone III. Modelling was done by using Response spectrum method. The steel moment resisting building frame was analysed with and without steel bracing system.

The main parameters considered to compare the seismic performance of buildings were bending moment, shear force, story drift and axial force and concluded that the braced building of the storey drift decreases as compared to the unbraced building which indicates that the overall response of the building decreases, the displacement of the building decreases depending upon the different bracing system employed and the bracing sizes, about the whole of performance of X braced building better than other types of braced building, also observed that as the size bracing section increases the displacements and storey drifts decreases for the braced buildings.

RafeelSabelli*et al.* (2013) gave the guide lines on seismic design of steel special concentrically braced frame systems. Due to the truss action generated by the braced frames, the lateral forces are effectively transferred to the foundation with well-defined energy dissipation system.
Braced frame action improves seismic characteristics like ductility, stiffness, energy dissipation, and decrease inter-storey drift of the structure.

ZasiahTafheem*et al.* (2013) studied on structural behaviour of steel building with concentric and eccentric bracing; a comparative study. In their study, a six storied steel building has been modelled and then analysed due to lateral seismic and wind loading, dead and live load.
The same steel building has been investigated for different types of bracing system such as concentric (crossed X) bracing and eccentric (V-type) bracing using HSS sections for knowing their performance.

The building performance has been evaluated in the terms of lateral storey displacement, total lateral displacement as well as axial force and bending moment in columns at different storey level. It has also been investigated for the effectiveness of various types of steel bracing on the structure. Importantly, the reduction in lateral displacement were been founded out for different types of bracing system in comparison to building with no bracing.

The author has been founded that the concentric X-bracing reduces more lateral displacement which significantly contributes to greater structural stiffness to the structure.

Mohammed Idrees Khan andMr.KhalidNayaz Khan (2014)has considered a typical15th-storey regular steel frame building was designed for various types of concentric bracings like Diagonal, V, X, and Exterior X and performance of each frame was carried outthrough nonlinear static analysis.

Three types ofsections i.e. ISMB, ISMC and ISA sections areused to compare for same patterns of bracingand concluded that the provision of bracingenhances the base shear carrying capacity offrames. The effects were more pronounced intaller structures, it was observed that due tobracing in both direction base shear capacity forV-Brace, Diagonal Brace, X-Brace, increases upto 40-50 % as compared with bare frame model,where as in Exterior X-Brace maximum baseshear increases up to 70 % as compared withbare frame model and ISMB Section givesmore base shear compare to angel and channelsection for similar type of brace also it isobserved that the displacement at roof level ofthe steel frame structure for V-Brace, DiagonalBrace, X-Brace, reduced up to 70-80 % ascompared with bare frame model, where as inExterior X-Brace maximum displacement alsoreduced up to 90 % as compared with bareframe model and ISMC Sections reduces moredisplacement compare to angel and beamsection for similar type of brace.

Siddiqi*et al.* (2014)has considered fivedifferent types of bracing systems andinvestigated for the use in tall building in order toprovide lateral stiffness and finally the optimizeddesign in terms of lesser structural weight andlesser lateral displacement.

For this purpose asixty storey regular shaped building wasselected and analysed for wind and gravity loadcombinations along both major and minor axesand concluded that lesser structural steel weightof a tall building was obtained when it wasbraced along the minor axis of bending ofcolumns in comparison of the situation whensame building was braced along the major axisof bending, among five different investigatedbracing systems, double bracing system yieldsminimum weight of structural steel, whencolumns were braced along their minor axis ofbending, provision of K bracing results inminimum value of lateral displacementcompared to other four types of bracingsystems, when columns were braced alongmajor axis, although lateral displacement valuesgo beyond the permissible limits but among fivetypes of bracing systems, which similar to thecase when columns are braced along their minoraxis of bending, K type bracing results in smallerlateral displacement compared to other types.

Nandi and Hiremath(2015) presented seismic performance of non-ductile buildings with eccentric steel bracing of inverted Y type was investigated. 10, 15 and 20 storey buildings were analysed by using pushover analysis. The analysis was carried out by using software SAP2000v17.

The effect of distribution of steel bracing over the height was studied and concluded that Energy absorption capacity is major requirement for every structure as EBF absorbs more energy as compared to bare and braced frames, Stiffness of building helps in resisting lateral force but more stiffness reduces energy absorption capacity as compared to bare and braced frame.

EBF provides moderate stiffness to building. Ductility is prime requirement for every building built in seismic zone. Increased area of bracing makes building stiffer and reduces ductility and energy absorption capacity of building and increased link length is vice-versa, EBF reduces all the seismic hazards efficiently hence EBFs are well suitable for seismic regions till 15storey.

Adithya*et al.* (2015)has considered a threedimensional structure with 4 horizontal bays ofwidth 4 meters, and 20 stories was taken withstorey height of 3m. The beams and columnswere designed to withstand dead and live loadonly. Wind and Earthquake loads weretaken by bracings.

The bracings were providedonly on the peripheral columns. Maximum of 4bracings were used in a storey for economicpurpose and studied the effects of various typesof bracing systems, its position in the buildingand cost of the bracing system with respect tominimum drift index and inter storey drift andfound that as per displacement criteria bracingswere good to reduce the displacement and themax reduction of 68.43% was observed insingle diagonal braces arranged as diamondshape in 3rd and 4th bay model compared tomodel without brace, the bending moment andshear force in columns were also reduced inbraced models and concluded that the concept of using steel bracing was one of theadvantageous concepts which can be used tostrengthen or retrofit the existing structures, thelateral storey displacements of the building weregreatly reduced by the use of single diagonalbracings arranged as diamond shape in 3rd and4th bay in comparison to concentric (X) bracingand eccentric (V) bracing system.

2.2 SCOPE OF PRESENT STUDY

Based on previous literature work, the behaviour of concentric braced frame system, braces and gusset plate connections, are better understood. This improved understanding can be translated into determination of response of concentric braced frames.

i. Study of displacement response of space steel frame under seismic and wind loadings in different bracing systems.
ii. Determine lateral drift at each floor level for Concentric Braced Frames (CBFs) for fully restrained column base.
iii. Study on moment and shear carrying capacity for different bracing systems in steel building.
iv. Determination of the most efficient bracing system in concentric braced frames.

CHAPTER 3

METHODOLOGY

Methodology is the systematic, theoretical analysis of the methods applied to a field of study. It comprises the theoretical analysis of the body of methods and principles associated with a branch of knowledge.

3.1 LOAD CONSIDERED

Dead loads

A load fixed in magnitude and in position is called a dead load.The dead load comprises of the weights of walls, partitions floor finishes, false ceilings, false floors and the other permanent constructions in the buildings. The dead load loads may be calculated from the dimensions of various members and their unit weights.

For floors; unit weight of reinforces cement concrete= 25 kN/m^3

Unit weight of steel is = 78.5 kN/m^3

Imposed loads

Imposed load is produced by the intended use or occupancy of a building including the weight of movable partitions, distributed and concentrated loads, load due to impact and vibration and dust loads. Imposed loads do not include loads due to wind, seismic activity, snow, and loads imposed due to temperature changes to which the structure will be subjected to, creep and shrinkage of the structure, the differential settlements to which the structure may undergo.

For residential buildings i.e. hostels

Hostels, hotels, boarding houses, lodging houses, dormitories, residential clubs:

Living rooms, bed rooms and dormitories = 4.0 kN/m^3 (IS: 875, Part 2- 1987)

Wind loads

The force on a structure arising from the impact of wind on it. As the height of building increases effect of wind increases. The wind normally blows horizontal to the ground at high wind speeds.

3.2 DESIGN OF WIND PRESSURE

The design wind pressure at any height above ground level shall be calculated by the following relationship between wind pressure and wind speed:-

P_Z=.6 V_Z^2

Where P_Z is design wind pressure in N/m^2 at height z and Vz is design wind velocity in m/s at height z,

Design Wind Speed (V_z)

The basic wind speed (V_z) for any site shall be calculated from and shall be modified to include the following effects to get design wind velocity at any height for the given structure:

a) Risk level;

b) Terrain roughness, height and size of structure; and

c) Local topography.

It can be mathematically expressed as follows:

V_Z= V_bxk_1xk_2xk_3

Where $V_b$is basic wind speed

k_1 is Probability factor (risk coefficient)

k_2 is terrain, height and structure size factor

k_3 is topography factor

Note: design wind speed up to 10m height from mean ground level shall be considered constant (IS: 875, Part 3- 1987)

Calculation of Wind Load:-

F = $\left(C_{pe} - C_{pi}\right) AP_z$

Where

C_{pe} = external pressure coefficient,

C_{pi} = internal pressure coefficient,

A = surface area of structural element or cladding unit, and

P_z = design wind pressure

Positive wind load indicates the force acting towards the structural element and negative away from it.

