ANALYSIS OF CONCENTRIC AND ECCENTRIC BRACED STEEL STRUCTURES

A PROJECT REPORT
SUBMITTED IN PARTIAL FULFILLMENT OF THE REQUIREMENTS
FOR THE AWARD OF THE DEGREE
OF
MASTER OF TECHNOLOGY
IN
STRUCTURAL ENGINEERING

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CANDIDATE'S DECLARATION

I hereby declare that the project report entitled "**ANALYSIS OF CONCENTRIC AND ECCENTRIC BRACED STEEL STRUCTURES"** submitted by me to Delhi Technological University (formerly Delhi College of Engineering) for partial fulfillment of the requirement for the award of the degree of M.TECH in **STRUCTURAL ENGINEERING** is a record of bonafide project work carried out by me under the guidance of **Mr. HRISHIKESH DUBEY**. I further declare that the work reported in this project has not been submitted either in part or in full, for the award of any other degree or certificate in any other institute or university.

Place: Delhi
Date: 30 June, 2019

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CERTIFICATE

I hereby certify that the Project Dissertation titled "**ANALYSIS OF CONCENTRIC AND ECCENTRIC BRACED STEELSTRUCTURES"** which is submitted by **PRAFUL RAZDAN (2K17/STE/012)** Civil Engineering Department**,** Delhi Technological University, Delhi in partial fulfilment 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
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ACKNOWLEDGEMENT

I would like to express gratitude to my mentor, **Mr. HRISHIKESH DUBEY** who gave me the golden opportunity to carry out this wonderful project on the topic **“ANALYSIS OF CONCENTRIC AND ECCENTRIC BRACED STEELSTRUCTURES”** which helped me in boosting my technical knowledge and experimental skills. His directions and support were the basic essence of motivation for me. I feel of paucity of words to express my sincere thanks to the honorable Head of Department of Civil Engineering, **Dr. NIRENDRA DEV**, for allowing me to utilize the department facilities and have been a constant source of motivation during the course of my project. I express my deepest sense of gratitude towards the Professors of Civil Engineering Department who helped me in formulating the problem statement and clarifying my doubts regarding the project. At last, I would like to thank my colleagues who helped me by actively participating in discussions and giving their valuable feedback. Their presence and support were invaluable Finally, I would like to thank my parents for their undying support, motivation and providing me the golden opportunity to study in this prestigious institution.

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ABSTRACT

Many existing steel strcuture ought to be retrofitted to vanquish the needs to restricts the vertical loading. In the present investigation a 9 stories steel building has been analysed and from that point broke down in view of sidelong tremor because of seismic forces, live weight subjected on it and dead weight. The presentation of the similar steel building has been investigated for various sorts of bracing frame work for instances eccentric (V)bracesand concentric (X) braces. The introduction of the structure has been overviewed subjected to horizontal storey displacement, the storey drifts, bending momentsand axial forces in different beamsand columns in the structure at various levels. The feasibility of various kinds of the bracingsystems supporting the structure has moreover been investigated. All themore significantly, the decrease in the horizontal displacement has been discovered for various kinds of the bracing frameworks in contrast with structure with no bracing. From the past examination, it has been noticed that the concentric (X) propping decrease progressively sidelonglateral displcement and in this wayfundamentally adds to more prominent stiffness to the structure.

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CHAPTER 1

INTRODUCTION

1.1 Bracing Introduction

The choice for a good fundamental system for a tall structure is a champion among the most problematic endeavors for the essential draftsman.Except if uncommon consideration is taken, a noteworthy bit of the basic material goes in drift control. For a customary 20-story steel structure, the expense of the bracing framework is about 30% of the complete expense. A perfect framework is the one for which the design could be administered by gravity load as it were.Braced steel outlines are known for their productivity in giving horizontal stiffness. The most effective bracing framework is the one which fills the double need of conveying gravity load notwithstanding controlling the lateral deflections because of horizontal loads.

John Hancock Building in Chicago of 100 storeys having X-bracing, 27-storey ALCOA Building with Diamond bracing in San Francisco,32-storey Town Center Tower in Southfield with Diamond bracing, Michigan, and K-supporting of 35-story Mercantile Bank Tower in St. Louis, just to give some examples, were designed by utilizing this philosophy. These structures brought about impressive investment funds in steel weight.

For example, an ordinary weight of structural steel for the 100-story Hancock Building is around 31 psf for each floor which would be required for a 30 to 60-story working with a customary beam pillar framework system. There have been not many occasions of fracture in supporting individuals in one to three story structures amid some serious seismic tremors. Among them, concentric K bracing of a three-story working of the University of Concepcion, Chile, which cracked amid the May 1960 quake, and In 1971 concentric X bracing of single story control plant working of Olive View Hospital Complex which broke amid San Fernando seismic tremor have been noted. Distributed data shows that these structures were not all around designed. For instance, the last structure was not designed as per the then present Los Angeles County Code, and besides, X supporting was given just in the North Wall which may have prompted torsional mode of vibration causing stressing of the bracing.

For a lot of given plan constrains, it is conceivable to design a structure with a few diverse bracing arrangements and stiffness combinations. The seismic reaction of open moment opposing steel framework which were proportioned by three philosophies: (i) Regular permissible pressure, (ii) weak girder-strong column, and (iii) Least weight. They noticed that a structured designed by permissible stress strategy resulted about strong girder-weak column segment extents which caused impressive inelastic distortion in the sections. They endorsed weak girder-strong column structure reasoning for unbraced moment safe frames.

The circumstance ends up being logically staggering in the design of the bracing framework. The strength or slenderness proportion of bracing individuals with respect to that of beams and columns could essentially influence the seismic conduct of a braced frame. Further, the seismic reaction of eccentrically propped casings could be altogether unique in relation to that of concentrically braced frames Because of complex conduct of supporting individuals and their associations, designers for the most part will in general be to some degree progressively moderate in planning these components. The subsequent extra weight of steel is viewed as very irrelevant with respect to the general load of the structure. Be that as it may, the excessively designed bracing member's may not generally deliver the ideal reaction of the struc-ture in case of strong shaking. The results of utilizing extremely strong bracing individuals are additionally not completely comprehended.

Steel braced outline is one of the auxiliary frameworks used to oppose parallel loads in multistoried structures. Steel propping is prudent, simple to raise, consumes less space. Braced frames are frequently used to oppose horizontal loads yet braces can meddle with building highlights. The steel props are ordinarily set in vertically balanced extents. This framework permits getting an extraordinary increment of stiffness with a negligible included weight, exceptionally successful for structures having an issue with Poor horizontal stiffness.

Bracings helps in inducing stiffness and steadiness to a structure under parallel stacking and furthermore to lessen displacement horizontally.The concentric bracings increment the horizontal stiffness of the casing and typically decay the drift horizontally. An expansion in the stiffness leads to an increase in inertia force because of seismic tremor. Bracings reduces shears and moments in columns segments, increment the axial pressure in the segments. Bracings with eccentric connection decreases parallel stiffness of framework and improve Dissipation of energy. EBFs have been utilized as this has settled notoriety as high-ductility frameworks and can possibly offer savvy arrangements in a moderate seismic areaThe main structure stacking parameter of significance is the most extreme burden liable to be knowledgeable about its lifetime. Paper examines the helper direct of steel working for both supported (counting Eccentric and concentric sorts) and unbraced conditions under static and horizontal stacking. The outcomes of static examination have been displayed and discussed in this paper. Finally, a comparable report has been shown to overview the best assistant presentation of steel working under parallel stacking. The essential purpose of the examination work has been to perceive the sort of propping which causes the least story dislodging such adds to progressively noticeable horizontal firmness.

1.2 Types of Bracings

Bracing systems are categorised in the following ways –

1. Concentric Braced System.
2. Eccentric Braced System.

Concentric bracing is oriented in such a way that all members (beams, columns, and bracing) meet at a common point . They provide the lateral resistance mainly through the axial force in the braces.The two major categories of concentric bracing are diagonal bracing and K-bracing. In addition, there is another type of bracing which is called cross bracing (X-bracing) . As shown in Figure, this is a construction site, where two concentric bracings can be identified, as cross bracing and diagonal bracing . It can be seen that all these bracings meet at a common point .The vertical cross bracing provides the lateral resistance to lateral load from both *X* and *Y* directions, mainly through the axial force in the structural members. Therefore, the diagonal member of this type of bracing is easy to design . It is also easy to assemble in the construction site. One of the disadvantages of concentric bracing is that the behavior of such bracing under cyclical loading is unreliable. In addition, efficient energy dissipation is difficult to achieve in concentrically braced frames. Therefore, they are rarely used in the seismic zones .

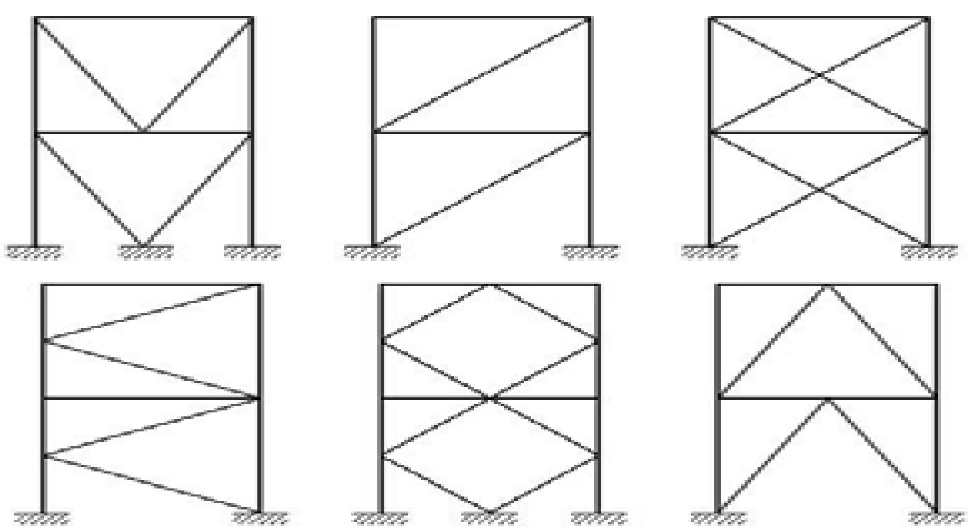

Fig. 1Concentric Frames

In Eccentrically braced frames (EBFs) the braces are offset from the columns or they do not intersect at the floor beams . Therefore, it results in an eccentrically connected bracing. Eccentric bracing can offer the same advantages as concentric bracing, while also providing significant ductility capacity and greater flexibility with architectural openings . Eccentric bracing is designed in a way that they do not buckle under extreme loading conditions. The axial forces induced in the braces are transmitted either to a column or to another brace largely through shear and bending in a segment of the beam called a link . The length of the link is notified by the letter *e* in Figure. In designing this type of bracing, the designer needs to ensure that under severe loading conditions the major inelastic activity takes place in the link . Therefore, the links can work as fuses to prevent buckling of the braces .

Eccentric bracing exhibits more ductile characteristic and greater energy dissipation capabilities than a concentrically braced frame of the same material. Therefore, this type of bracing is heavily used in earthquake zones due to the high ductility they provided through the link elements. However, concentrically braced frames can be used in moderate seismic regions .

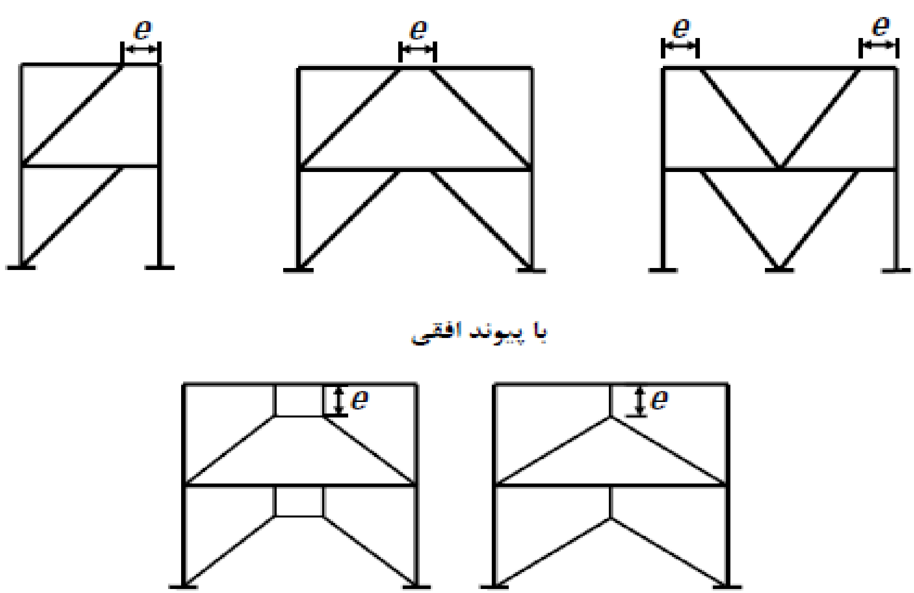

Fig. 2 Eccentric frames

The fundamental job of absorption and devaluation of inductive energy coming about because of a seismic tremor plays by the connecting beam. Then again, link beam acts like breakers and show flexibility and ductility.

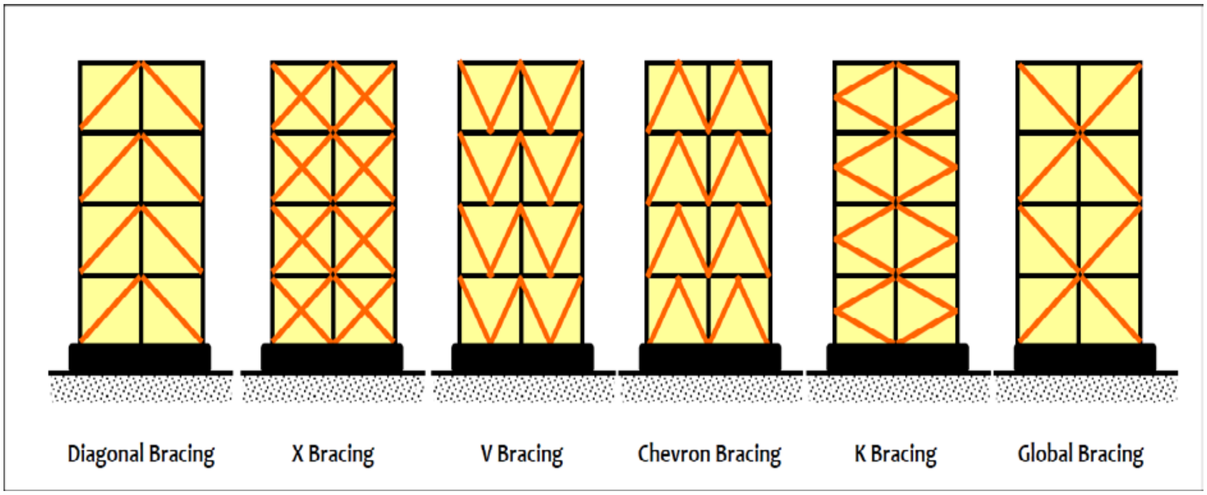

Fig 3Different Bracing Systems

1.3 Seismic Design Aspects

Structural weight is an important parameter as it controlscseismiccdesign along with stiffness as it initiates inertia force due to force generated by earthquakeand is also proportionate to mass of building. One main aspect of structural designingto be flexible or elastic when a structure experience seismic force due to earthquake without harm may make the task not possible financially. Thus, it may be basic for building to experience hurt and disseminatecenergy contribution to itcamid the seismic tremor.Along these lines, customary seismic tremor safe structure philosophy necessitates that typical structures ought to have the option to stand up to:

a) Minorshaking: with no harm to any components
b) Moderate shaking: auxiliary componentsminorly harmed, but causes little damage to noncbasic components of the structure
c) Extreme(nasty) shaking: cause damage to the basic components, but building should collapse .

Hence design structure for a part say 10% to 12% of the seismic force on the off chance that structure remains elastic when the earth starts to shake with peak ground motion and in this manner allowing harm or damage. However, sufficient starting stiffness must be ensured to keep up a key separation from fundamental harm under minor shaking. In this manner, the design against tremor impact is called seismic safe structure and not quake proof design.

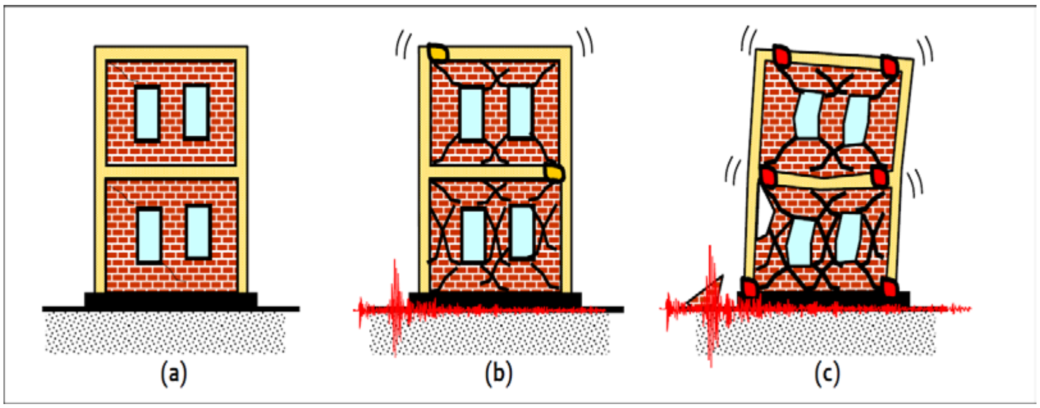

Fig.4 Shaking modes under Seismic Effect

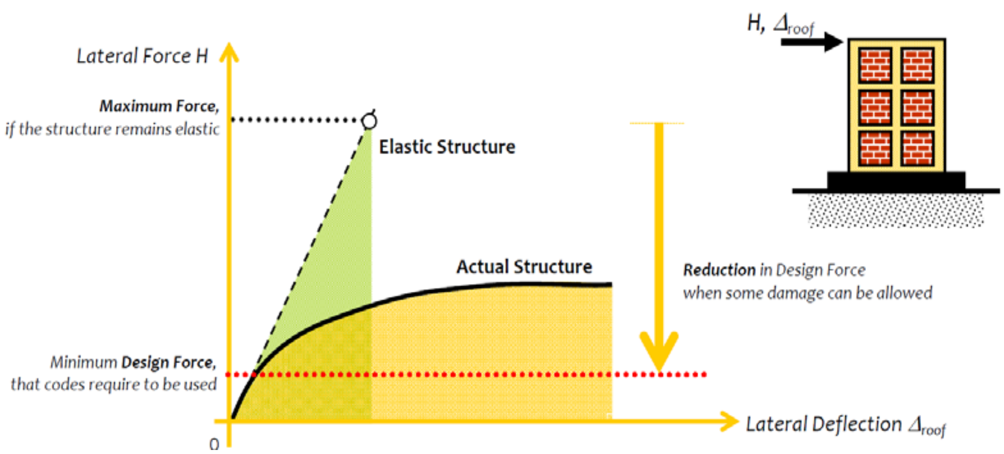

Fig. 5 Earthquake Design curve

The design for just a small amount of the flexible dimension of seismic impluse is conceivable,bychance it meets the demande of large relocation of building with basic harm without breakdown and solidarity loss. This property is ductility one of property. It is moderately easy to configuration with appropriate sizing and choosing good materials that can help the structure to maintain lateral strength and starting initial stiffness. In rundown, the forces induced by the earth shaking below structure can be called as displacement-type and force type is for force caused by wind.

Quake shaking needs structures, equipped for opposing certain relative displacement because of the forced displacement at its base, whereas wind needs the structures to opposeforces.Conceivable to appraise along exactness with a nasty force which can possibly put on a structure and it’s not definitely know greatest displacement forced under the structure. For a similar greatest displacement survived by structure, wind configuration requires just flexible conduct in the whole range of displacement, yet in seismic tremor configuration, two alternatives, to be specific, plan the structure to stay elastic or to experience inelastic conduct.

1.4 The Four Virtues of Earthquake Resistant Buildings

All structures are projecting outwards as a vertical cantilevers from the surface. Henceforth, during nasty earth shaking these cantilever projecting out of the surface from the groung experience whiplash effect.Consequently, to prevent them from harm or damage, we need to consider some unique consideration to prevent from sudden jerky. Structures become costly, whenever designed not to have any harm during solid seismic earth shaking.

Furthermore, they ought to be sufficiently able to not continue any harm amid feeble seismic tremor shaking and also ought to prevent any sideways large sway, notwithstanding amid powerless quakes. Also, fourthly, they ought not to collapse amid the normal solid quake shaking to be supported by them even with critical basic harm. These contending requests are suited in structures expected to be quake-safe.By joining four alluring attributes in them. These attributes called the four temperances of seismic tremor safe structures are:

1. Great seismic design, without any decisions of the compositional type of the structure that is negative to great seismic tremor execution and that, does not

present more up to date complexities in the structure conduct than what the seismic tremor is as of now imposing.

2. In every plan direction a required basic lateral stiffness should be provided to resists nasty ground shaking and do not collapse, and do not even think about keeping the expense of development within proper limits, alongside to resists its own weight i.e. the dead weight it should have required vertical strength atleastminiumand in this way avoid breakdown under solid quake shaking .

3. As a rule malleability in it to suit the forced parallel twisting between the base and the highest point of the structure, close by the perfect arrangement of lead to an authoritative position .

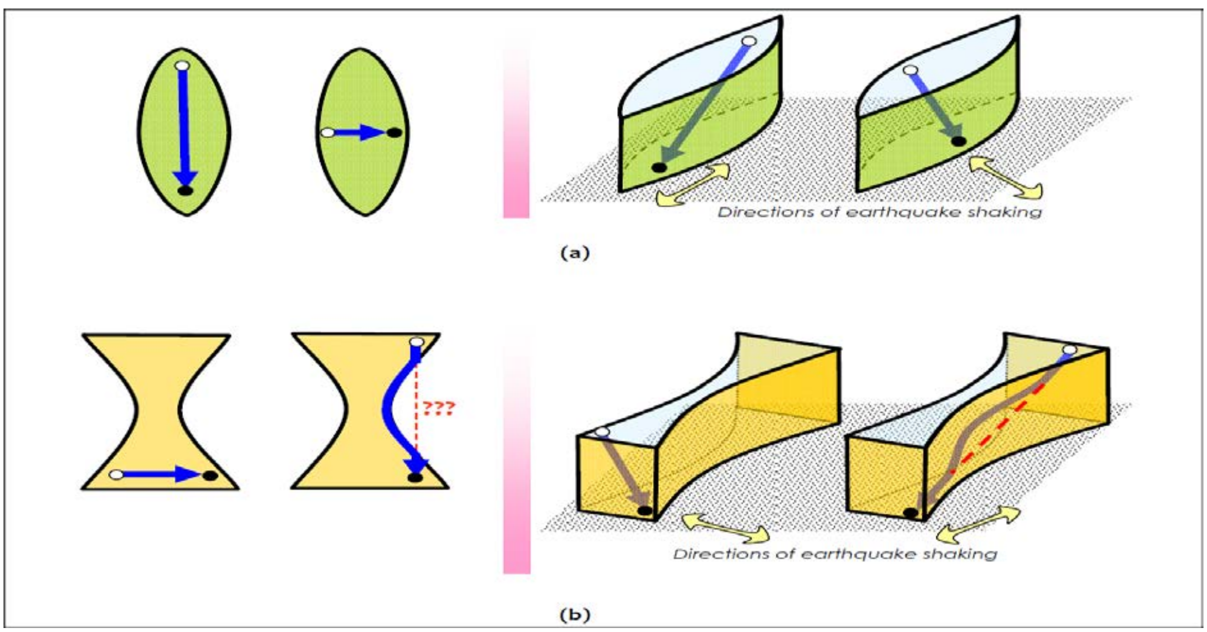

Fig.6 Geometrical configuration (a) convex, (b) concave

The conduct of structures amid tremors depend basically on these four ideals. Regardless of whether anybody of these isn't guaranteed, the presentation of the structure is relied upon to be poor. For a structure to perform acceptably amid quakes, it must meet the reasoning of earthquake resistant design. There are four parts of structures that architects and configuration engineers work with to make the seismic-resistant design of a structure, in particular, seismic basic design i.e. configuration, horizontal stiffness, ductility and horizontal strength, notwithstanding different angles like structure, style, usefulness and solace of the structure. Horizontal stiffness, ductility and horizontal strength of structures can be guaranteed by carefully following most seismic plan codes. The great seismic auxiliary arrangement can be guaranteed by following reasonable engineering highlights that outcome in great structural behaviour.

1.5 Seismic Structural Configuration

Seismic structural setup involves three fundamental perspectives, to be specific

a) Geometry, shape and size.

b) Size and Locationof structural components.

c) Size and Location non-structural components.

Impact of the geometry of a structure on its seismic tremor execution is best comprehended from the fundamental geometries of convex and concave shapes. The line joining any two inside territory of the raised focal point lies totally inside the focal point. Be that as it may, the equivalent isn't valid for the inward focal point a piece of the line may lie outside the territory of the sunken focal point. Structures with convex geometries are liked to those with concave geometries like the previous demonstrates earthquake performance execution. The structure with convex shape has direct load paths ways for exchanging seismic tremor shaking incited inertia forces to their bases for any bearing of ground shaking, while concave structures require bowing of load paths for shaking of the ground alongside specific headings that result in pressure concentration at all focuses where the load paths twist.

In view of the above exchange, typically constructed structures can be set in two classes, to be specific basic and complex. Structures with rectangularplans i.e. straight rise stand most obvious opportunity with regards to doing great amid a tremor since inertia forces are exchanged without twisting because of the structure geometry. Regardless, structures with troubles and focal openings offer geometric necessity to the movement entry of inertia powers; these powers ways need to twist before accomplishing the ground.

1.6 Structural Stiffness, Strength and Ductility

Sidelong firmness suggests the basic starting solidness ofstructure, in spite of the way that firmness of structurediminishes with growing mischief. Parallel quality insinuates the best opposition that structure offers in the midst of its entire history of security from relative disfigurement. Pliability along sidelong distortion insinuates the extent of the most outrageous misshapening and the glorified yield twisting. The greatest twisting analyzes to the most extreme disfigurement supported, if the heap distortion bend does not drop, and to 85% of an authoritative burden on the dropping side of the heap misshapening reaction bend after the apex quality or the sidelong quality is come to if the heap deformity bend drops ensuing to accomplishing peak quality .

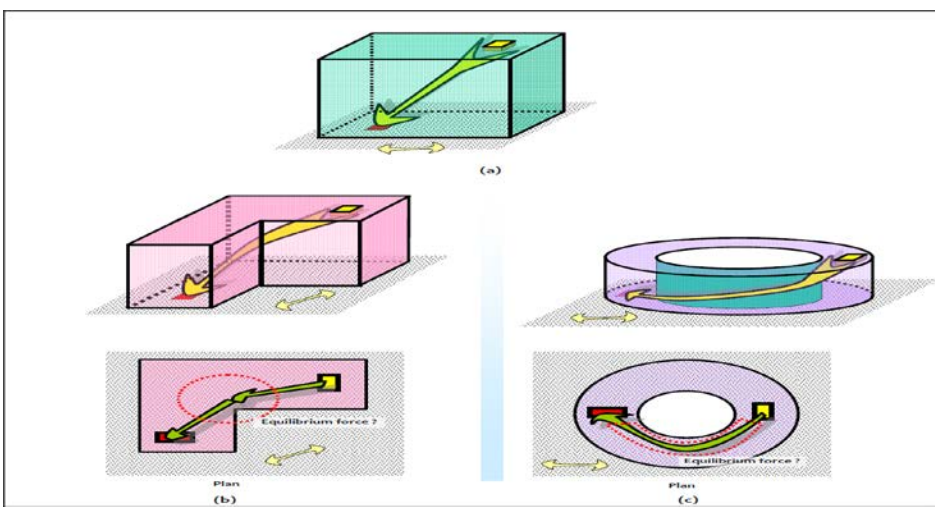

Fig.7 Building Types (a) simple, (b) and (c) complex

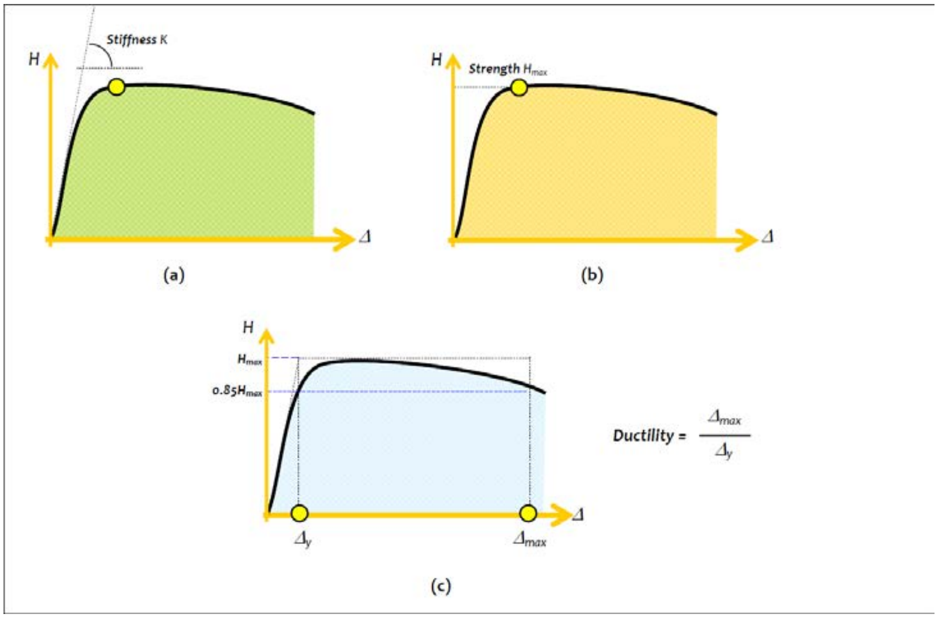

Fig.8 Load deformation curves

1.7 Purpose And Scope

Motivation behind this examination is to understand impact of various part extends on the seismic reaction of the braced frame. A 9-story structure is utilized in this investigation so as to confine the expense of calculations. Two kinds of supporting examples were selected eccentric and concentric. The reaction of these bracing systems was considered under the May 1940 El Centro, February 1941 Northern Californiaand September 1994 South Lake Tahoeground movement. 9-story, single bay, eccentrically braced frames were investigated under the above underground movements.These frames had a weak girder-strong brace, weak girder-intermediate brace and strong girder-weak brace individuals.This explanatory investigation is displayed ends are additionally determined with respect to choosing a fitting hysteresis model for bracing individuals in a given circumstance. 9-story, concentrically supported casings were additionally investigated ground movements. These frames had weak girder-intermediate brace and strong girder weak brace members.

CHAPTER 2

LITERATURE REVIEW

1. Tafheem and Khusru

- Lateral story displacements of the structure are enormously decreased by the utilization of concentric (X) supporting in contrast with eccentric (V) propping framework .
- The sidelong stiffness, the concentric (X) supporting has been discovered the most appropriate one for the steel building contemplated under the present investigation .
- The inter-storey drift is significantly decreased within the sight of the bracing framework.

2. Jagadeeshet al.

- The results of the performed inelastic examinations demonstrate that concentric Bracing systems are ideal to restrict tremor caused due to seismic forces.
- It outlines that store drift in the concentric bracing decrease with respect to the without supported edge.
- The displacement of verticalirregular structure is reduced 54% by Use of concentric propping framework in contrast with without supporting framework.
- Subsequently, propping framework has more effect on the Restriction on floor to floor displacements. The most extreme base shear for the concentric supporting casing is diminished by 24.58% when contrasted with without bracing frame.

3. Ziaulla el al.

- The results inferred that story drift of the model with concentric (X)propping was found to give results better for linear static contrasted with the eccentric bracing model .
- Also, a model with concentric (X) propping was found to give results better story drift for Pushover .
- Overall the model with concentric (X) propping supporting system was found to give results better for both straight and non-linear investigation .
- The concentric altered V supported model was found to give results better for story drift when contrasted with different models rendering it to be superior to the rest .

4. Chimeh et al.

- Comparison of the pushover bends of the reestablished structures stacking designs (loads patterns) demonstrated that the conduct of the Eccentrically Braced Frames (short association bar) is incredibly improved than various systems.
- Pushover and non-direct time history analysis showed that the breaking point of the Inverted V upheld 4 story edge increase conclusively by the Zipper column segments of the structure anyway this system isn't uncommonly profitable for the 8 story packaging.

5.Naqash et al.

- The paper tended to the planned method of Cross Concentric Braced Frames and Eccentric Braced Frames as indicated by Euro code 8 arrangements. Furthermore, concise tables are given for the two support frameworks where the examinations of the Euro code 8 with AISC seismic arrangements are exhibited, which pursue the limit configuration approach.
- From the tables, it is apparent that the plan arrangements of AISC are straight forward, for example on account of overstrength factor, an estimation of 2.0 is recommended by

AISC code rather a progressively sensible methodology is given on account of Euro code 8. The overstrength in Euro code 8 for CCBF is given as the proportion of the hub plastic obstruction of the support to the pivotal plan activity.

- Moreover, the slenderness confinements, just as the minimum over strength prerequisite should be satisfied. On account of EBF, the overstrength factor in EC8 is given by the proportion of the plastic shear protection from the connected structure shear activity when the connection is short or the proportion of the plastic flexural protection from the connected plan flexural activity when the connection is long.
- In general, it is reasoned that the seismic arrangements of EC8 appear to be convoluted contrasted with that of AISC with clear contrasts in the proposed estimations of the significant variables that are regularly embraced by the seismic codes. These require a more detail investigation of the two codes in future examinations by showing some contextual analyses consolidating the structure methods of the two current seismic codes. This will permit showing a reasonable image of the two codes.

6. Goel and Rai

- A four-story building was chosen as the principal topic structure for the investigation since it endured the most emotional setback due to brace failure amid the January 1994 Northridge seismic tremor and its regular concentrically braced frame structures planned by current codes and practice.
- Analyses were directed to survey the condition of-strength of the harmed structure to decide the need and degree of fixes and overhaul. The floor diaphragms were accepted inflexible, which enabled the six propped casings to be connected together at the floor levels. Every one of the columns outside the propped braced bays generally alluded to as "gravity segments", were lumped into one super" section and attached to the braced bays with the pin connected unbending links. Additionally, the gravity sections were thought to be nonstop along with the height of the structure. Therefore, the gravity sections and the supported casings experience equivalent horizontal displacements at each floor level.