Seismic loads

When earthquakes occur, a buildings undergoes dynamic motion. This is because the building is subjected to inertia forces that act in opposite direction to the acceleration of earthquake excitations. These inertia forces, called seismic loads, are usually dealt with by assuming forces external to the building. Since earthquake motions vary with time and inertia forces vary with time and direction, seismic loads are not constant in terms of time and space. In designing buildings, the maximum story shear force is considered to be the most influential, therefore in this chapter seismic loads are the static loads to give the maximum story shear force for each story, i.e. equivalent static seismic loads. Time histories of earthquake motions are also used to analyze high-rise buildings, and their elements and contents for seismic design. The earthquake motions for dynamic design are called design earthquake motions

List of Indian Standards on Earthquake Engineering:-

1. IS 1893 (Part I), 2002: Indian Standard Criteria for Earthquake Resistant Design of Structures

2. IS 4326, 1993: Indian Standard Code of Practice for Earthquake Resistant Design & Construction of Buildings.

3. IS 13827, 1993: Indian Standard Guidelines for improving Earthquake Resistance of Earthen Buildings

4. IS 13828, 1993: Indian Standard Guidelines for Improving Earthquake Resistance of Low Strength Masonry Buildings

5. IS 13920, 1993 Indian Standard Code of Practice for Ductile Detailing of Reinforced Concrete Structures Subjected to Seismic Forces.

6. IS13935, 1993: Indian Standard Guidelines for Repair and Seismic Strengthening of Buildings

3.3 A REVIEW OF ANALYSIS (IS 1893 (PART I), 2002):-

Equivalent Static Analysis

Response Spectrum Analysis

Time History Analysis

Equivalent Static Analysis – An Overview

The equivalent static method is the simplest method of analysis. Here, force depend upon the fundamental period of structures defined by IS Code 1893:2002 with some changes. First, design base shear of complete building is calculated, and then distributed along the height of the building, based on formulae provided in code. Also, it is suitable to apply only on buildings with regular distribution of mass and stiffness.

Following are the major steps in determining the seismic forces:-

3.3.1 DETERMINATION OF BASE SHEAR

The total design lateral force or design base shear along any principal direction is determined by the expression:-

V = AW

Where,

A = design horizontal seismic coefficient for a structure

W = seismic weight of building

The design horizontal seismic coefficient for a structure A is given by:-

A = (ZISa)/ 2Rg

Z is the zone factor in Table 2 of IS 1893:2002 (part 1).

I is the importance factor

R is the response reduction factor; Sa/g is the average response acceleration coefficient for rock and soil sites as given in figure 2 of IS 1893:2002 (part 1). The values are given for 5% damping of the structure.

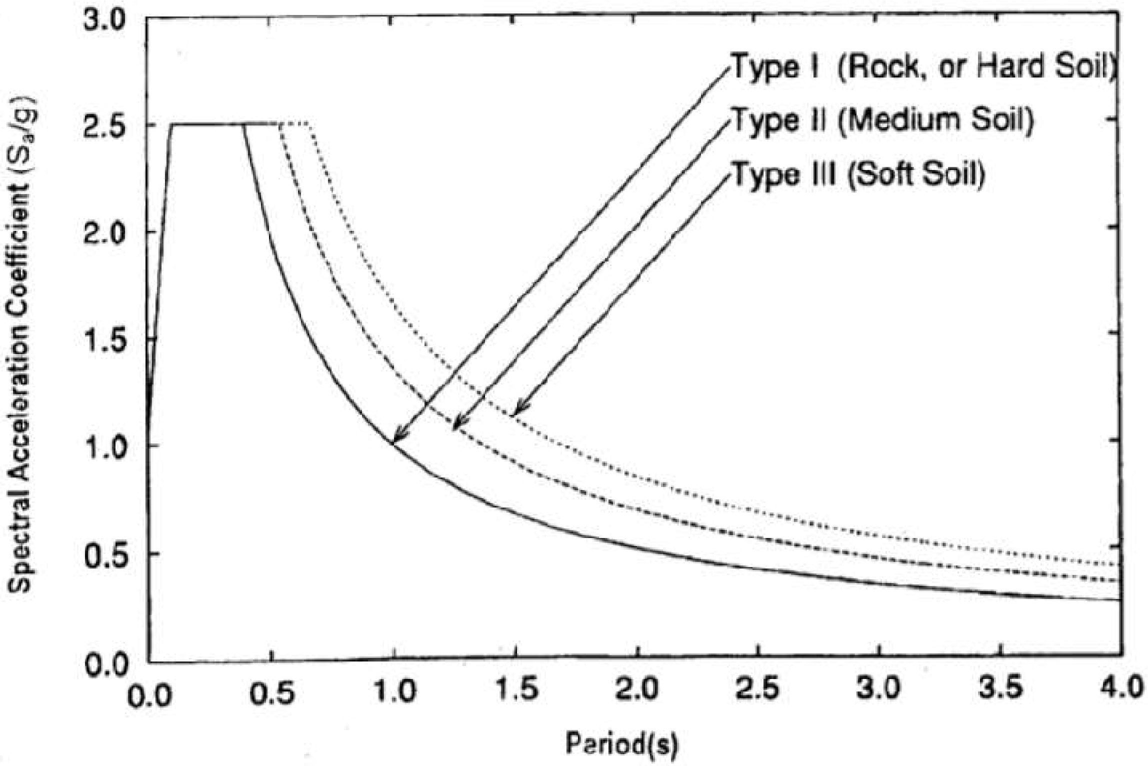

Fig. 3.1 Spectral Acceleration Coefficient Vs. Period

For rocky, or hard soil sites

$$\frac{s_a}{g} = \begin{Bmatrix} 1+15T; 0.00 \le T \le 0.10 \\ 2.50; 0.10 \le T \le 0.40 \\ 1.00/T; 0.40 \le T \le 4.00 \end{Bmatrix}$$

For medium soil sites

$$\frac{s_a}{g} = \begin{Bmatrix} 1+15T; & 0.00 \le T \le 0.10 \\ 2.50; & 0.10 \le T \le 0.55 \\ 1.36/T; & 0.55 \le T \le 4.00 \end{Bmatrix}$$

For soft soil sites

$$\frac{S_a}{g} = \begin{cases} 1+15T; & 0.00 \le T \le 0.10 \\ 2.50; & 0.10 \le T \le 0.67 \\ 1.67/T; & 0.67 \le T \le 4.00 \end{cases}$$

T is the fundamental natural period for buildings calculated as per clause 7.6 of IS 1893:2002 (part1).

Ta = $0.075h^{0.75}$ for moment resisting frame without brick infill walls

Ta = $0.085h^{0.75}$ for resisting steel frame building without brick infill walls

Ta = $0.09h/\sqrt{d}$ for all other buildings including moment resisting RC frames

h is the height of the building in m and d is the base dimension of building at plinth level in m.

3.3.2 LATERAL DISTRIBUTION OF BASE SHEAR

The total design base shear has to be distributed along the height of the building. The base shear at any story level depends on the mass and deformed shape of the building. Earthquake forces tend to deflect the building in different shapes, the natural mode shape which in turn depends upon the degree of freedom of the building. A lumped mass model is idealized at each floor, which in turn converts a multi storied building with infinite degree of freedom to a single degree of freedom in lateral displacement, resulting in degrees of freedom being equal to the number of floors.

The magnitude of lateral force at floor (node) depends upon:-

- Mass of that floor
- Distribution of stiffness over the height of the structure
- Nodal displacement in given mode

Distribution of base shear along the height is done according to this equation:-

$$Q_i = V_B \times \frac{W_i\, h_i^2}{\sum_{j=1}^{n} W_j\, h_j^2}$$

Where,

Q_i = Design lateral force at floor i,

W_i = Seismic weight of floor i

h_i = Height of floor *i* measured from base and

N = Number of storeys in the building at which the masses are located.

3.4 Load calculations

Loads and Load combinations are given as per Indian standards. (IS 875:1984, IS 1893:2002 and IS 800:2007*)*

Seismic Loading

Seismic load is given as per IS 1893- 2002. Following assumptions are used for the calculation.

Zone factor – 0.24

Soil type – 2 (medium Soil)

Importance Factor – 1.50

Response reduction – 4.00

Foundation Depth- 3.00 meter

Damping Ratio - 0.02

Wind loading

Wind load is given as per IS 875 (Part 3)-1987

Basic Wind speed – 47.00 m/s

Terrain category – 2

Class – B

Probability Factor ‘k1’ = 1.07

Terrain Height & Structure Size Factor ‘k2’

Topography Factor 'k3'

Dead loads

For floors; unit weight of reinforces cement concrete= 25 kN/m^3

Unit weight of steel = 78.5 KN/m^3

Assume depth of slab= 125 mm

Wall Self Weight = 5.00 KN/m

Floor Load

Slab Dead Weight = 3.13 Kn/m^2

Floor Finish = 0.75 Kn/m^2

Total Dead Floor Weight = 3.88 Kn/m^2

Imposed loads

For residential buildings i.e. hostels

Hostels, hotels, boarding houses, lodging houses, dormitories, residential clubs:

Living rooms, bed rooms and dormitories = 4.0 kN/m^2 (IS: 875, Part 2- 1987)

3.4.1 LOAD COMBINATIONS

1) 1.5 (DL+ IL)
2) 1.2 (DL+ IL $\pm$ EL)
3) 0.9 DL$\pm$ 1.5 EL
4) 1.2 (DL+ IL $\pm$ WL)
5) 0.9 (DL$\pm$ 1.5 WL)

CHAPTER 4

ANALYSIS OF MODELS

4.1 GENERAL

The building used for this study was designed to the standards presented by the IS 800: 2007 and IS 1893 (Part 1): 2002

A multi-story steel building is analysed in STAAD Pro.

The design of the building is dependent upon the minimum requirements as prescribed in the Indian Standard Codes. The minimum requirements pertaining to the structural safety of buildings are being covered by way of laying down minimum design loads which have to be assumed for dead loads, imposed loads, wind loads and other external loads, the structure would be required to bear.

4.2 STEEL FRAMES

The frame used for this study is a 20 (G+19) storey, steel braced structures. The typical floor height is 3 m with a total 60 m of the building. In plan, the sides span 20 meter by 20 meter divided into 5 meter square bays as shown in figure 4.1

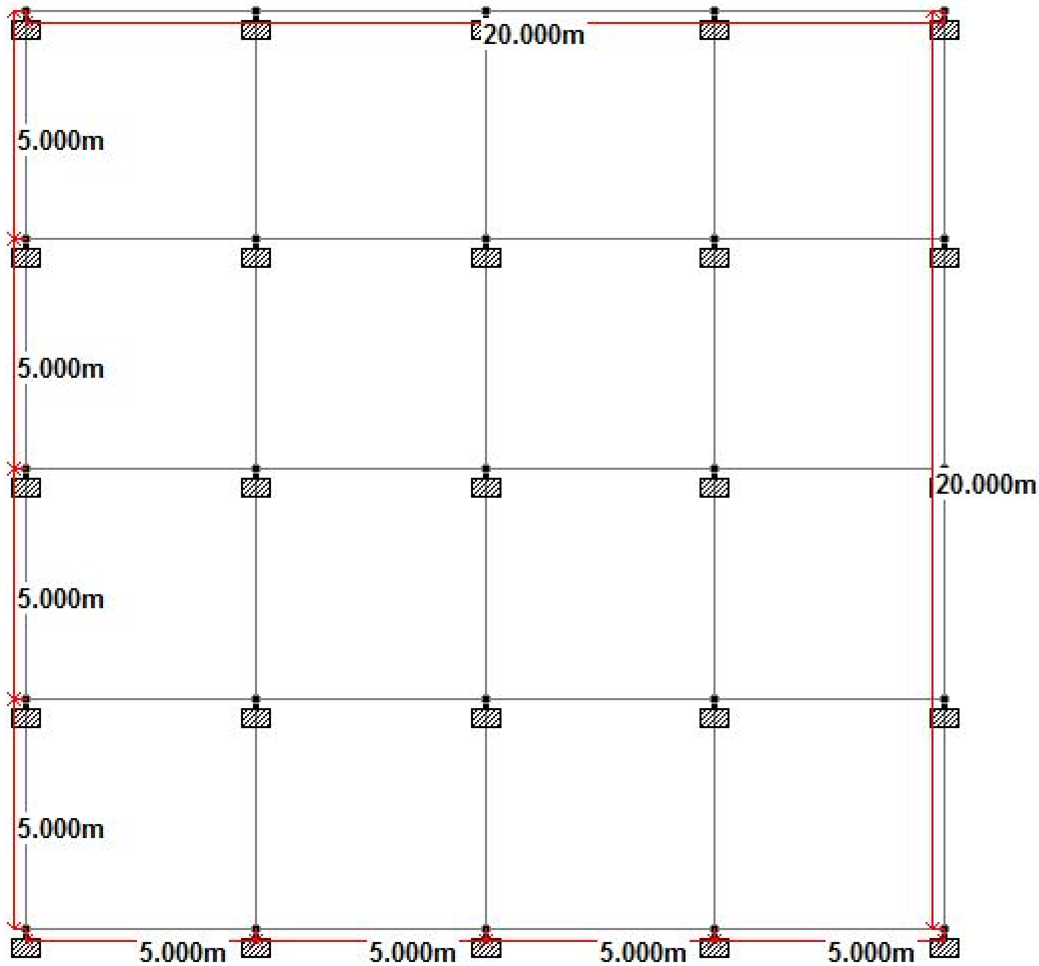

Fig. 4.1 Structural Floor Plan Of Steel Concentric Frame

Following models are considered for the analysis:

1. Without Brace model
2. Forward Analysis
3. Backward Analysis
4. brace model
5. Inverted V-brace model
6. K- brace model
7. V- brace model

Model 1:- Steel frame without bracing

Steel Concentric Frame with a plan area 20.0mX20.0m is modelled and analysed in STAAD Pro. Consider that the steel frame is located in seismic zone 3 (Z= 0.24) as per IS 1893 (Part 1): 2002, also at medium soil site.

Steel section chosen for columns and beams 180012A40012 for column and for beam 180016A50020

Elevation view of the concentric frame as shown in the figure 4.2

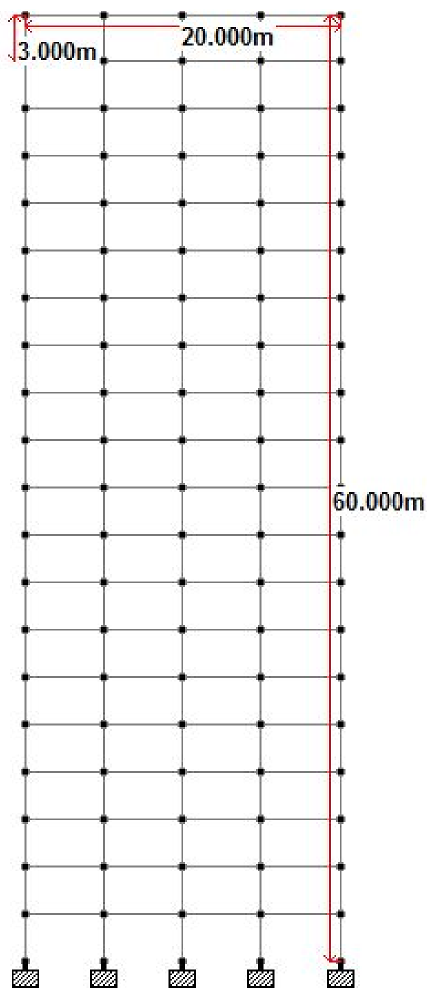

Fig. 4.2 Elevation of the Steel frame without bracing

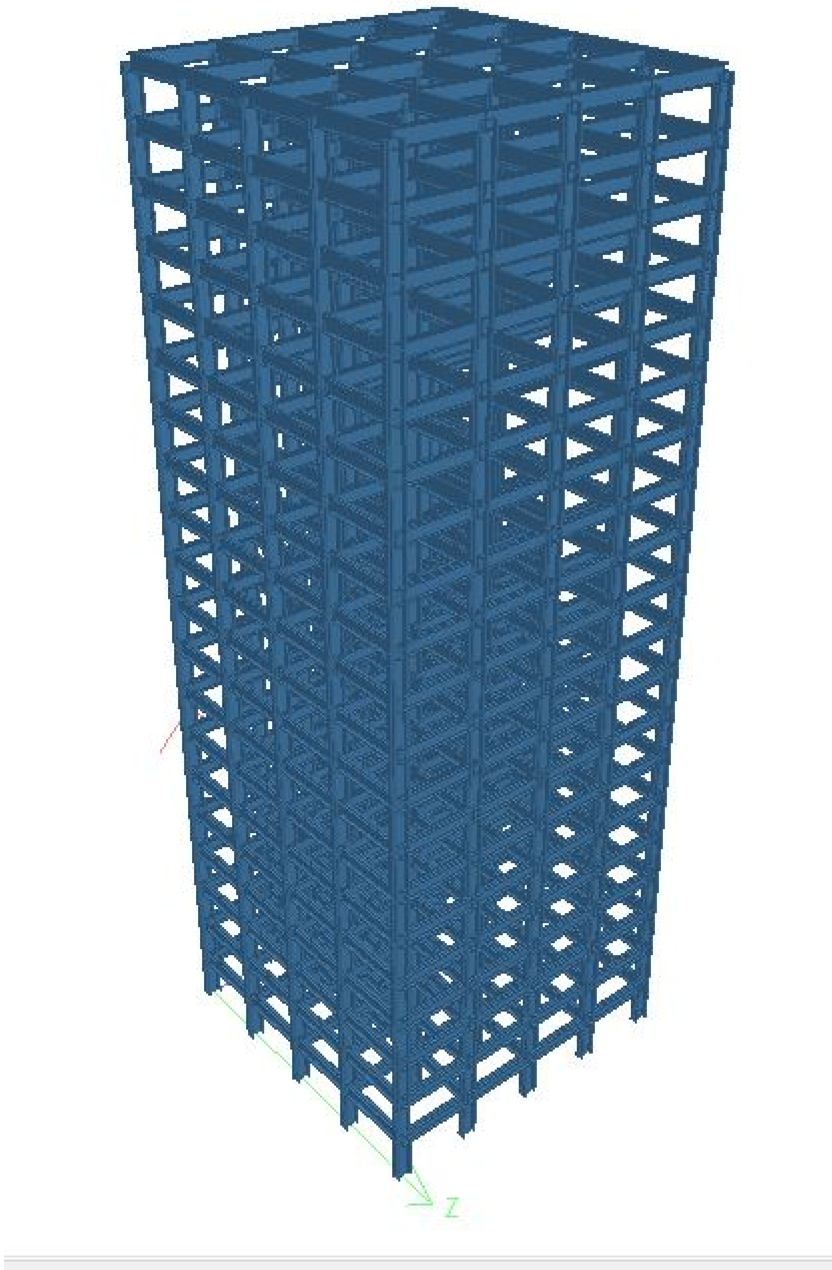

Fig. 4.3 3D view of steel building without bracing

This is a STADD 3D model of G+19 steel building without bracing system. All supports are fixed.