- Upgrading to the uncommon concentric supporting framework by upgraded flexibility of braces and aversion of serious plastic pivoting in the bars associated with chevron propping are basic to averting total breakdown of concentric propped structures under extreme ground movements.
- The ponder working may have potential issues related with non-malleable braces and weak beam-strong brace design reasoning followed in the past plan practice. Such structures can be required to encounter broad basic harm in a moderate ground shaking and progressively genuine harm including total breakdown amid serious ground movements.

7. Xue Ming Han

- Elements impacting the structure design of eccentrically braced frames when the seismic loading does not assume an essential job in the part choice are examined. Twenty-two frames with the arrangement of eccentrically braced horizontal forces opposing frameworks are intended to examine the impact of structure tallness, link length and different dimensions of the proportion of wind to seismic tremor load.
- These structures were broke down under monotonic and dynamic sidelong loads. It is inferred that smaller seismic links overstrength elements might be utilized for prop and section configuration in moderate seismic districts than as of now determined in the structure standard. The lesser ductility requests experienced likewise propose that the enumerating prerequisites for short connections can be relaxed.

CHAPTER 3

METHODOLOGY

3.1 Seismic Design Force

A_h is described as $(ZS_aI)/(2gR)$, structure has $T \leq 0.1$ sec then estimation of A_h won't be equal to as much as Z/2 whatever estimation of I/R . This declaration tries to ensure an irrelevant plan power for strong building. This declaration is generous exactly when the essential mode time frame $T \leq 0.1$ sec in spite of the way that the code does not demonstrate so . In the case of higher modes, this repression should not to be constrained and this ought to be redressed in the code.

Indian StandardsBureau has issued a draft revision to change the above arrangement from (Z/2) to (Z/4). This appears to require when one thinks about a SMRF (Response Reduction Factor R = 5.0) with T under 0.1 seconds versus a SMRF with T more noteworthy than 0.1 seconds. Accepting significance factor of 1.0, and zone IV (Z=0.24g) working with T=0.11 second will be intended for A_h as 0.06g, while a structure with T= 0.09 second will be intended for (Z/2) as 0.12g.

The issue is more mind-boggling than simply evolving (Z/2) to (Z/4). For example, what occurs for structures with R-esteem not the same as 5.0, state an OMRF building (R=3.0) situated in seismic zone II (Z=0.10). On the off chance that significance factor is 1.0, a structure T=0.11 second will be intended for a coefficient of 0.042, while a structure with T=0.09 second will be intended for 0.05g or 0.025g relying upon whether Z/2 or Z/4 is utilized, separately. Subsequently, it appears to the creator that the substitution of (Z/2) by (Z/4) isn't the right methodology. The codes have customarily pursued an alternate methodology for exceptionally hardened structures: they just deny the utilization of a rising piece of the range bend between T=0 second to T=0.1 second for static examination, and for the primary method of the dynamic investigation.

Configuration Base Shear $V_B$is given by:

$$V_B = \mathrm{A} \times \mathrm{W}$$

Here,

A = lateralseismic coefficient
W = seismic weight

The design seismiccoefficient fora structure A_n is given by :

$$\mathrm{A_n} = \frac{\mathrm{ZIS_a}}{\mathrm{2Rg}}$$

Here,

I = Importance Factor
Z = zone factor as per IS:1893 2016, Table-2.
R = reaction decrease factor.

S_a/g = Average response quickening coefficient for soils and shakes locales according to IS 1893:2016 (section 1).
The extent of parallel force at the floors (nodes) rely on:

- Floor Mass of particular level.
- Distribution of firmness along tallness of the building
- Nodal dislodging in modes

IS 1893:2016 (section 1) utilizes an illustrative circulation of horizontal power alongside the stature of the structure. Dissemination of base shear along the tallness is finished by this condition:

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

Here:

Q_i = structure parallel force at the floor i

W_i= seismic load at floor i,

h_j = tallness of floor i quantified from the establishment

n = numbers of stories in the structure.

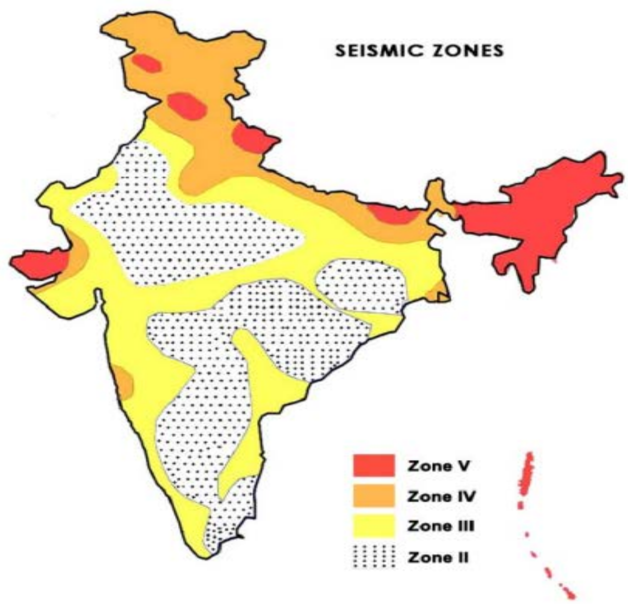

Fig. 9 Seismic Zone of India IS: 1893 (Part 1) – 2007

3.2 Response Spectrum of a Ground Motion

A structure can be scientifically imagined to be a gathering of identical basic structures each having just a single common time of oscillation, relating to one of the modes of oscillation of the structure. These are known as the degree of single freedom (SDoF) structures relating to every method of oscillation of the first structure. A SDOF structure has mass m, stiffness k and related basic damping. Therefore, all the SDOF structures with a similar extent of mass and firmness have a similar regular time of

$$T_n = 2\pi\sqrt{\frac{m}{k}}.$$

Such a lot of structures with an equivalent normal period (or frequency) of oscillation and same basic damping display same time history of reaction (i.e. increasing speed, speed and uprooting), when exposed to a similar seismic tremor ground shaking. In this way, it is helpful to distinguish heretofore the potential reactions of various such SDoF structures when it experience a particular seismic tremor due to ground shaking at various characteristic periods.

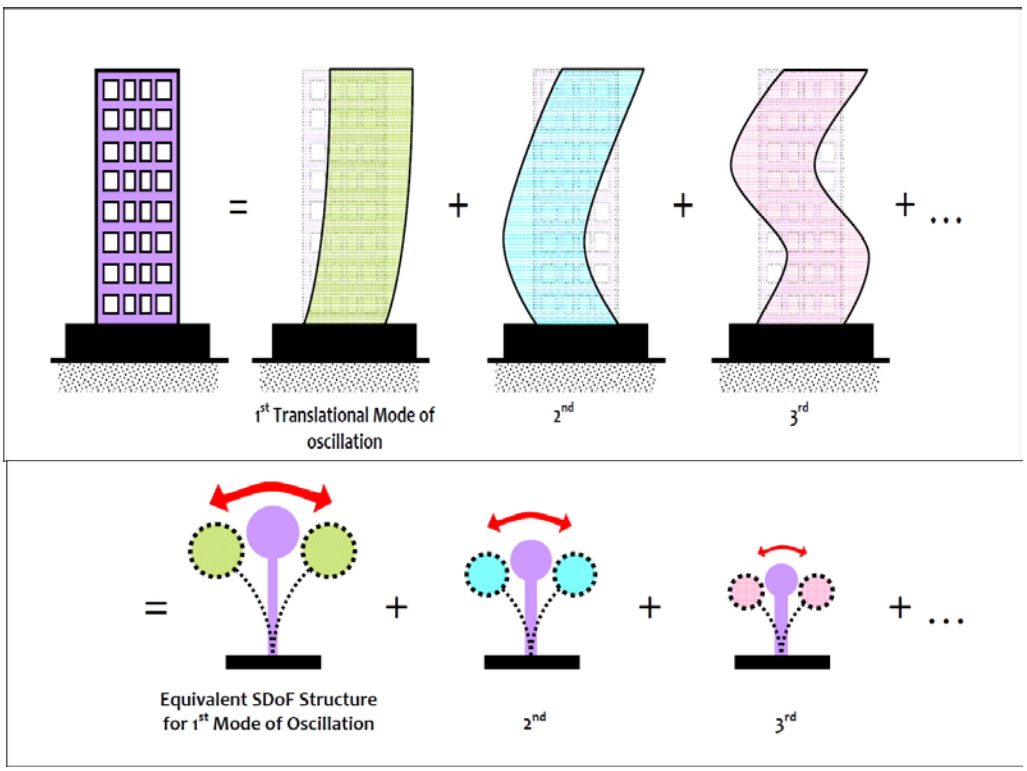

Fig. 10 Equivalent SDoF Structures of different oscillation of building

This is valuable in considering various structures in an area exposed to similar ground movements and comprehend their reaction. One can theoretically think about mounting structures of various unique qualities on a railroad wagon and shake the equivalent with uniform movementof ground. Ideally, the response of different structures be particular because they get assorted data energies from a comparable seismic tremor.

Comparaing wagon against their contrasting SDoF structures relating with its horizontaltranslational methods of wavering the results proves there are different responses to a comparative ground development.

3.3 Response Spectrum of Acceleration

Normal performing the design of structures for seismic forces we usally take the most nasty or lager induced by the shaking of the ground. This large induced force can be seen in two different ways: (i) stiffness k timesdisplacement x identified with elastic forces or(ii) mass mtimes acceleration a, identified with inertia forces, *i.e.*

$$\mathbf{F = ma \text{ or } F = kx}$$

Further, there is a final value of the above reactions which are very important since upon then the design depends, A particular range of SDOF, many natural period with the equivalent damping a diagram is plotted for the most nasty response under a similar tremor ground movement . The plotted diagram by this method is known as the Response Spectrum of the specific quake movement. One such response range identifying with the stimulating of the structure called the Response Spectrum of Acceleration for 5% damping under the action of 1940 Imperial Valley seismic tremor ground development .

3.4 Maximum Considered Earthquake (MCE) and Design Basis Earthquake (DBE)

Maximum Considered Earthquake MCE: defined as the most serious quake impacts considered by this standard, and Design Basis Earthquake DBE: as fit is the seismic tremor which canbe sensibly relied upon to happen at any rate once amid the design life of the structure . The IBC 2003 characterizes MCE as relating to 2 per cent likelihood of being surpassed in 50 years (multi-year return period), and the DBE as compared to 10 per cent likelihood of being surpassed in 50 years .

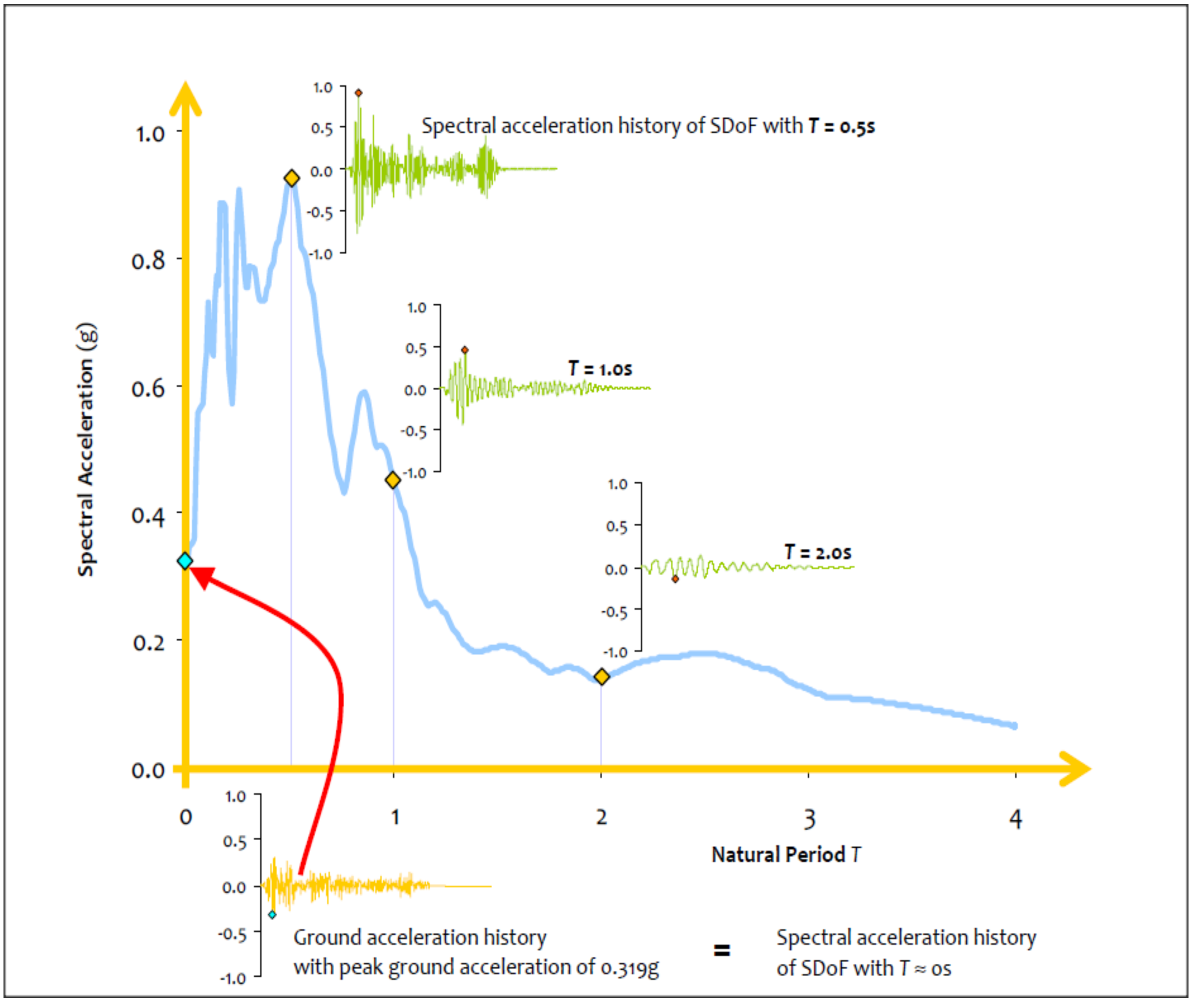

Fig. 11 Acceleration Response Spectrum

Since the seismic zone map in Indian code did not depend on probabilistic peril examination, it is absurd to expect to diminish the likelihood of event of a specific dimension of shaking in a given zone dependent on this code. Along these lines, the utilization of terms, for example, MCE and DBE does not include any new data, and can once in a while cause disarray and questions. For example, somebody may contend that the estimation of Z=0.36 for MCE in zone V of the code infers that the PGA esteem in zone V can't surpass 0.36g, which isn't the goal of the code. For example, amid the 2001 Bhuj quake, ground quickening ~0.6g has been recorded at Anjar situated at 44 km from the focal point.

3.5 Procedure For Dynamic Analysis

Step-1: Contingent upon the area of the structure site, recognize the seismic zone and allot Zone factor (Z) Use Table 3 alongside Seismic zones guide or Annex of IS - 1893 (2016)

Step-2: Figure out the seismic load of the structure (W) according to Cláuse 7.7.5.4, IS - 1893 (2016) – Seismic load of floors (Wi)

Step-3: Set up mass [M] and stiffness [K] lattices of the structure utilizing an arrangement of masses lumped at the floor levels with mass having SDOF.

Step-4: Utilizing [M] and [K] of the past advance and utilizing the standards of elements process the modular frequencies, {w} and comparing mode shapes, [φ]

Step-5: Register modular mass M_k of mode k utilizing the accompanying association with n being number of modes considered

$$M_k = \frac{[\sum_{i=1}^{n} W_i \emptyset_{ik}]^2}{g \times \sum_{i=1}^{n} W_i \emptyset_{ik}^2}$$

Step-6: Register modular support factors P_k of mode k utilizing the accompanying association with n being number of modes considered

Step-7: Figure plan horizontal power (Qik) at each floor in every mode (for example for the ith floor in mode k) utilizing the accompanying relationship

$$Q_{ik} = A_{hk} * \phi_{ik} * P_k * w_i$$

Where,

$A_{h(k)}$=Designspectrum valueof horizontal acceleration as inCláuse 6.4.2 of IS 1893(2016) using the natural preiod [$T_k = \frac{2\pi}{\omega_k}$] vibration of mode k.

Step -8: Compute storey shear forces in each mode (V_{ik}) acting in storey i in mode k as given by,

$$V_{ik} = \sum_{i+1}^{n} Q_{ik} \qquad \text{[Clause 7.7.5.4(e) of IS 1893 (2016)]}$$

Step -9:Figure story shear powers because of all modes considered, V_i in a story i, by joining shear powers because of every mode as per Clause 7.7.5.3 of IS1893 (2016). i.e., either CQC or SRSS modular blend techniques are utilized.

Step-10:At last process structure lateral forces at every story as,

$$F_{roof} = V_{roof} \text{ and} F_i = V_i - V_i + 1$$

3.6 Characteristics Of Buildings

3.6.1 Natural Period

Characteristic of natural Period Tn of a structure is the time taken by it to experience one complete cycle of oscillation. It is an inalienable property of a structure constrained by its mass m and firmness k. These three quantities are connected by its units are seconds (s). Along these lines, structures that are overwhelming (with bigger mass m) and adaptable (with littler stiffness k) have bigger common period than light and firm structures. Structures sway by interpreting alongside X, Y or Z bearings, or by turning about X, Y or Z axes. At the point when a structure sways, there is a related state of oscillation.

$$T_n = 2\pi\sqrt{\frac{m}{k}}$$

The reciprocal ($1/T_n$) of the natural period of structure, is the Natural Frequency f_n (Hz). The structure offers the least opposition when shaken at its regular recurrence (or common that is all). Thus, it experiences bigger swaying when shaken at its regular recurrence than at other frequencies. Generally, common periods (T_n) of 1 to 20 story typical Rcc and steel structures are in the scope of 0.05 - 2.00s.

Reverberation will happen in a structure, just if the recurrence at which ground shakes are enduring at or close to any of the normal frequencies of structure and connected over an all-encompassing timeframe. However, seismic tremor ground movement has takeoffs from these two conditions. In the first place, the ground movement contains a bin of frequencies that are consistently and haphazardly changing at each moment of time. There is no certification that the ground shaking contains a similar recurrence (and that excessively near fn of the structure) all through or notwithstanding for a continued length. Second, the little length for which the ground shaking happens at frequencies near fn of the structure is lacking to manufacture full conditions as a rule of the typical ground movements.

3.6.2 Fundamental Natural Period of Building

Each structure has various natural frequencies, at which it offers the least opposition to shaking prompted by outside impacts (like quakes and wind) and inward impacts (like engines/motors fixed on it). Each one of these natural frequencies and the related deformation state of a structure establishes a Natural Mode of Oscillation. The technique for influencing with the smallest normal recurrence (Furthermore, greatest standard period) is known as the Fundamental Mode; the related regular time span T1 is known as the Fundamental Natural time Period and furthermore related typical repeat f1 the Fundamental Normal Frequency. Further, standard structures held at their base from interpretation in the three directions, have directions have

1. Three central translational normal periods, Tx_1, Ty_1 and Tz_1, related with its translational swaying alongside X and Y bearings,
   and vertical translationaloscillation along with the Z heading, separately.

2. One fundamental rotational normal period $T\theta_1$ related with its rotation about an axis parallel to Z axis.

The amount of characteristic techniques for a structure is unendingness. In any case, for configuration purposes, the amount of modes is restricted. For instance, when the restricted part model (FEM) of the structure is prepared, the structures are discretised into people meeting at hubs. Each hub has 6 degrees of freedom. Henceforth, for a structure with numerous hubs, the most extreme degrees of opportunity can be tallied to be limited, state N. Here, the structure has N normal methods of swaying. In ordinary structures, N can be enormous. Yet, frequently, just a couple of modes are important for designing estimations to evaluate the reaction of structures.

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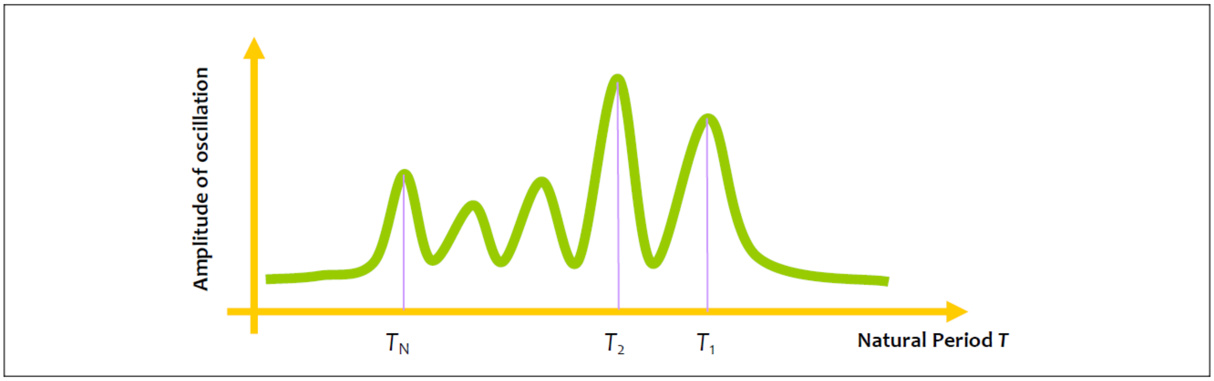


Fig. 12 Natural Periods

3.6.3 Mode Shapes

Disfigured shape of a building when it ossilates by seismiv effect related to its natural period is known as Mode shape. Henceforth, a building is required to have mode shape equal to the number of natural period. A structure has endless amounts of the natural time frame i.e. natural period. Be that as it may, in the scientific displaying of the structure, as a rule, the structure is discretised into various components. The palce where different section meets is called nodes. There are 3Cartesian directionsin which each node decipher and pivot about the 3 axes. Thus, in a seismic event this way amount of hubs of discretisation is N, then6N techniques will be available to the building to ossicales, and periods also modes which are associated with these are 6N will be in the state of influencing. Contorted shape which is formed when structure ossilatesat the largest real natural preiodis named its first mode shape.

Correspondingly, the following contored shapes due to ossiclation at different periods which is decreasing progressively are called second, third mode shapeand so on.

3.6.4 Fundamental Mode Shape of Oscillation

When the ossications of a building is taking place, it will possibly ossicaltes along along 3essential directions to be specific, unadulterated (X) translational, unadulterated (Y) translational and unadulterated pivot about (Z) direction. Standard structures have these unadulterated mode shapes. Different mode shape of Erratic structures are very unique or one of its kinds. Mode shape cannot be clubed with one another as they are autonomous. The entire lot of the mode shape generlise response of whole building. The commitments of various methods of wavering change; generally, commitments of certain modes command. It is essential to attempt to make structures ordinary to the degree conceivable. In any case, in standard structures as well, care ought to be taken to find and size the basic components with the end goal that torsional and blended methods of swaying don't take an interest much in the general oscillatory movement of the structure.

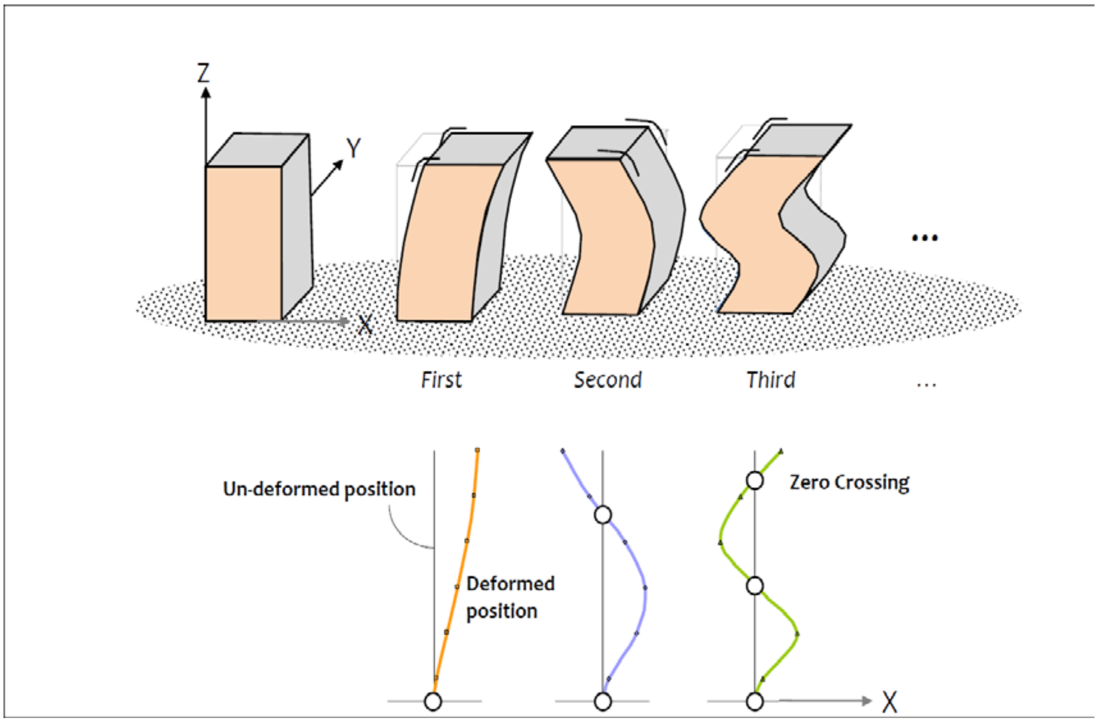


Fig. 13 Oscillation of different modes

3.6.5 Factors impacting Mode Shapes:

- Impact of Flexural Stiffness of Structural members like beams and columns.
- Impact of Axial Stiffness in Vertical Members.
- Impact of Degree of Fixityie hinged or fixed at Member Ends.
- Impact of tallness Building.
- Impact of Unreinforced Infill Walls in RC Frames.

3.6.6 Damping

Structures set to swaying by tremor shaking, at last, with time come back to rest. A result of the dispersal of the oscillatory imperativeness through change to various sorts of essentialness, like warmth and sound. Arrangement of this change is damping. In standard encompassing shaking of structure, various components impede its development, for example, pull due to air resistance around structure, contact of different units of structure andRccmicrocracking in auxiliary units. It is known as Basic damping.

When Solid tremor shaking happens, structures are harmed. Bars and bond of the RC structures enter a nonlinear extent of material lead. As inelastic exercise happens this rises the condition of hysteresis which lead to increasing motion which is harmful to structure.Damping is differing for different trademark techniques for faltering of a structure. In any case, Indian seismic codes recommend the usage of 5% damping in each and every trademark strategy for the influencing of braced strong structures and 2% for steel structures.

3.7 Time History Analysis

Linear or non-linear analysis of dynamic basic reaction under the stacking which may contrast as indicated by determined time work. The essential overseeing condition for the dynamic reaction of the multi level of opportunity framework is given by the above condition.

The given condition can be illuminated by numerical coordination technique, for example, Runge-Kutta strategy, Newmark combination strategy and Wilson – ϴ strategy. The ETABS Software figures the auxiliary reactions at each time step and in this manner tackles the administering time condition.

3.8 Load Combinations For Steel Structures

Design of steel structures, Inaccordence to IS code 1893 (2016) section 1 we will provide load combination.

- 1.2 [DL+ LL ± (EL_X± 0.3 EL_Y)] and 1.2 [DL+ LL ± (EL_Y ± 0.3 EL_X)]
- 1.5 [DL ± (EL_X ± 0.3 EL_Y)] and 1.5 [DL ± (EL_Y ± 0.3 EL_X)]
- 0.9 DL ± 1.5 (EL_X ± 0.3 EL_Y) and 0.9 DL ± 1.5 (EL_Y ± 0.3 EL_X)

CHAPTER 4

STRUCTURAL MODELLING

4.1 Frame Geometry

Model 1 refers to concentric braced building and Model 2 is Eccentric braced building on which seismic analysis is done using ETABS.

4.2 Model 1

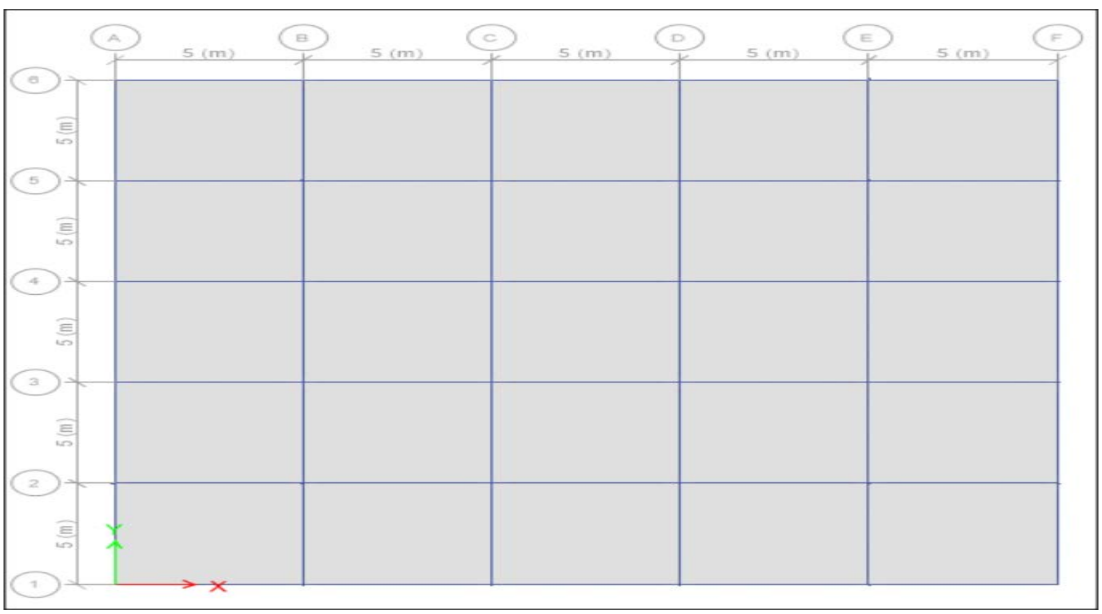


Fig. 13 Plan

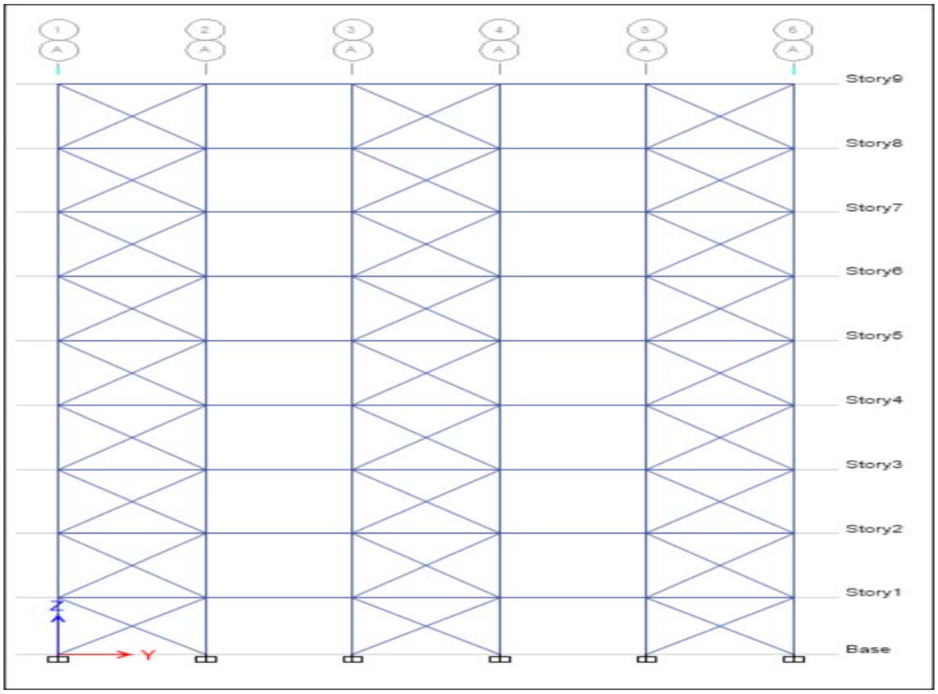


Fig. 14 Side elevation

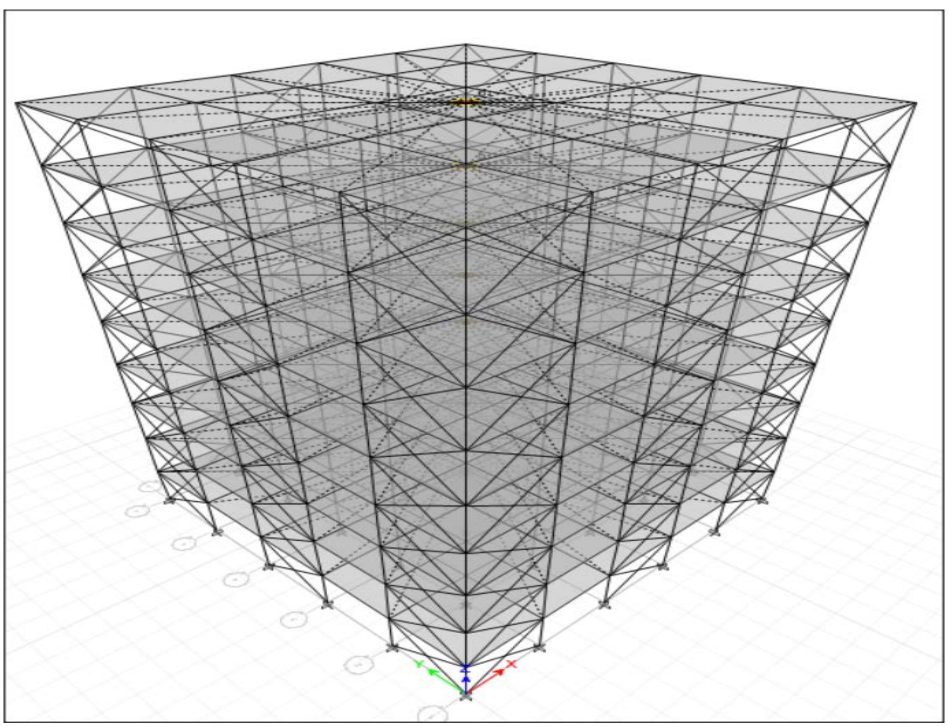


Fig. 15 3D view of concentric Building

The Bay width is 5 m and storey height for the ground floor is 3.5m and rest of the storey height above first floor is 4m.