Steel section chosen for columns and beams 180012A40012 for column and for beam 180016A50020

Total weight of steel used is 18256.156 KN

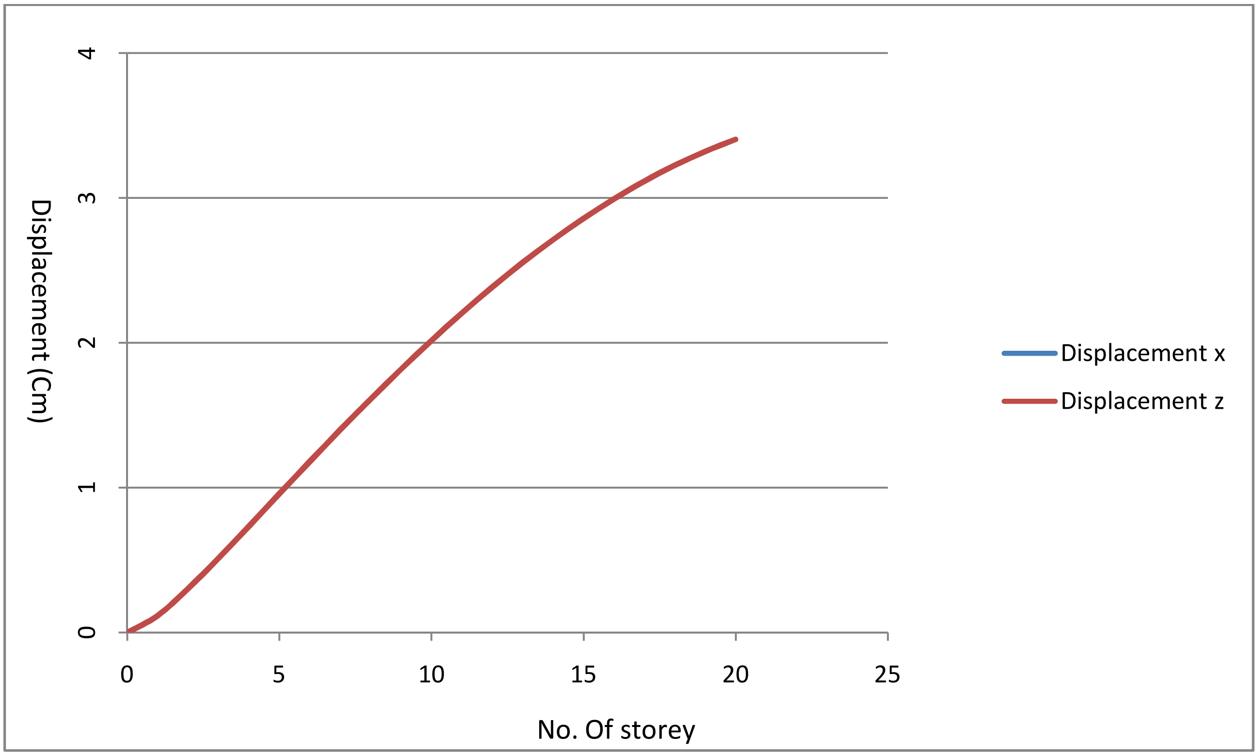

Fig. 4.4Storey displacement in X & Z - direction without bracing

It is observed from fig. 4.4 that as the height of building increases displacement is increasing. The maximum storey displacement is at the top floor that is 34.036 mm and minimum is at first floor that is 1.163 mm.

The maximum storey displacement is 16.677 mm is more with reference to backward bracing. And 0.77 more at first floor with reference to backward bracing.

Model 2:- Steel frame with backward bracing

All supports are fixed and steel section used for beams are I100012A50012 and columns are I100012B50040 and for bracing system it is ISWB 600

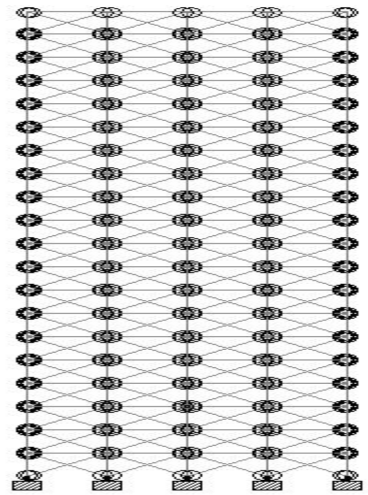

Figure 4.5 Elevation of Backward Bracing

Totel weight of steel used is 24011.502 KN

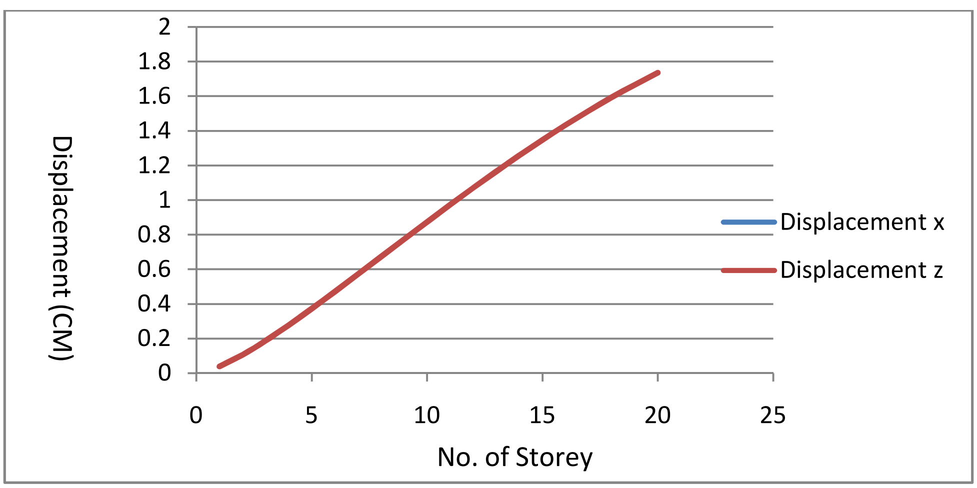

Fig. 4.6Storey displacement in X & Z - direction Backward Bracing

It is observed from fig. 4.6 that as the height of building increases displacement is increasing

The maximum storey displacement is at the top floor that is 17.359 mm and minimum is at first floor that is 0.3833mm.

Model :- 3 Steel Frame with forward bracing

All supports are fixed and steel section used for beams are I80012B50012 and columns are I125012A50020 and for bracing system it is ISJC 200

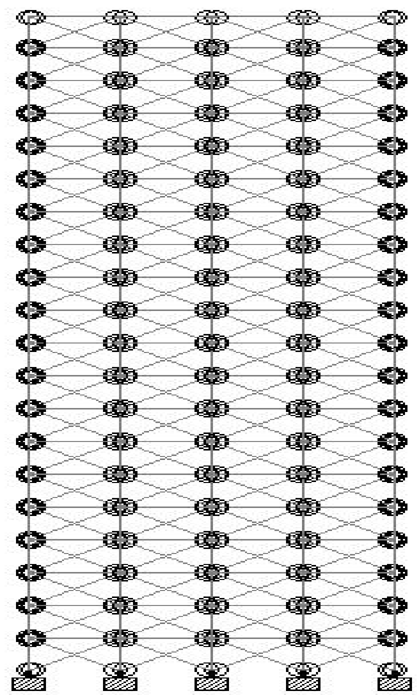

Figure 4.7 Elevation of Forward Bracing

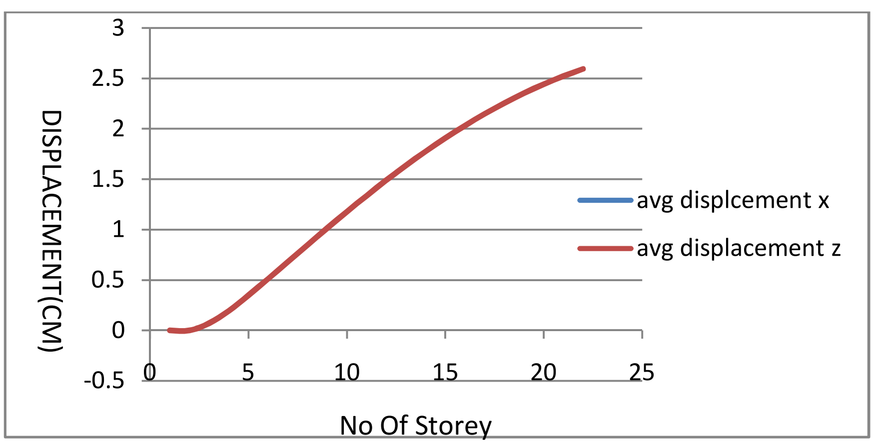

Fig. 4.8Storey displacement in X & Z - direction Forward Bracing

It is observed from fig. 4.8 that as the height of building increases displacement is increasing

The maximum storey displacement is at the top floor that is 25.93 mm and minimum is at first floor that is 0.719mm

Model 2:-Steel frame with X- bracing

All supports are fixed and steel section used for beams are I80016B50012 and columns are I100012B55025 and for bracing system it is ISMC200H

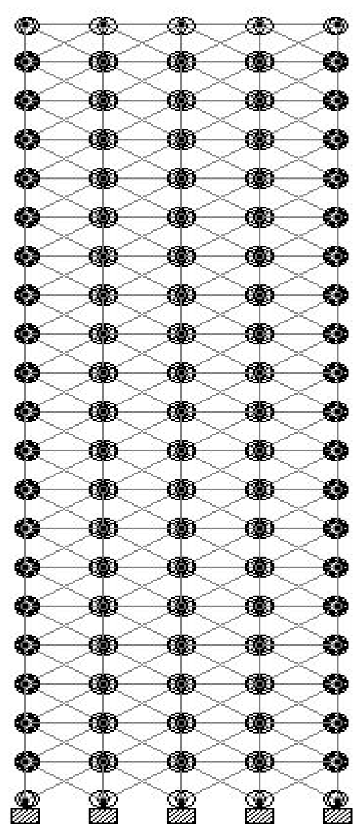

Fig. 4.9 Elevation of the Steel frame with X- bracing

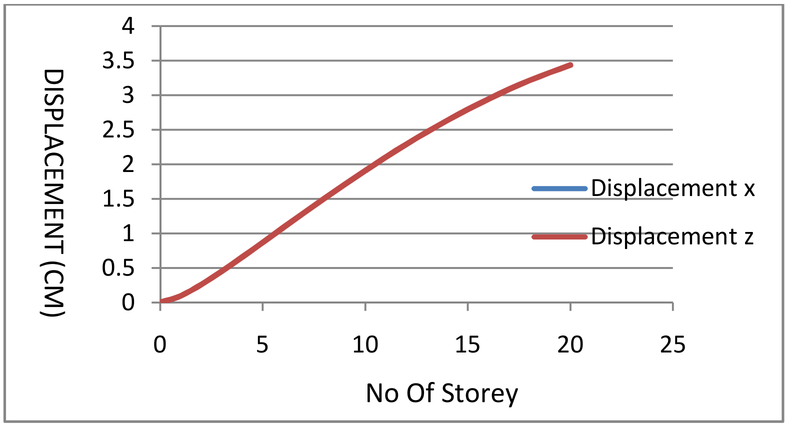

Fig. 4. 10 Storey displacement in X & Z - direction with X- bracing

It is observed from fig. 4.10 that as the height of building increases displacement is also increasing. The maximum storey displacement is at the top floor that is 34.357 mm and minimum is at first floor that is 0.977 mm.

Model 3:- Steel frame with inverted V- bracing

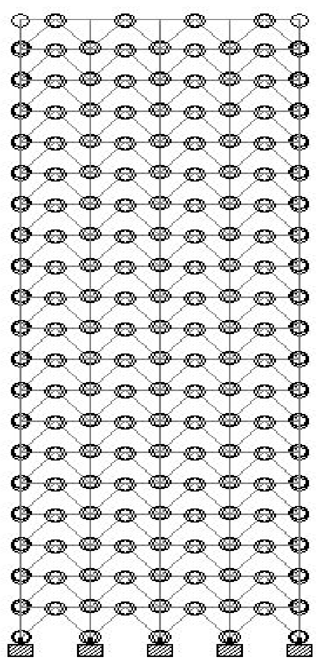

Fig. 4.11 Elevation of the Steel frame with inverted V- bracing

In the above fig. 4.11, elevation of steel frame is shown when it is subjected to inverted V-bracing from all outer sides. All supports are fixed and steel section used for beams are 1800400A50012 and columns are 180016A50020 and for bracing system it is ISMC 200H

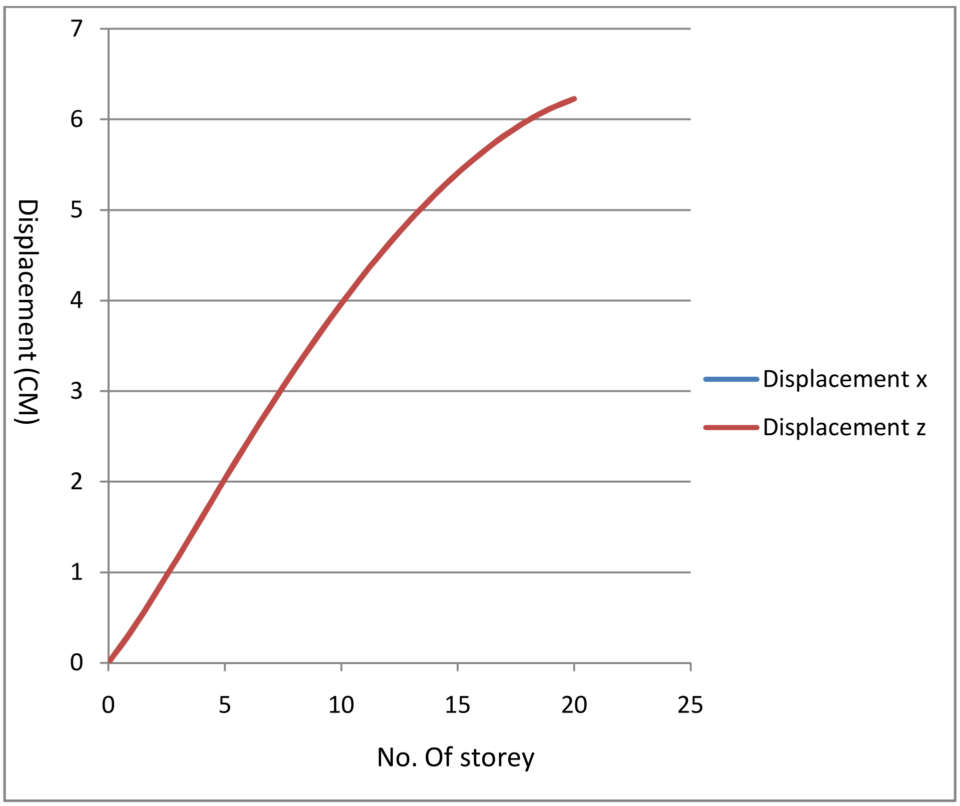

Fig. 4.12 Storey displacement in X & Z- direction with inverted V- bracing

It is observed from fig. 4.12 that as the height of building increases displacement is also increasing. The maximum storey displacement is at the top floor that is 62.287 mm and minimum is at first floor that is 3.553mm

Total weight of steel used is 17230.682 KN

Model 4:- Steel frame with K- bracing

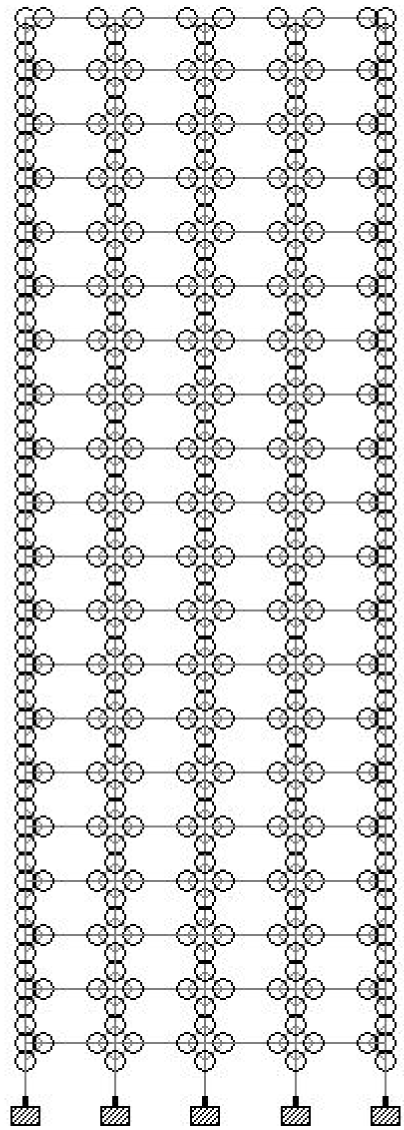

Fig. 4.13 Elevation of the Steel frame with K- bracing

In the above fig. 4.13, elevation of steel frame is shown when it is subjected to inverted V-bracing from all outer sides. All supports are fixed and steel section used for beams are I80012B50012 and columns are I80012A40012 and for bracing system it is ISA 100×100×6

K-bracing is symmetric for steel building as shown in above figure.