The steel section selected for the analysis purpose are respectively given below:

Column – Steel tube 300 x 150 x 15

Beam – ISWB 500

Concentric Bracing section – ISHB 400

4.3 Model 2

The Bay width is 5 m and storey height for the ground floor is 3.5m and rest of the storey height above first floor is 4m. The steel section selected for the analysis purpose are respectively given below:

Column – Steel Tubes 300 x150 x 15

Beam – ISWB 400

Eccentric Bracing section – ISHB 300

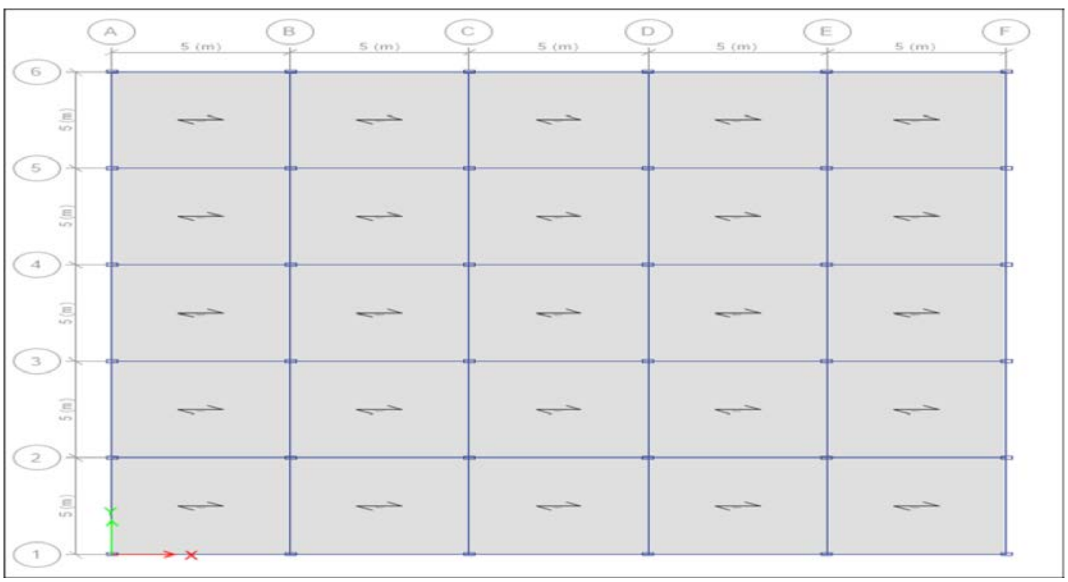


Fig. 16 Plan

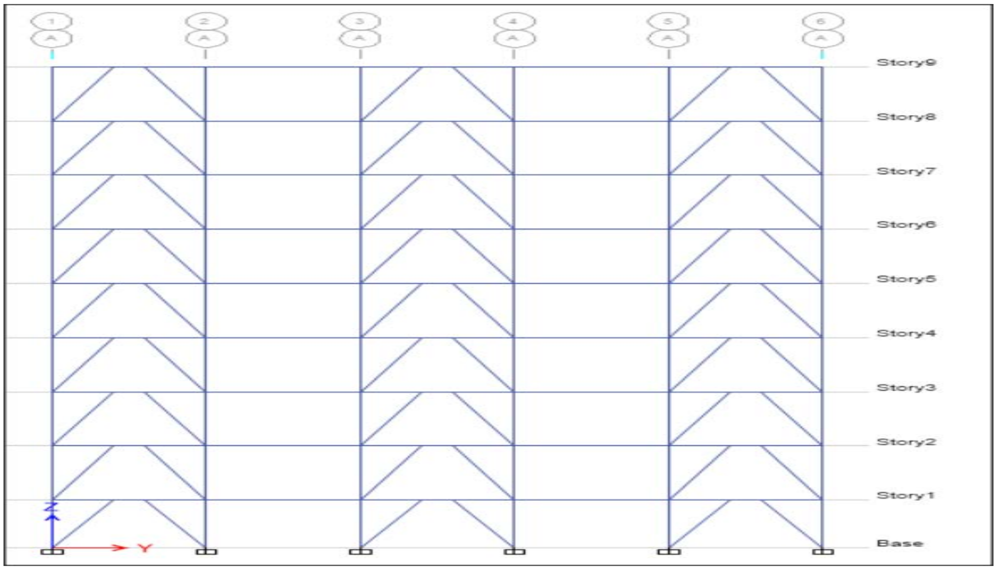


Fig. 17 Elevation

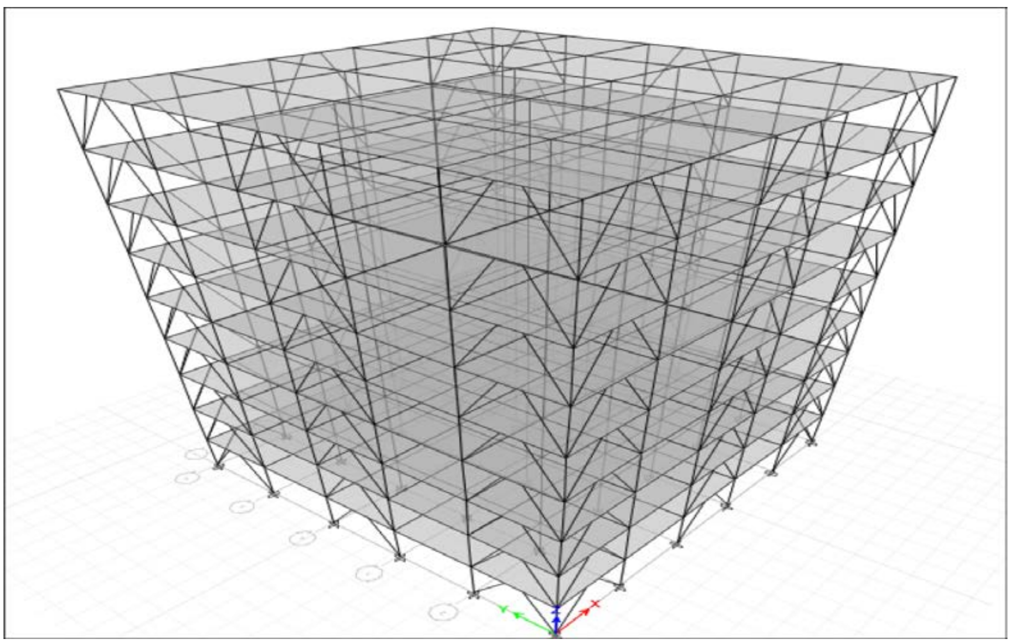


Fig. 18 3D view of Building with eccentric bracing

4.4 General structural information:

- Designing and analysis of G+9 building situated in delhi.
- Height 35.5m.
- Length (x direction) 25m.
- Length (y direction) 25m.
- Using - IS: 800 2007-Code Of Practice For General Construction In Steel.

- IS: 1893, IS 4326 - Code of practice for earthquake.
- Slab Thickness- 125mm
- Material used:FE-250, M -30, HYSD- 500(as Rebar)

4.5 Frame Design

The structure casing used in this examination is believed to be arranged in Indian seismic zone IV with medium soil conditions. Seismic weights are assessed by IS 1893:2016 and structure of steel parts are passed on as per IS 800 - 2007 standards.The gravity loads consist of dead load as self-weight of thestructure and a live load as floor load of 3kN/m^{2}at every floor . The design seismic coefficient of horizontal accerelation(A_h) is calculated as per IS 1893:2016

$$A_h = Z.I/2.R$$

Here, Zone factor Z = 0.24, Response reduction factor, R = 4.5 for concentric braced building and R=5 for eccentric braced building and Importance factor I = 1.2. The design base shear (V_B) is calculated as per IS 1893:2016.

$$V_B = A_h.S_a/g.W$$

Period for analysis = 0.085H$^{0.75}$, which is found to be 1.253 sec for eccentric braced structure and 1.079 for concentric braced structure.

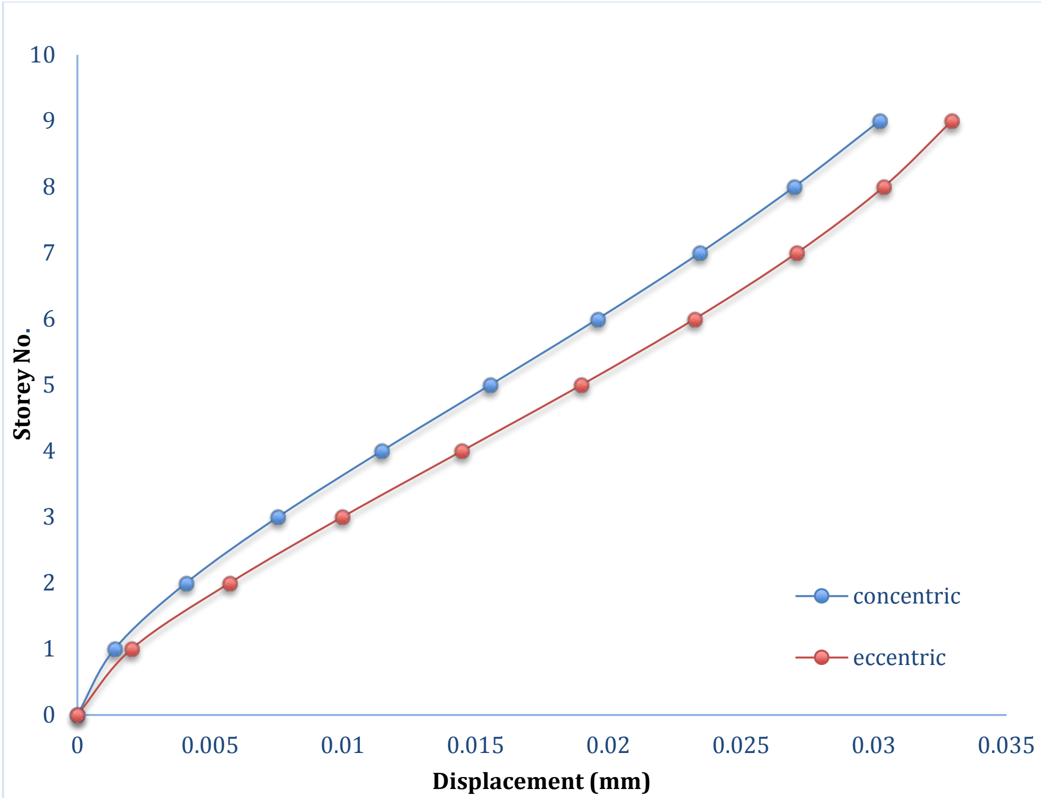
CHAPTER 5

RESULTS AND CONSLUSION

5.1 Comparing Maximum Storey Displacements Of Concentric And Eccentric Braced Structure:

Table 1Storey displacement

TABLE: Story Response				
Story	**Elevation**	**Location**	**X-Dir**	**X-Dir**
	m		Concentric	Eccentric
9	35.5	Top	0.030221	0.032944
8	31.5	Top	0.027001	0.030371
7	27.5	Top	0.023449	0.027099
6	23.5	Top	0.019599	0.023255
5	19.5	Top	0.015551	0.018988
4	15.5	Top	0.011465	0.014488
3	11.5	Top	0.007557	0.009984
2	7.5	Top	0.004095	0.005735
1	3.5	Top	0.001404	0.002043
Base	0	Top	0	0



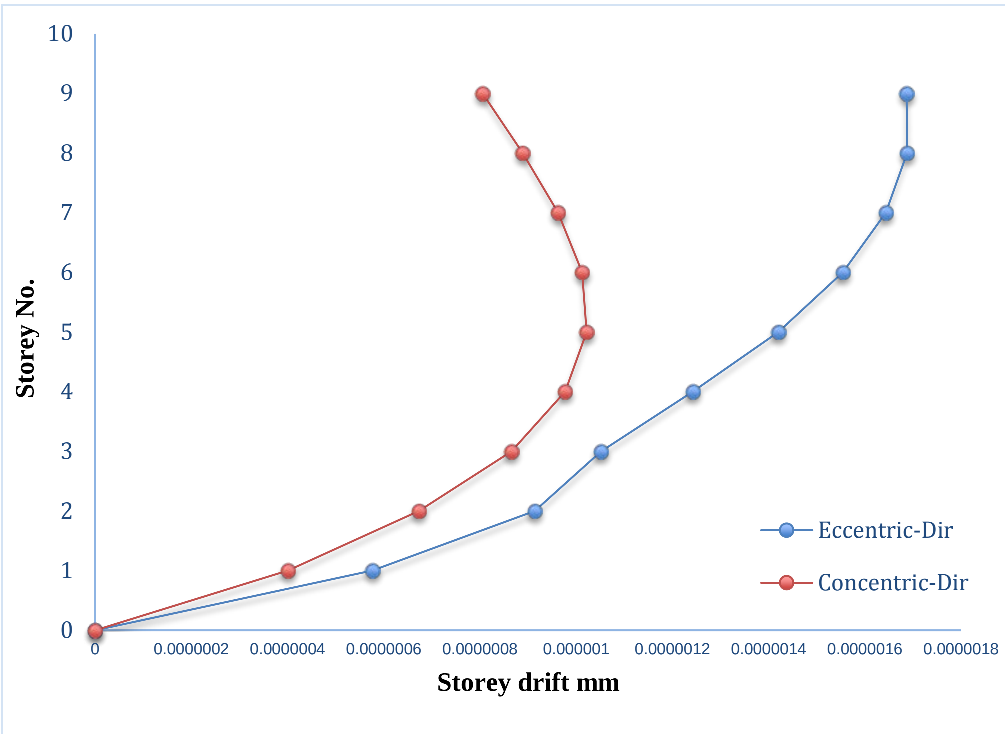
After observing the storey displacement results and knowing that lateral storey displacement in any direction is greatly reduced by the bracing system. It has also been noted that concentric (X) bracing reduces storey displacement considerably. Therefore it can be said that concentric bracing provides greater lateral stiffness to the steel structure than eccentric (V) bracing.

5.2 Comparing Maximum Storey Drift of Concentric And Eccentric Braced Structure:

Table 2 Storey Drift

TABLE: Story Response				
Story	**Elevation**	**Location**	**X-Dir (mm)**	**X-Dir (mm)**
	m		**Eccentric**	**Concentric**
9	9	Top	1.68677E-06	8.04965E-07
8	8	Top	1.68776E-06	8.8811E-07

7	7	Top	1.64347E-06	9.62442E-07
6	6	Top	1.55437E-06	1.01206E-06
5	5	Top	1.41993E-06	1.02151E-06
4	4	Top	1.24228E-06	9.76985E-07
3	3	Top	1.05142E-06	8.65387E-07
2	2	Top	9.13611E-07	6.72922E-07
1	1	Top	5.76718E-07	4.01086E-07
0	0	Top	0	0



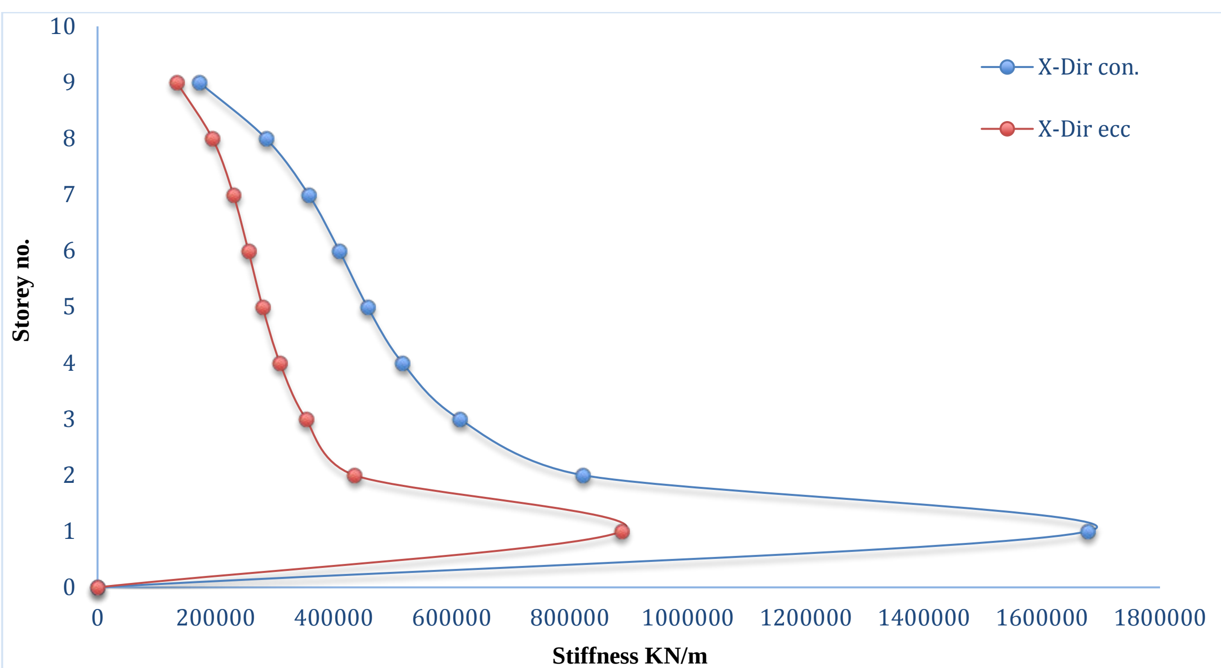
It very well may be seen that eccentrically braced frames give more drift in structure when contrast with concentrically supported edges due with lesser horizontal firmness as looked at concentrically propped edges. Inter storey drift decline as we go upward in light of the fact that the impact of the tremor likewise diminishes as we go upwards. We can say that Concentric X propped edges have most extreme sidelong stiffness as we would we be able to see they produce the least drift while eccentrically braced frames produce greater drift.