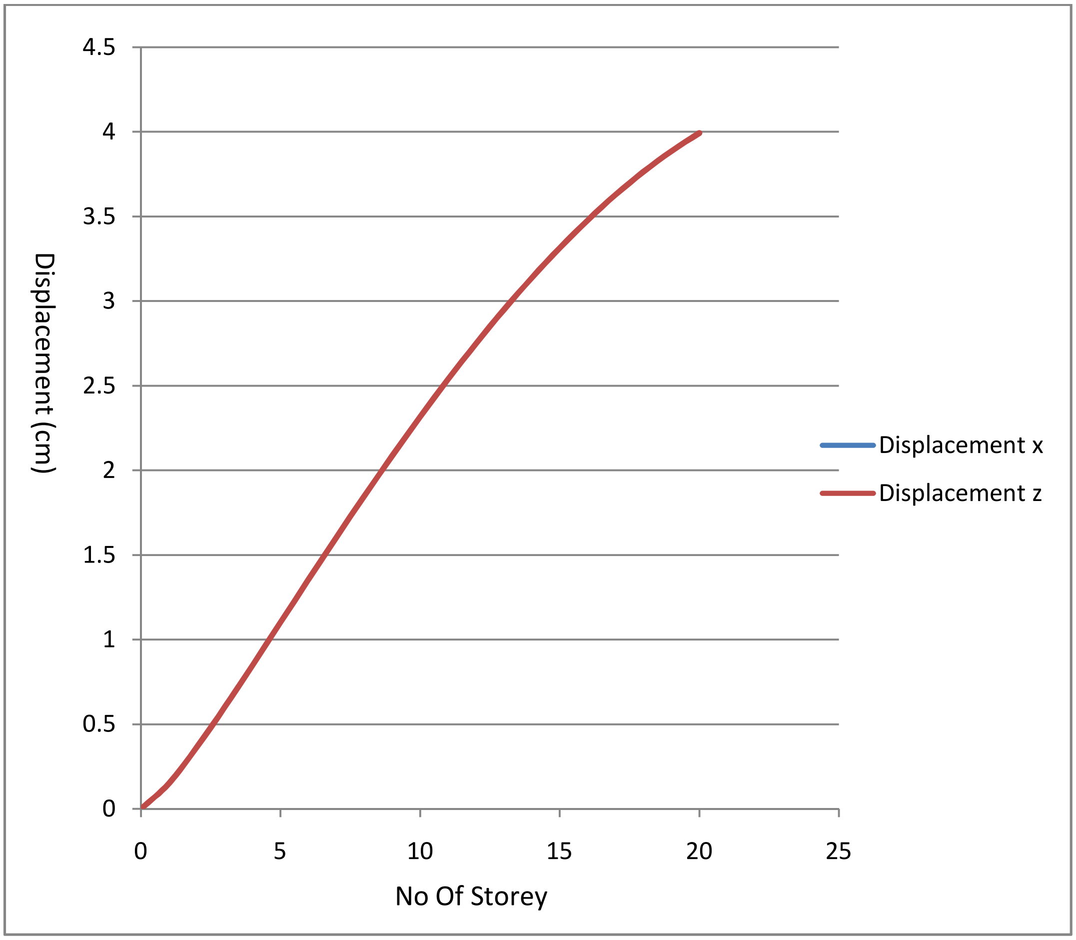

Fig. 4. 14Storey displacement in X & Z - direction with K- bracing

It is observed from fig. 4.14 that as the height of building increases displacement is also increasing. The maximum storey displacement is at the top floor that is 39.924 mm and minimum is at first floor that is 1.501 mm.

Total weight of steel used is 16601.846 KN

Model 5:- Steel frame with V- bracing

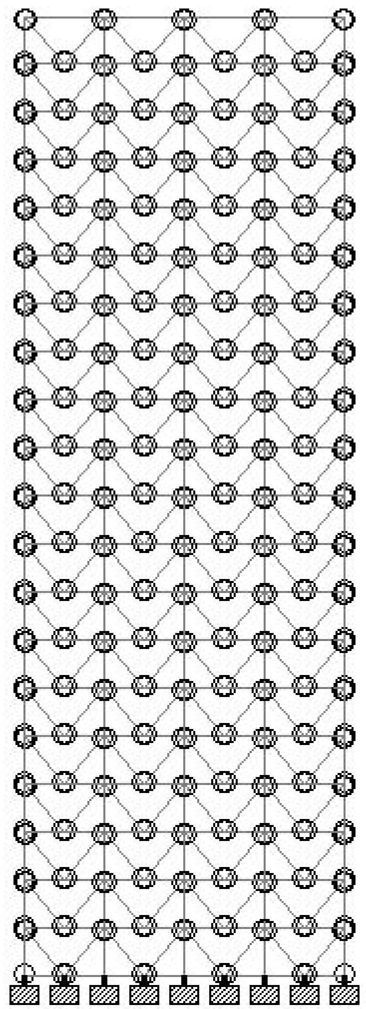

Fig. 4.15 Elevation of the Steel frame with V- bracing

In the above fig. 4.15, elevation of steel frame is shown when it is subjected to V-bracing from all outer sides. All supports are fixed and steel section used for beams are I80012A40012 and columns are I100012A40012 and for bracing system it is ISA 80×80×10

.

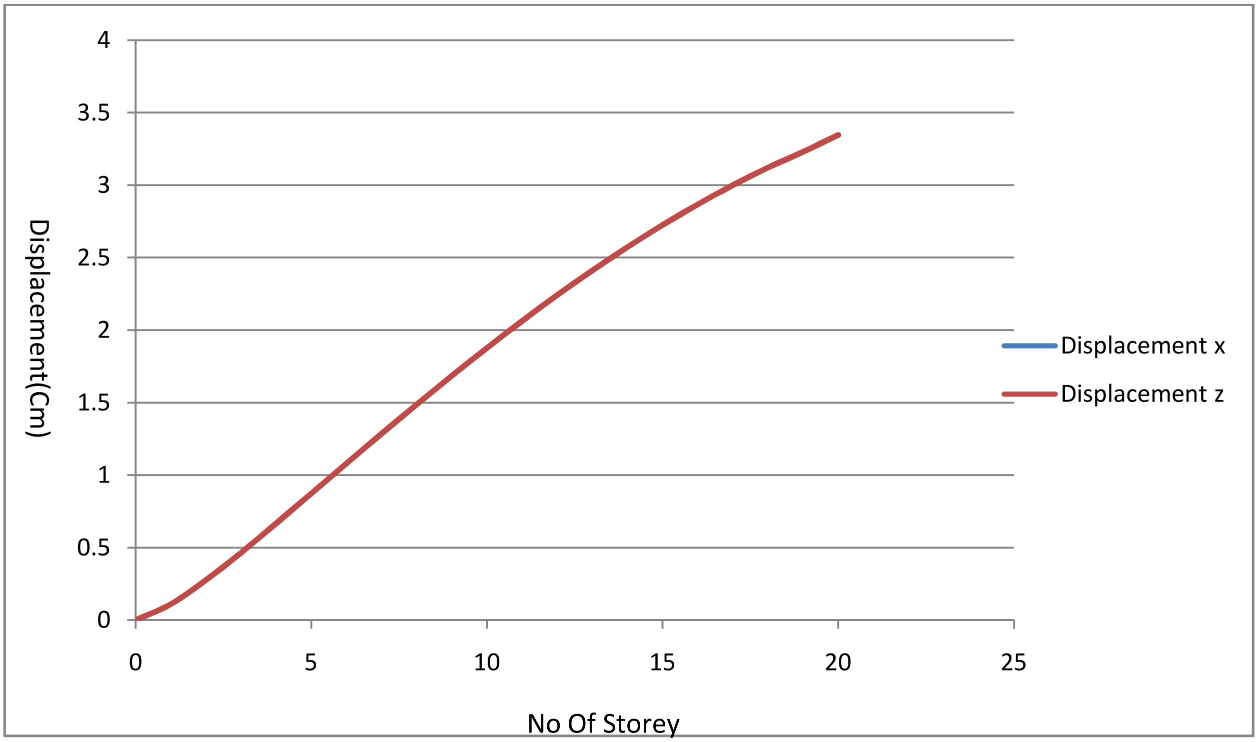

Fig. 4. 16 Storey displacement in X & Z - direction with V- bracing

It is observed from fig. 4.12 that as the height of building increases displacement is also increasing. The maximum storey displacement is at the top floor that is 33.485 mm and minimum is at first floor that is 1.094 mm.

Total weight of steel used is 17862.372 KN

We can also see in this case that top storey displacement for inverted V-bracing is less as compared to without bracing but more as compared to X-bracing.

CHAPTER 5

RESULTS AND DISCUSSIONS

5.1 STORY DISPLACEMENT

Storey lateral displacement at each floor level in X-direction is presented in the table from 5.1 to 5.5

Table 5.1Storey displacement in X & Z-direction without bracing at different levels

Storey	Displacement X	Displacement Z
0	0	0
1	0.1163	0.1163
2	0.3029	0.3029
3	0.5128	0.5128
4	0.7333	0.7333
5	0.9572	0.9572
6	1.1796	1.1796
7	1.398	1.398
8	1.6108	1.6108
9	1.8167	1.8167
10	2.015	2.015
11	2.2045	2.2045
12	2.384	2.384
13	2.553	2.553
14	2.711	2.711
15	2.8575	2.8575
16	2.9923	2.9923
17	3.1149	3.1149
18	3.224	3.224
19	3.3197	3.3197
20	3.4036	3.4036

Table 5.2Storey displacement in X & Z-direction with X- bracing at different levels

Storey	Displacement X	Displacement Z
0	0	0
1	0.0977	0.0977
2	0.2621	0.2621
3	0.4522	0.4522
4	0.6588	0.6588
5	0.8711	0.8711
6	1.0847	1.0847
7	1.2972	1.2972
8	1.5067	1.5067
9	1.7119	1.7119
10	1.9119	1.9119
11	2.1053	2.1053
12	2.2912	2.2912
13	2.4687	2.4687
14	2.6374	2.6374
15	2.7967	2.7967
16	2.9461	2.9461
17	3.0853	3.0853
18	3.2129	3.2129
19	3.3292	3.3292
20	3.4357	3.4357

Table 5.3Storey displacement in X & Z-direction with inverted V- bracing

Storey	Displacement X	Displacement Z
0	0	0
1	0.3553	0.3553
2	0.756	0.756
3	1.1709	1.1709
4	1.5984	1.5984
5	2.0284	2.0284
6	2.446	2.446
7	2.8505	2.8505
8	3.2391	3.2391
9	3.6108	3.6108
10	3.9648	3.9648
11	4.299	4.299
12	4.6109	4.6109
13	4.8999	4.8999
14	5.1657	5.1657
15	5.4079	5.4079
16	5.6262	5.6262
17	5.8204	5.8204
18	5.9862	5.9862
19	6.122	6.122
20	6.2287	6.2287

Table 5.4Storey displacement in X & Z-direction with K- bracing at different level