5.3 Comparing StoreyStiffness of Concentric And Eccentric Braced Structure:

Table 3Storey Stiffness

TABLE: Story Response			X-Dir	X-Dir
Story	**Elevation**	**Location**	**Eccentric**	**Concenteric**
	m		kN/m	kN/m
9	35.5	Top	134366.924	172435.4
8	31.5	Top	194702.679	285402.4
7	27.5	Top	230111.134	357651.9
6	23.5	Top	255545.741	409622.3
5	19.5	Top	279565.624	457347.9
4	15.5	Top	309017.346	516611.5
3	11.5	Top	353298.947	613430.6
2	7.5	Top	435082.824	822222.1
1	3.5	Top	888061.511	1677821
Base	0	Top	0	0

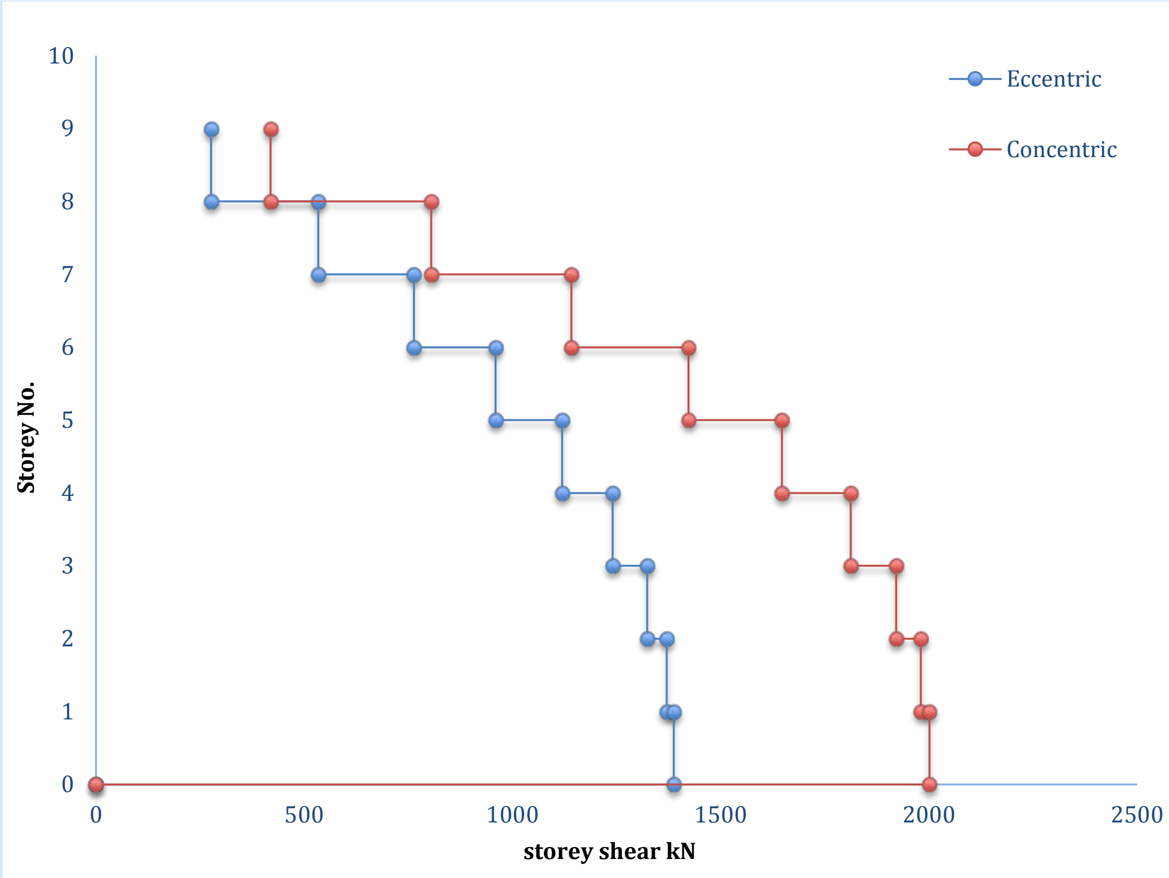
It very well may be seen in the figure below that eccentrically braced frames has less stiffness in structure when contrast with concentrically supported edges. Due to which eccentric braced system will have more displacement, drift and relative higher storey shear as well as base shear.



5.4 Comparing Maximum StoreyShear of Concentric and Eccentric Braced Structure:

Table 4 Story Shear

TABLE: Story Response			Eccentric	Concentric	
Story	**Elevation**	**Location**	**X-Dir**	**Location**	**X-Dir**
	m		kN		kN
9	35.5	Top	276.4	Top	418.8
		Bottom	276.5	Bottom	418.8
8	31.5	Top	534	Top	805.1
		Bottom	534	Bottom	805.2
7	27.5	Top	763.3	Top	1141.3
		Bottom	763.3	Bottom	1141.3
6	23.5	Top	959.4	Top	1422.9
		Bottom	959.4	Bottom	1422.9
5	19.5	Top	1119	Top	1646.9
		Bottom	1119	Bottom	1646.9
4	15.5	Top	1240.3	Top	1812.3
		Bottom	1240.3	Bottom	1812.3
3	11.5	Top	1323.4	Top	1921.5
		Bottom	1323.4	Bottom	1921.5
2	7.5	Top	1370.7	Top	1980.8
		Bottom	1370.7	Bottom	1980.8
1	3.5	Top	1387.3	Top	2001.3
		Bottom	1387.3	Bottom	2001.3
0	0	Top	0	Top	0
		Bottom	0	Bottom	0



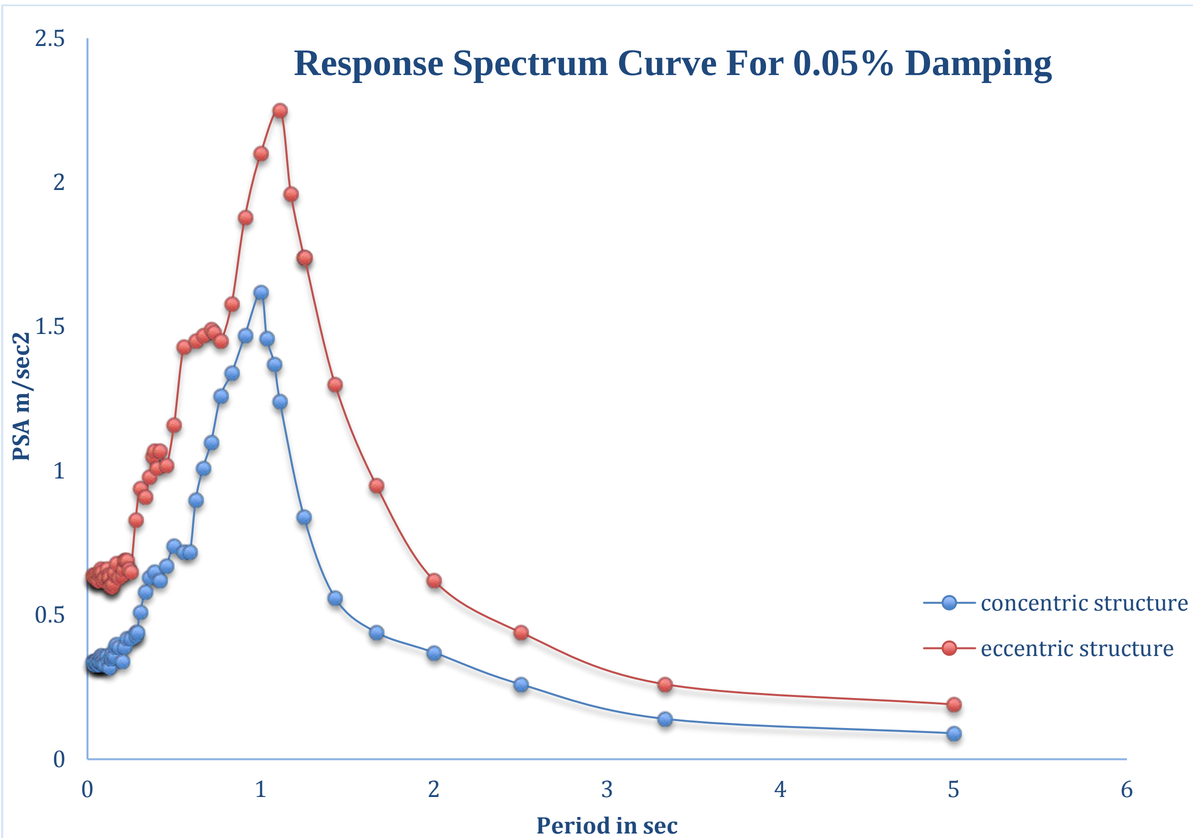
5.5 Comparing Response spectrum curve of Concentric and Eccentric Braced Structure:

Table 5 Response Sprctrum Curve

TABLE: Psuedo Spectral Acceleration, PSA			
Concentric structure	**Damping 0.05**	**Eccentric structure**	**Damping 0.05**
sec	m/sec²	sec	m/sec²
0.03	0.34	0.03	0.64
0.036	0.33	0.036	0.63
0.04	0.34	0.04	0.63
0.045	0.34	0.045	0.64
0.05	0.34	0.05	0.63
0.056	0.33	0.056	0.62
0.061	0.33	0.061	0.62
0.067	0.34	0.067	0.64
0.067	0.34	0.071	0.65

0.071	0.34	0.077	0.66
0.077	0.36	0.083	0.65
0.08	0.36	0.091	0.62
0.083	0.35	0.1	0.63
0.088	0.34	0.111	0.66
0.089	0.34	0.118	0.63
0.091	0.33	0.123	0.64
0.1	0.33	0.125	0.63
0.111	0.36	0.131	0.6
0.118	0.34	0.133	0.6
0.125	0.32	0.143	0.6
0.133	0.35	0.145	0.61
0.136	0.35	0.154	0.64
0.138	0.36	0.155	0.65
0.143	0.37	0.167	0.68
0.154	0.36	0.182	0.63
0.161	0.39	0.2	0.64
0.167	0.4	0.206	0.66
0.182	0.39	0.213	0.69
0.2	0.34	0.218	0.69
0.213	0.39	0.227	0.69
0.227	0.42	0.238	0.66
0.25	0.42	0.25	0.65
0.278	0.44	0.278	0.83
0.28	0.43	0.303	0.94
0.287	0.44	0.333	0.91
0.303	0.51	0.357	0.98
0.333	0.58	0.376	1.05
0.357	0.63	0.385	1.07
0.385	0.65	0.399	1.01
0.417	0.62	0.417	1.07
0.455	0.67	0.455	1.02
0.5	0.74	0.5	1.16
0.556	0.72	0.556	1.43
0.592	0.72	0.625	1.45
0.625	0.9	0.667	1.47
0.667	1.01	0.714	1.49
0.714	1.1	0.73	1.48
0.769	1.26	0.769	1.45
0.833	1.34	0.833	1.58
0.909	1.47	0.909	1.88
1	1.62	1	2.1
1.035	1.46	1.111	2.25

1.079	1.37	1.174	1.96
1.111	1.24	1.25	1.74
1.25	0.84	1.253	1.74
1.429	0.56	1.429	1.3
1.667	0.44	1.667	0.95
2	0.37	2	0.62
2.5	0.26	2.5	0.44
3.333	0.14	3.333	0.26
5	0.09	5	0.19



In Eccentric acceleration response is very higher in the range up to 2 to 2.5 (m/sec^2) and gives greater response for the structural systems having the natural vibrton period in range of 1 to 5 sec as compared to that of a structure with concentric system.

The current spectral shape gives the appropriate comparisons between Eccentric and concentric sytems. From the observation say that in the eccentric systems have greater need of concentration on the earthquake resistant design and their criteria compare with response developed for the concentric system.

5.6 Modal Mass Participation Ratio:

The modes utilized in the investigation in the specific bearing for tremor shaking ought to be with the end goal that the entirety of all out masses of the modular masses of the method of examination is in any event 90% of the whole seismic weight.

5.6.1 For Eccentric Braced Structure:

Table 6 Modal Mass Participation Ratio

Case	Mode	Period sec	UX	UY	SUM UX	SUM UY
Modal	1	1.253	0	0.7418	0	0.7418
Modal	2	1.174	0.7487	0	0.7487	0.7418
Modal	3	0.73	9.37E-07	0	0.7487	0.7418
Modal	4	0.399	0	0.1591	0.7487	0.9009
Modal	5	0.376	0.1513	0	0.8999	0.9009
Modal	6	0.238	5.14E-07	0	0.8999	0.9009
Modal	7	0.218	0	0.0473	0.8999	0.9482
Modal	8	0.206	0.046	0	0.9459	0.9482
Modal	9	0.155	0	0.022	0.9459	0.9702
Modal	10	0.145	0.0219	0	0.9679	0.9702
Modal	11	0.131	0	0	0.9679	0.9702
Modal	12	0.123	0	0.0128	0.9679	0.983

5.6.2 For Concentric Braced Struture:

Table 7 Modal Mass Participation Ratio

Case	Mode	Period sec	UX	UY	SUM UX	SUM UY
Modal	1	1.079	0	0.7165	0	0.7165
Modal	2	1.035	0.7219	0	0.7219	0.7165
Modal	3	0.592	0	0	0.7219	0.7165
Modal	4	0.287	0	0.1676	0.7219	0.8841
Modal	5	0.28	0.1643	0	0.8862	0.8841
Modal	6	0.161	0	0	0.8862	0.8841
Modal	7	0.138	0	0.0604	0.8862	0.9444

Modal	8	0.136	0.0595	0	0.9457	0.9444
Modal	9	0.089	0	0.0279	0.9457	0.9723
Modal	10	0.088	0.0275	0	0.9732	0.9723
Modal	11	0.08	0	0	0.9732	0.9723
Modal	12	0.067	0	0.0142	0.9732	0.9865

So for both concentric and eccentric braced structure modal masses of modes becomes 90% of the entire seismic weight at mode 8 respectively.