Stroey	Displacement X	Displacement Z
0	0	0
1	0.1501	0.1501
2	0.365	0.365
3	0.6012	0.6012
4	0.8498	0.8498
5	1.103	1.103
6	1.3552	1.3552
7	1.6043	1.6043
8	1.8484	1.8484
9	2.0862	2.0862
10	2.3166	2.3166
11	2.538	2.538
12	2.7491	2.7491
13	2.9493	2.9493
14	3.1377	3.1377
15	3.314	3.314
16	3.4776	3.4776
17	3.6284	3.6284
18	3.7645	3.7645
19	3.8856	3.8856
20	3.9924	3.9924

Table 5.5Storey displacement in X & Z-direction with V- bracing at different levels

Storey	Displacement X	Displacement Z
0	0	0
1	0.1094	0.1094
2	0.276	0.276
3	0.4642	0.4642
4	0.6671	0.6671
5	0.8737	0.8737
6	1.0802	1.0802
7	1.2856	1.2856
8	1.4875	1.4875
9	1.6852	1.6852
10	1.8781	1.8781
11	2.0642	2.0642
12	2.2426	2.2426
13	2.4127	2.4127
14	2.574	2.574
15	2.7261	2.7261
16	2.8685	2.8685
17	3.0015	3.0015
18	3.1225	3.1225
19	3.2319	3.2319
20	3.3485	3.3485

Table 5.6 Storey displacement in X & Z-direction with forward bracing at different levels

Storey	Displacement X	Displacement Z
0	0	0
1	0.0719	0.0719
2	0.1937	0.1937
3	0.3497	0.3497
4	0.5126	0.5126
5	0.6804	0.6804
6	0.8487	0.8487
7	1.015	1.015
8	1.1778	1.1778
9	1.3361	1.3361
10	1.489	1.489
11	1.6357	1.6357
12	1.7753	1.7753
13	1.9074	1.9074
14	2.0315	2.0315
15	2.1472	2.1472
16	2.2545	2.2545
17	2.3528	2.3528
18	2.4416	2.4416
19	2.5211	2.5211
20	2.593	2.593

Table 5.7 Storey displacement in X & Z-direction with Backward bracing at different levels

Storey	Displacement X	Displacement Z
1	0.0383	0.0383
2	0.1056	0.1056
3	0.1867	0.1867
4	0.2766	0.2766
5	0.3721	0.3721
6	0.4705	0.4705
7	0.5708	0.5708
8	0.6718	0.6718
9	0.7729	0.7729
10	0.8734	0.8734
11	0.9727	0.9727
12	1.0701	1.0701
13	1.1653	1.1653
14	1.2577	1.2577
15	1.347	1.347
16	1.433	1.433
17	1.5152	1.5152
18	1.593	1.593
19	1.6664	1.6664
20	1.7359	1.7359

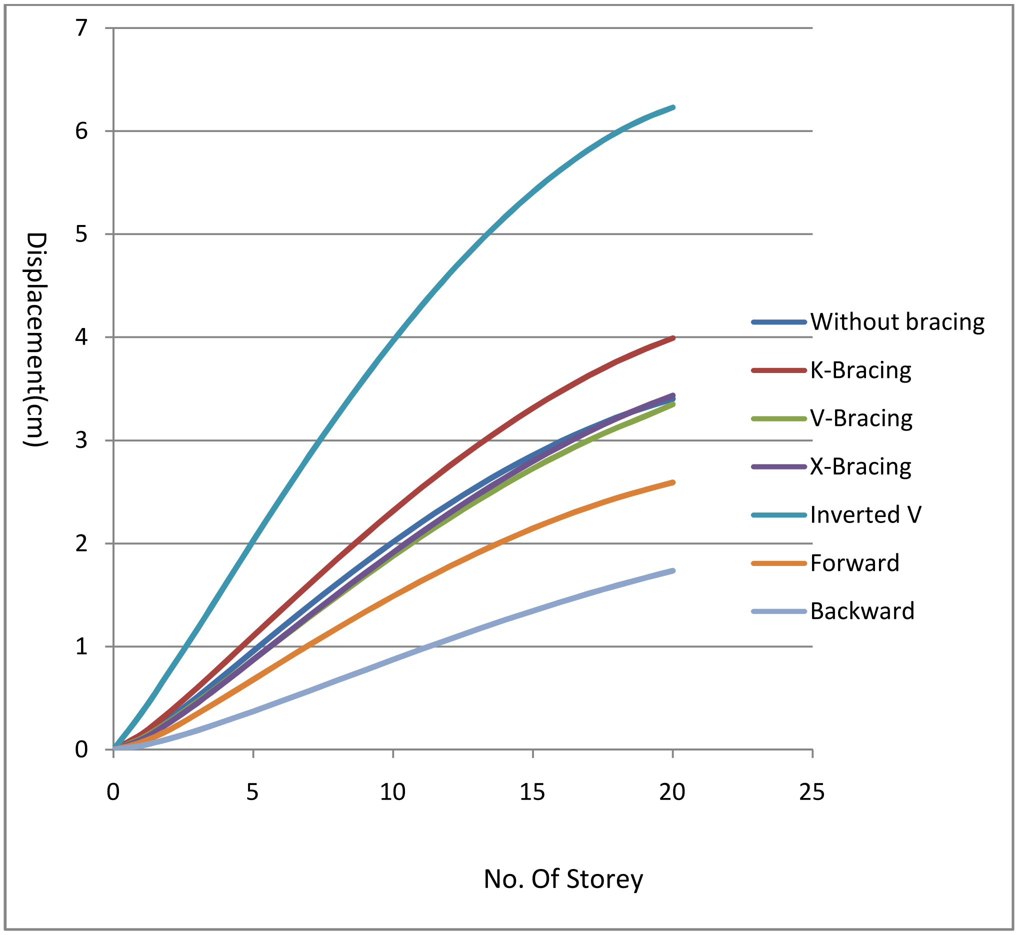

Fig. 5.1 Comparison of displacements in X & Z-direction for different bracing system

It is observed from figure 5.1 that as the number of storey increases, displacement increases for without bracing and with bracing. The maximum top storey displacement for without bracing is maximum that is 47.929 mm and minimum for X-bracing that is 12.491 mm.

Displacement at first floor for without bracing is 4.802 mm and for X-bracing, it is 1.52 mm.

5.2 STOREY SHEAR

Table 5.8 Storey shear for different bracing system

storey	backward	forward	k bracing	inverted v	v bracing	x bracing	without bracing
20	3854.71	2883.43	2194.1	2824.19	2422.14	3519.44	2577.01
19	9448.03	7013.02	9448.03	6829.74	5890.32	8552.01	5890.32
18	14564.25	10634.01	14564.25	10331.35	8942.32	13045.56	7964.64
17	19171.79	13719.09	19171.79	13309.98	11566.69	16965.78	10125.17
16	23302.21	16324.99	23302.21	15809.23	13815.37	20352.73	11877.4
15	26996.92	18513.47	26996.92	17887.38	15746.26	23259.59	13318.47
14	30289.85	20350.86	30289.85	19626.53	17421.48	25746.15	14550.15
13	33216.55	21919.81	33216.55	21110.86	18895.56	27888.29	15650.84
12	35827.19	23299.73	35827.19	22413.23	20218.93	29771.88	16670.81
11	38193.73	24557.88	38193.73	23604.83	21444.7	31486.72	17632.95
10	40397.73	25758.68	40397.73	24746.91	22609.83	33121.24	18549.86
9	42506.04	26955.16	42506.04	25886.13	23734.76	34745.03	19428.12
8	44546.52	28176.21	44546.52	27056.7	24836.55	36393.39	20273.54
7	46511.1	29433.49	46511.51	28271.16	25922.47	38067.89	21102.04
6	48368.75	30721.31	48368.75	29519.42	26989.45	39738.6	21930.34
5	50071.61	32004.45	50071.61	30707.96	28025.84	41348.8	22774.37
4	51567.77	33225.25	51567.77	31957.54	29004.47	42818.31	23629.09
3	52795.2	34308.63	52795.2	33014.54	29881.87	44066.52	24452.17
2	53667.91	35137.53	53667.91	33832.28	30574.33	44975.71	25154.04
1	53854.05	35313.89	53854.05	34004.99	30722.9	45162.93	25322.45
0	53854.05	35313.89	53854.05	34004.99	30722.9	45162.93	25327.6

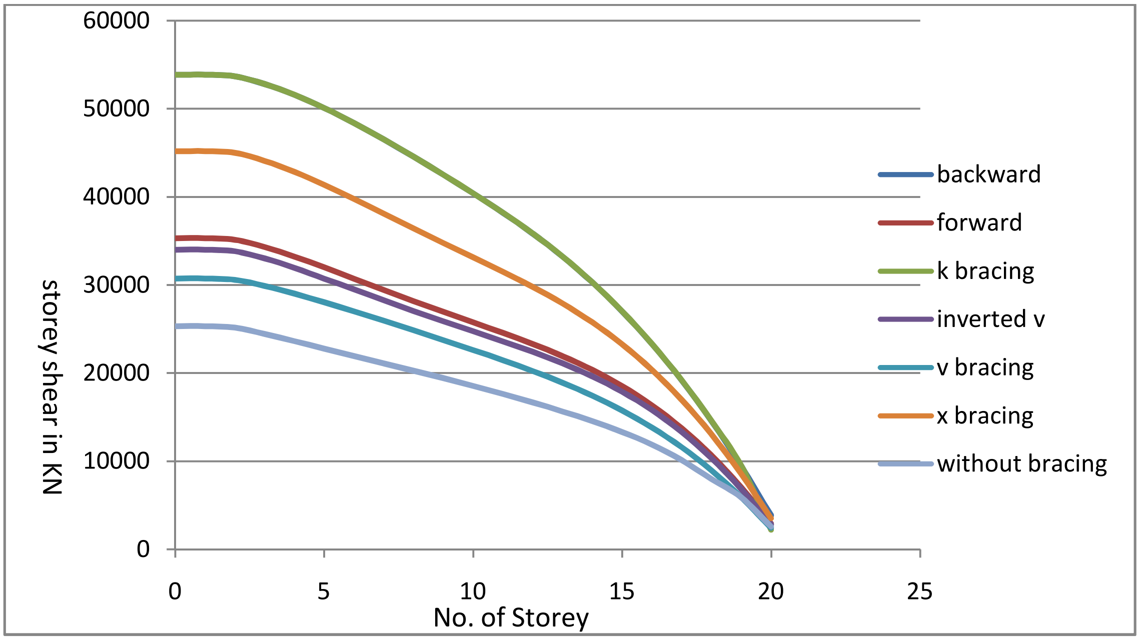

Fig. 5.2 Comparison of Storey Shear for different bracing system

5.2 SHEAR FORCE

Table 5.9 Maximum Shear Force

Models	**Shear Force(kN)**
Without bracing	37826.570
X-bracing	59496.387
Inverted-V bracing	45395.121
K-bracing	36305.188
V-bracing	42193.262
Backward Bracing	71376.016
Forward Bracing	46996.742

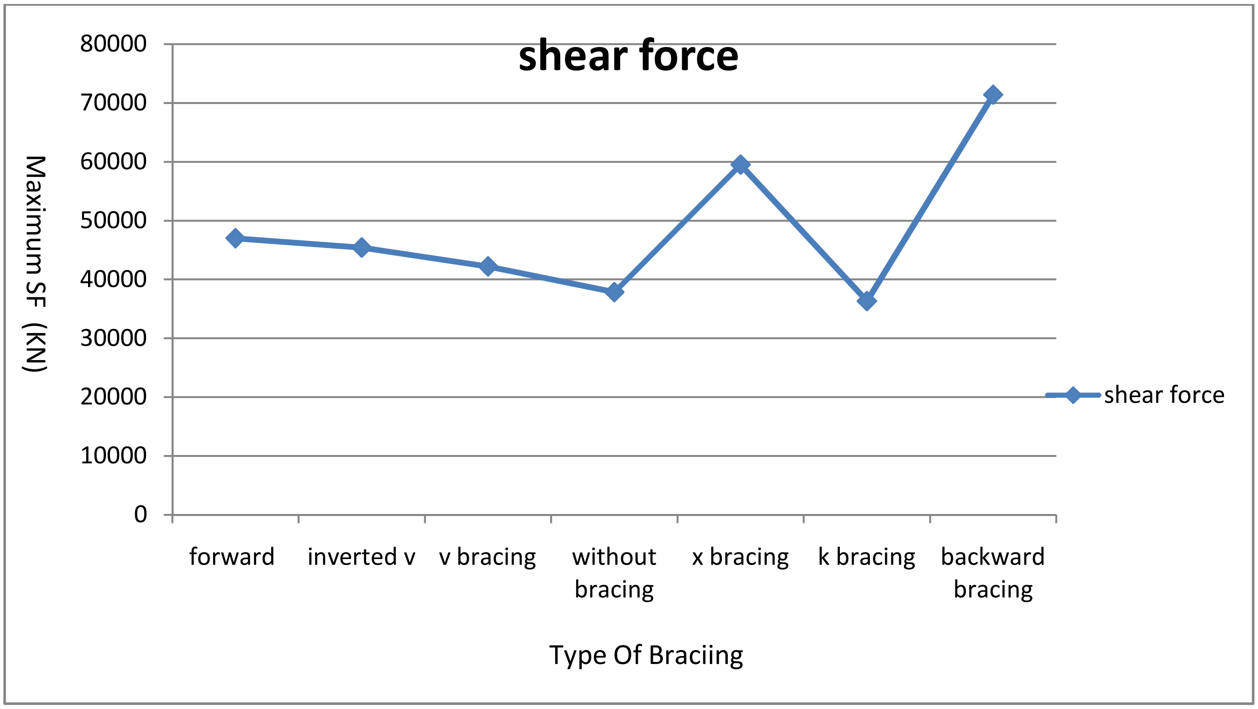

Fig. 5.3 Comparison in Shear Force for different bracing systems

Maximum shear force for steel building for without bracing and with bracing is shown in figure 5.3. Maximum shear force found for backward bracing that is 71376.016 kN and minimum for without bracing that is 37826.570 kN.

For Backward -bracing, shear force is 71376.016 kN and difference between without bracing and backward bracing is less.

5.3 BENDING MOMENTS

Table 5.10 Bending Moments

Models	Bending moments(kN-m)
Without bracing	7459.444
X-bracing	8551.276
Inverted-V bracing	6867.884
K-bracing	5784.231
V-bracing	7021.314
Backward bracing	10266.527
Forward bracing	10606.850

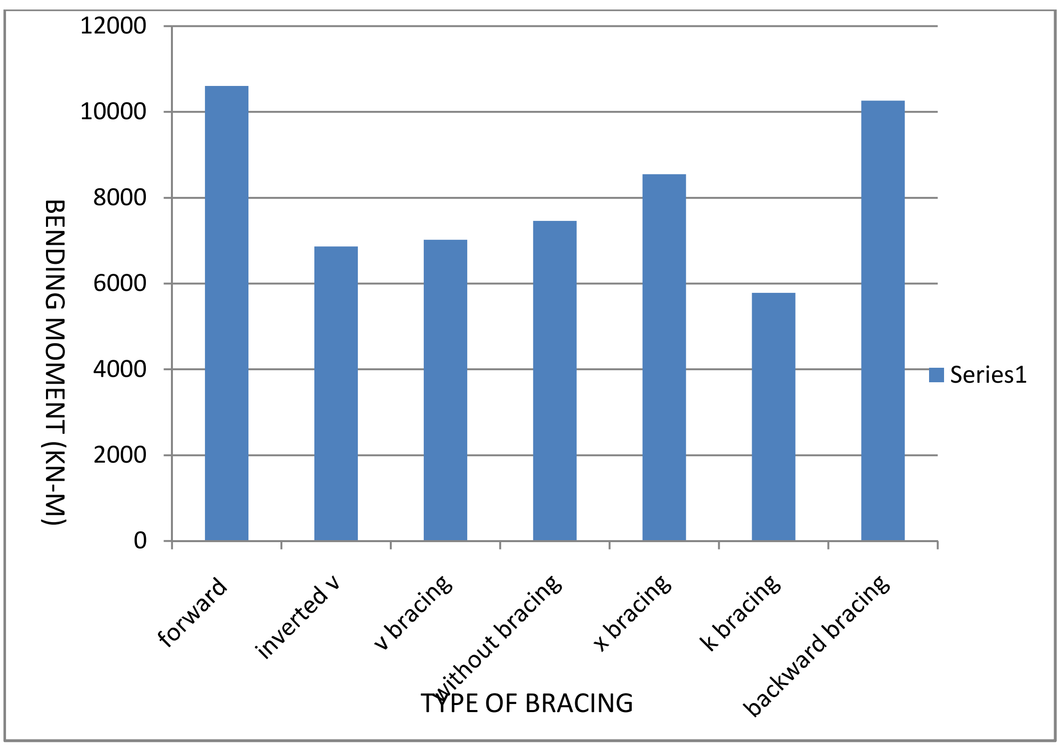

Fig. 5.4 Comparison in Bending moments for different bracing systems

Maximum bending moment M_y and M_z (whichever is greater) is shown in figure 5.4 for steel building for without bracing and with bracing.Maximum bending moment found for Forward bracing that is 10606.850 kN-m and minimum for K-bracing that is 5784.231 kN-m.

For Forward bracing, maximum banding moment is 10606.850 kN-m.

CHAPTER 6

CONCLUSIONS

6.1 GENERAL

This paper has presented a general review of structural systems for tall buildings.Unlike the height-based classifications in the past, a system-based broad classification has been proposed. Various structural systems within each category of the new classification have been described with emphasis on innovations.

Whenever a structure is provided with Bracings though it may concentric or eccentric then it gives more resistance to lateral deflection and also it suitable in earthquake prone areas. The performance of the building has been evaluated in terms of lateral storey displacement, storey drift as well as axial force and bending moment in columns at different storey level. Improvement can be achieved by redesigning the brace and floor beams to a weak brace and strong beam system, as in Special CBFs. EBFs provide a unique combination of stiffness, strength and ductility, making them a viable lateral load resisting system for steel structures subject to earthquake loads.

On the basis of present study the following conclusions can be drawn:

- Out of all the bracing systems, inverted Backward bracing system is giving maximum shear force.
- Forward bracing system is producing maximum bending moment in comparison to the other bracings.
- Most suitable bracing system is backward bracing system.
- Lateral displacement at the top floor is reduced approximately 50% for Backward braced in frame structure compared to without bracing system.

6.2 SCOPE OF FUTURE STUDY

Braces are likely to develop significant bending moments and shear forces in actual applications and the effect of this on behaviour of braced frame is unclear. An analytical study can be made to stimulate this behaviour.

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