5.7 Modal Period and Frequencies

5.7.1 For Concentric Braced Struture:

Table 8 Modal Period and Frequencies

Case	Mode	Period sec	Frequency cyc/sec	Circular Frequency	Eigenvalue rad^2/sec^2
Modal	1	1.079	0.927	5.8234	33.9124
Modal	2	1.035	0.966	6.0707	36.8539
Modal	3	0.592	1.69	10.6157	112.6929
Modal	4	0.287	3.486	21.904	479.7841
Modal	5	0.28	3.569	22.4227	502.7787
Modal	6	0.161	6.2	38.9583	1517.7523
Modal	7	0.138	7.234	45.4544	2066.1043
Modal	8	0.136	7.333	46.0767	2123.0654
Modal	9	0.089	11.214	70.4594	4964.5282
Modal	10	0.088	11.329	71.1837	5067.1176
Modal	11	0.08	12.555	78.8874	6223.2273
Modal	12	0.067	15.002	94.2576	8884.4901

5.7.2 For Eccentric Braced Structure:

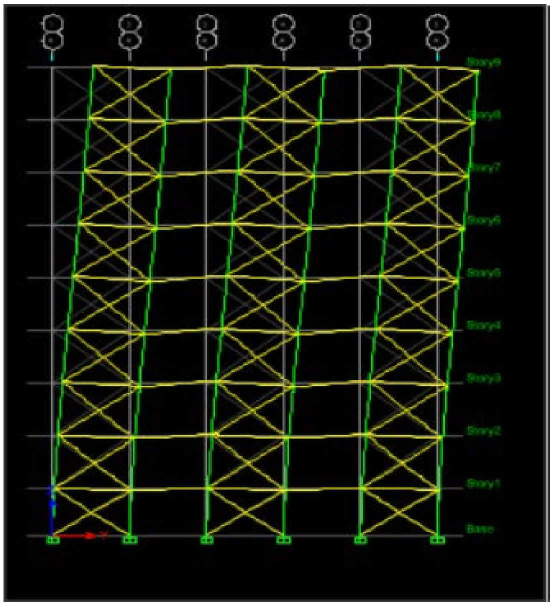
Table 9 Modal Period and Frequencies

Case	Mode	Period sec	Frequency cyc/sec	Circular Frequency	Eigenvalue rad^2/sec^2
Modal	1	1.253	0.798	5.0165	25.1654
Modal	2	1.174	0.852	5.3509	28.6326
Modal	3	0.73	1.37	8.6063	74.0691

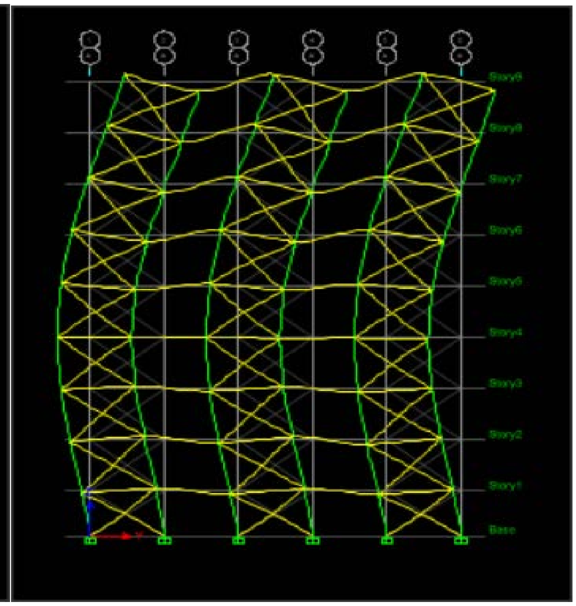
Modal	4	0.399	2.506	15.7466	247.9553
Modal	5	0.376	2.662	16.7233	279.6673
Modal	6	0.238	4.204	26.4121	697.6017
Modal	7	0.218	4.595	28.8699	833.473
Modal	8	0.206	4.861	30.5422	932.8272
Modal	9	0.155	6.469	40.6466	1652.1456
Modal	10	0.145	6.885	43.2597	1871.403
Modal	11	0.131	7.638	47.9937	2303.3947
Modal	12	0.123	8.146	51.1819	2619.587

5.8 Modes Shapes

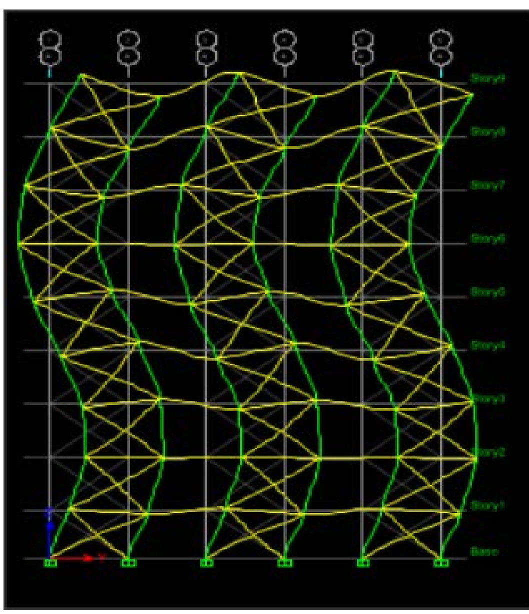
5.8.1 For Concentric Braced Struture:



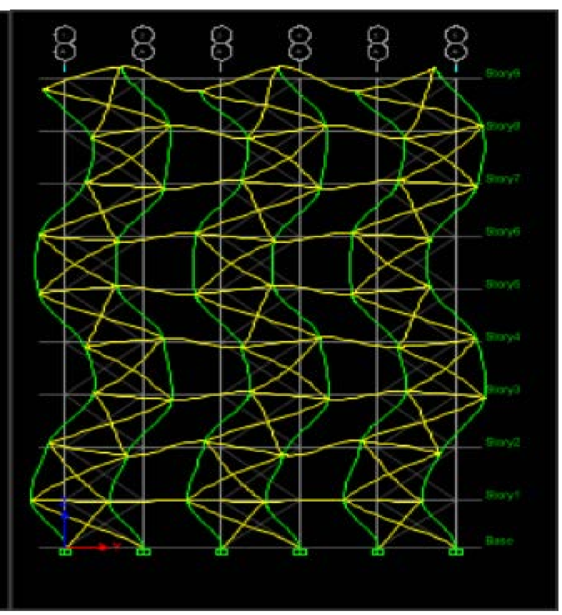
Mode 1



Mode4



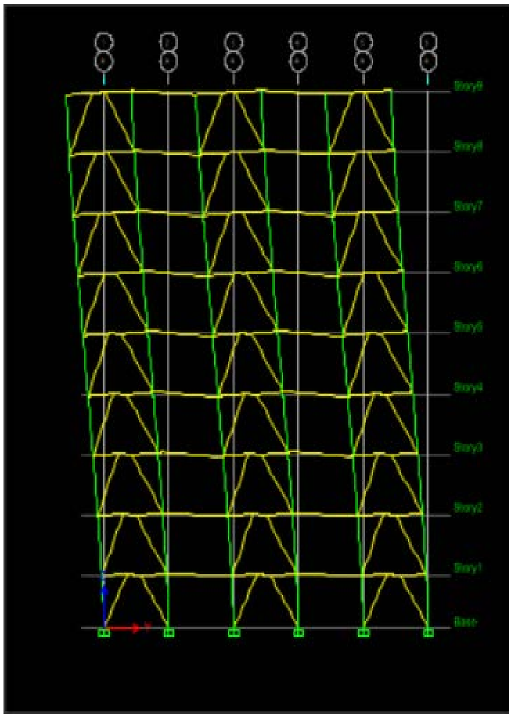
Mode 7



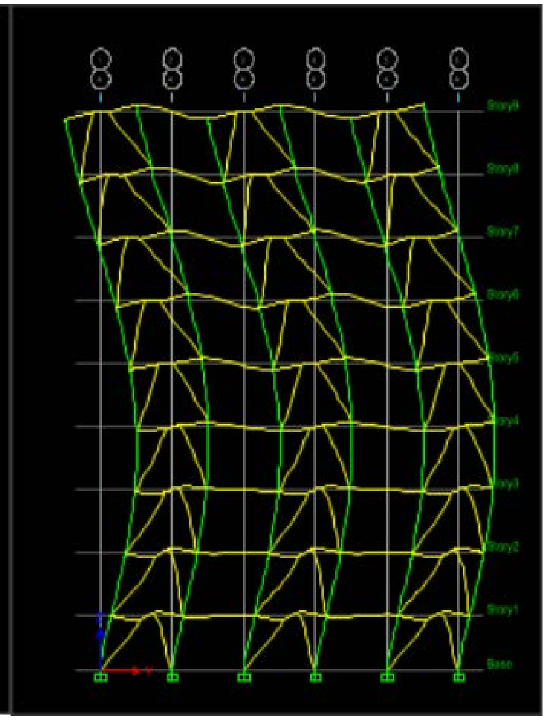
Mode 12

Fig 19. Concentric modes

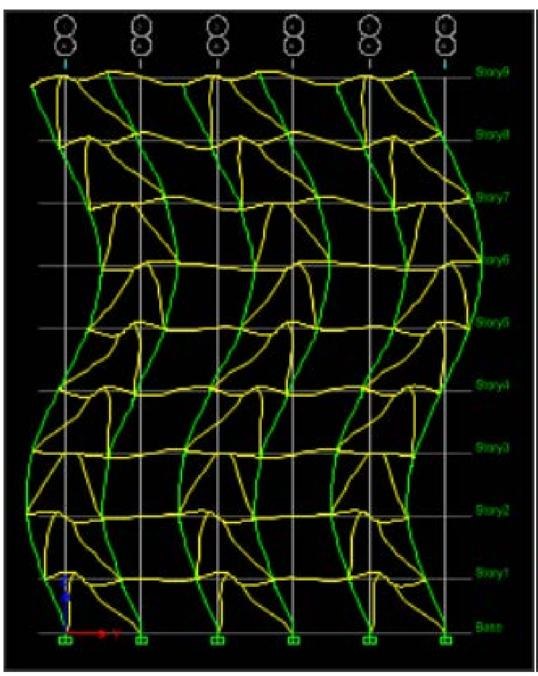
5.8.2 For Eccentric Braced Structure:



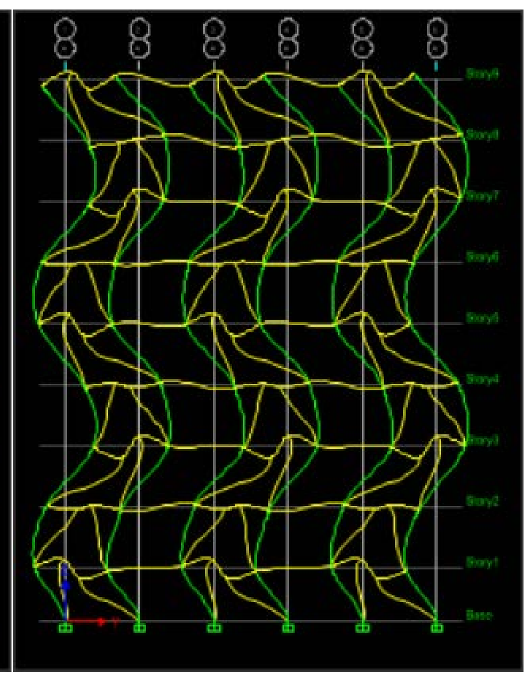
Mode 1



Mode 4



Mode 7



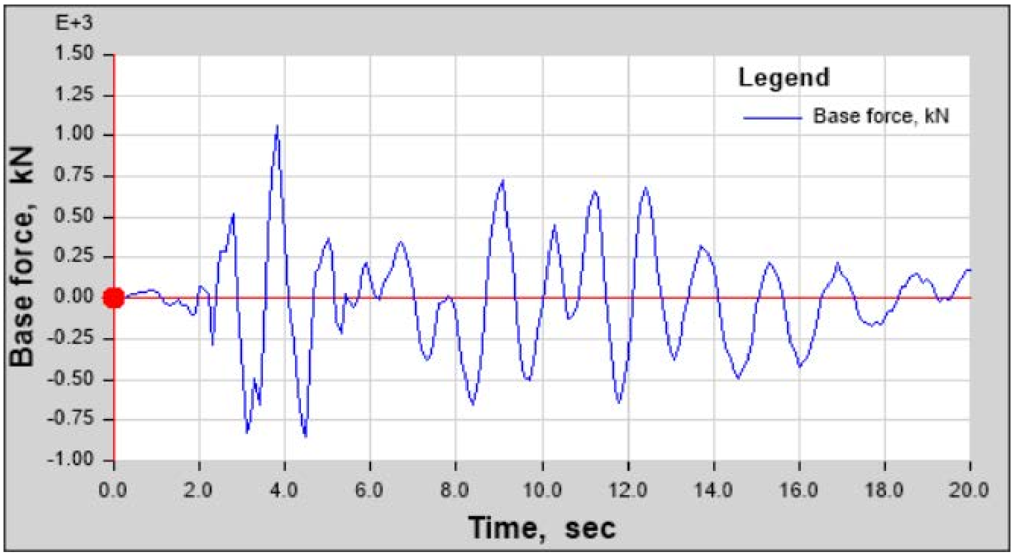
Mode 12

Fig 20. Eccentric modes

5.9 Time Histroy Plots

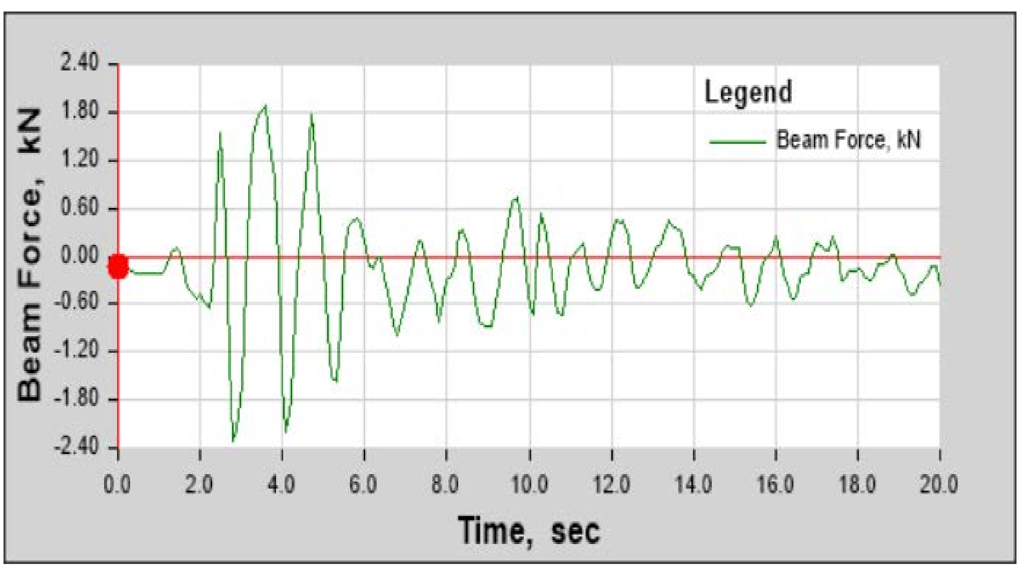
5.9.1 For Eccentric Braced Struture:

1. Base Force Vs Time



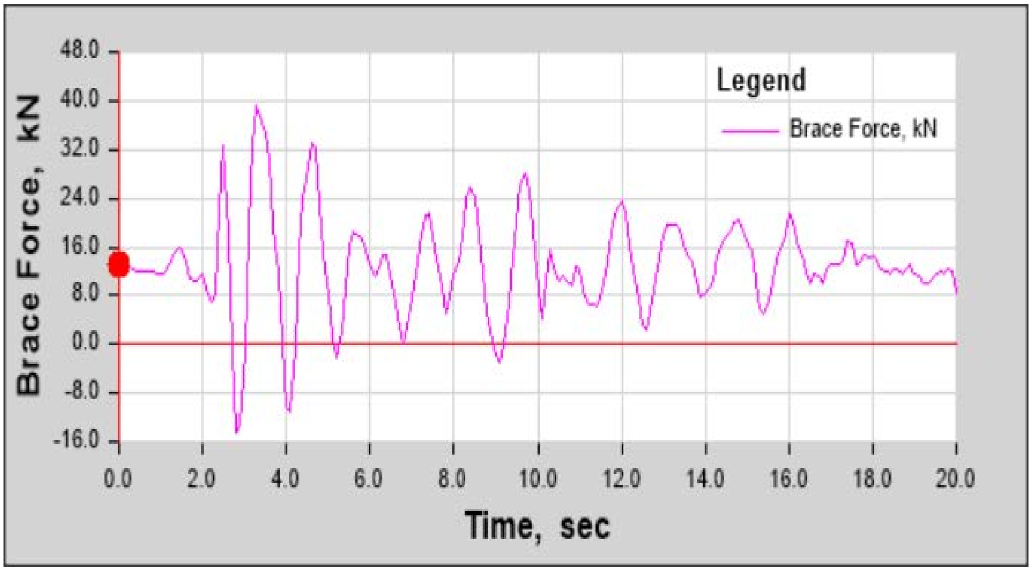
2. Beam Force Vs Time

   For Beam B31 AtStorey 9:



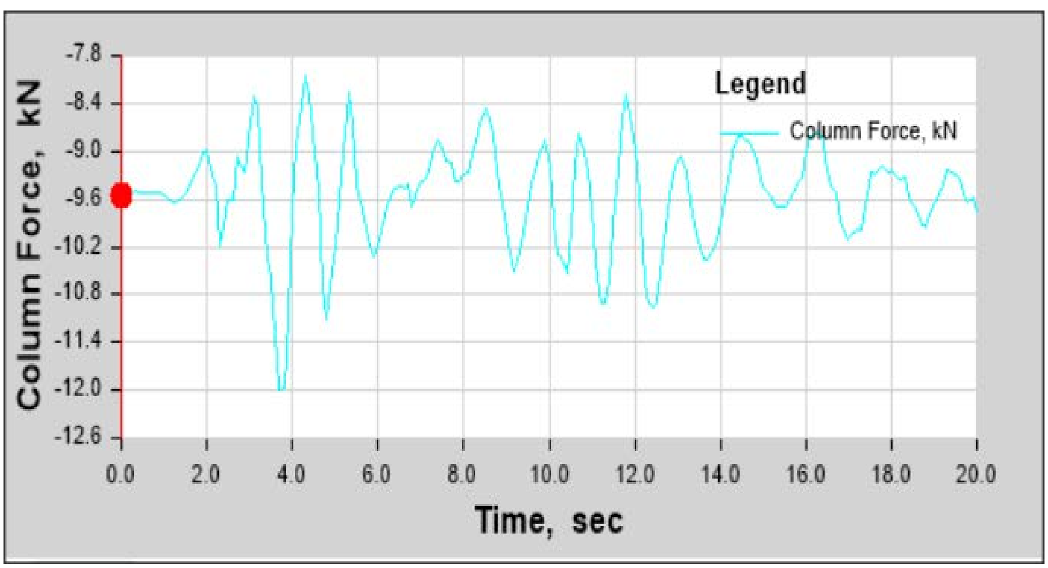
3. Brace Force Vs Time

For Brace D13 At Storey 9



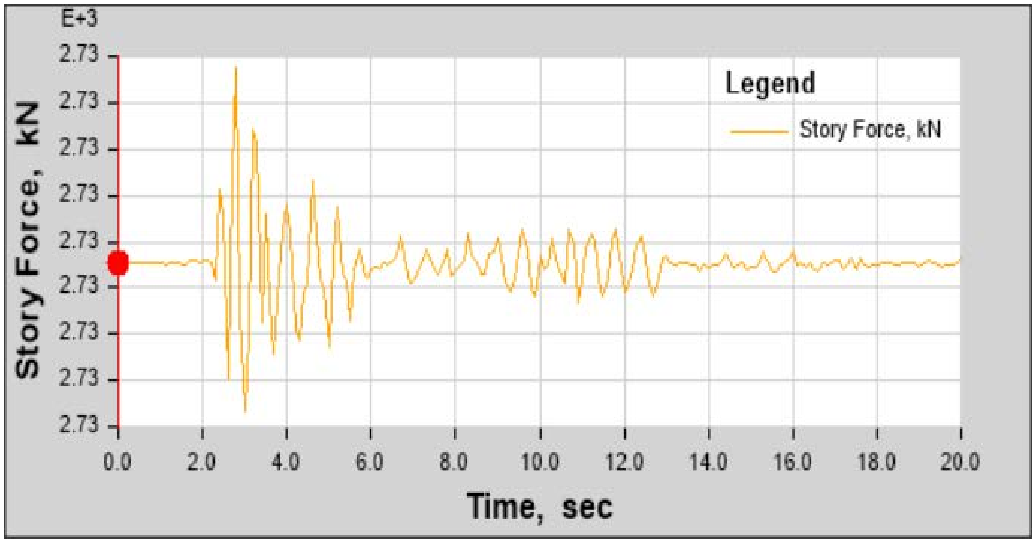
4. Column Force vs Time

For column C1 at Storey 9



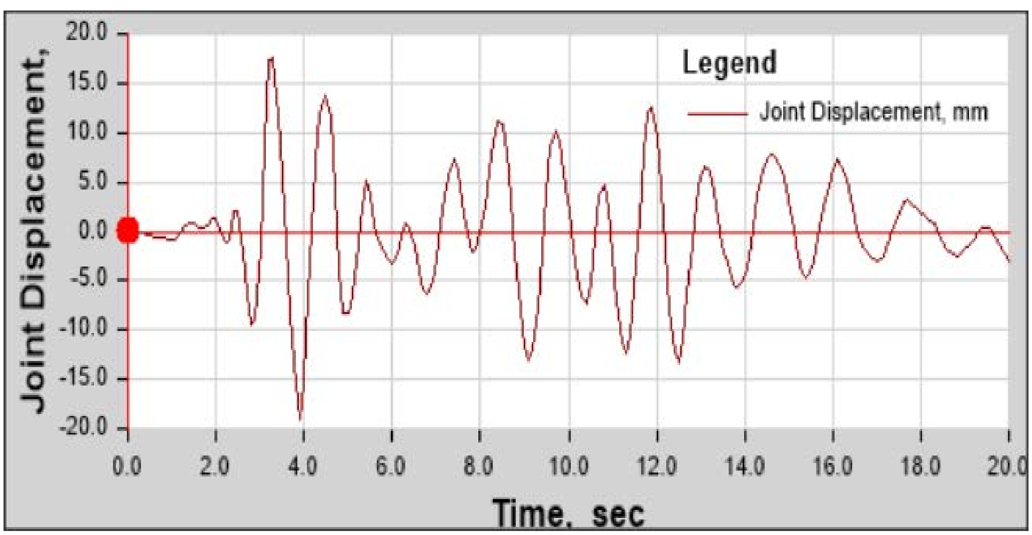
5. Storey Force vs Time

At storey 9:



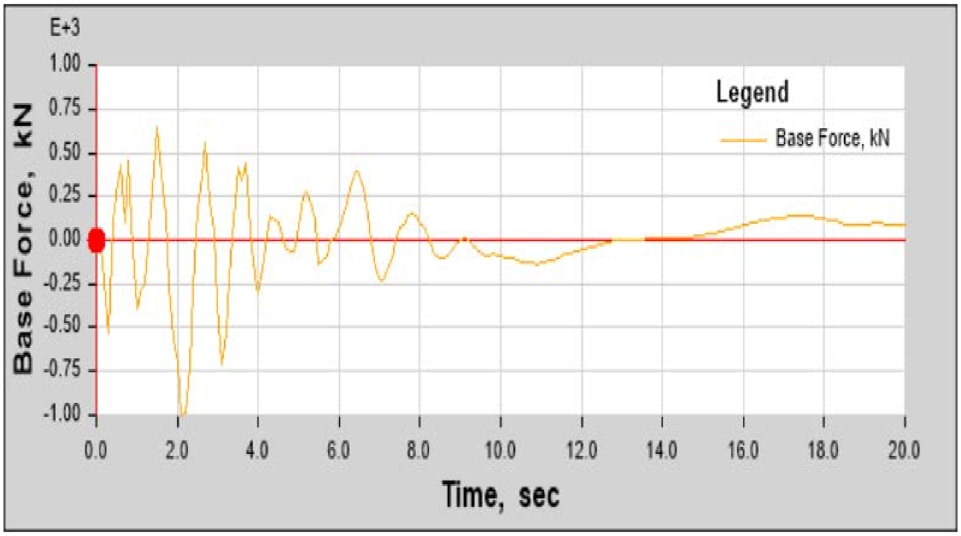
6. Jiont Displacements vs Time

Joint 1 at Storey 9:



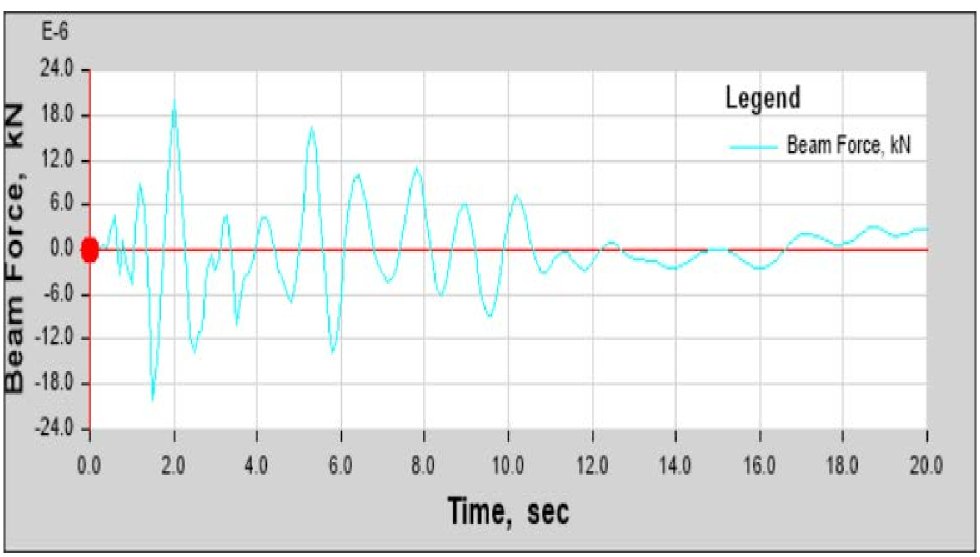
5.9.2 For Concentric Braced Struture:

1. Base Force Vs Time



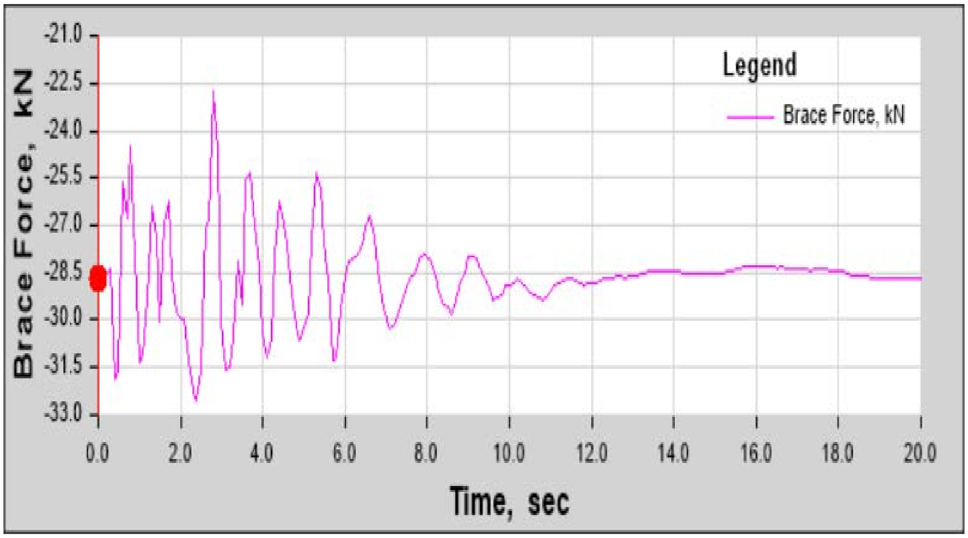
2. Beam Force Vs Time

   For Beam B1 Storey 9:



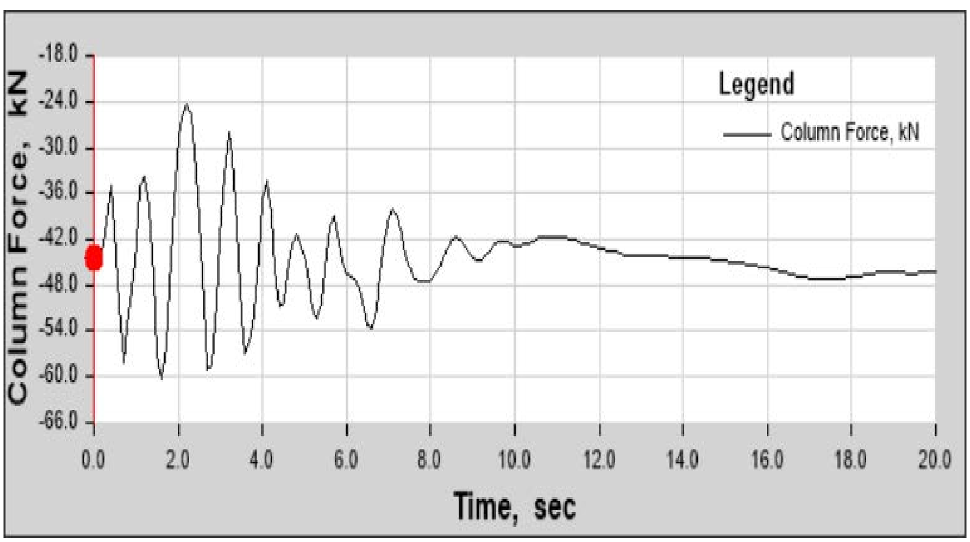
3. Brace Force Vs Time

For Brace D1 at Storey 9:



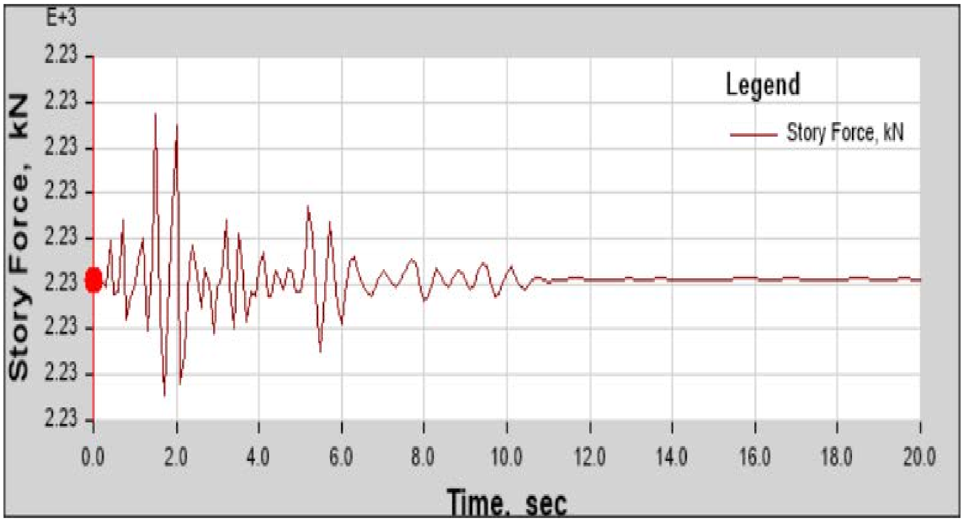
4. Column Force vs Time

For Column C1 at storey 9



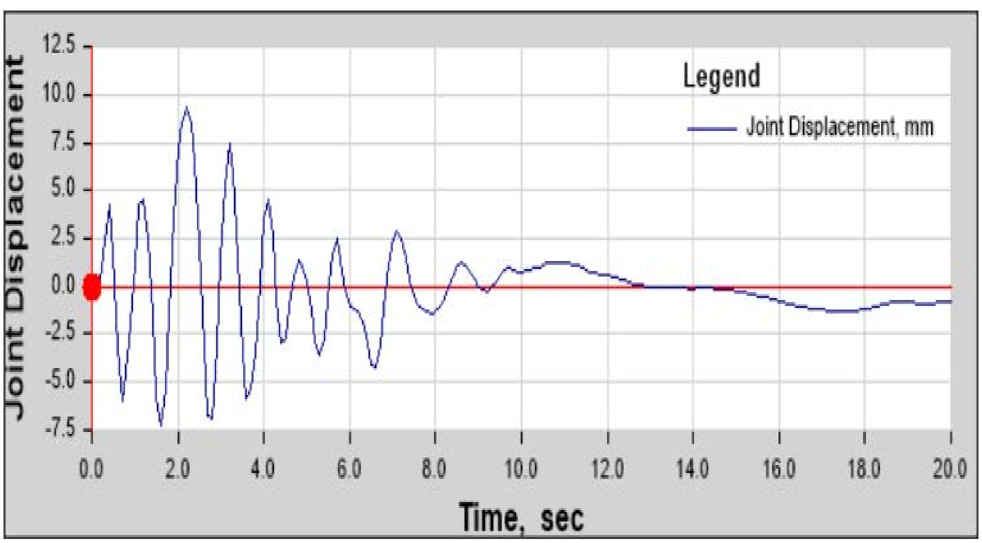
5. Storey Force vs Time

At Storey 9:



6. Jiont Displacements vs Time

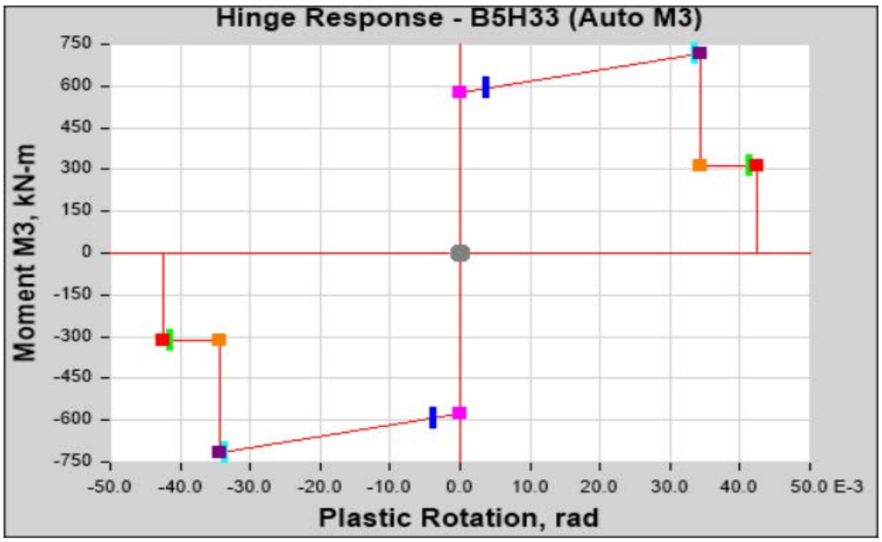
Joint 1 At Storey 9:



5.10 Hinges Result

5.10.1 For Concentric Braced Struture:

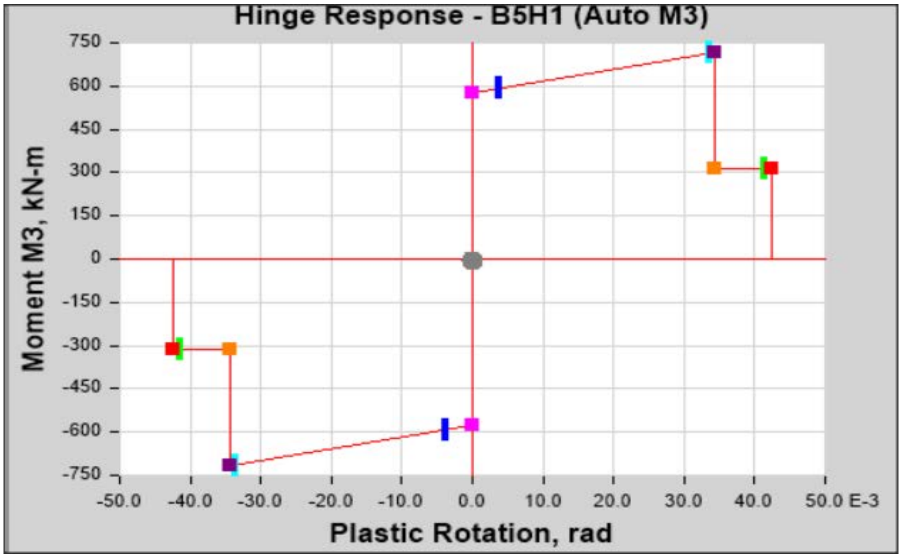
1. For Beam 5 at Storey 1:
   B5-Hinge 33 (Auto M3)



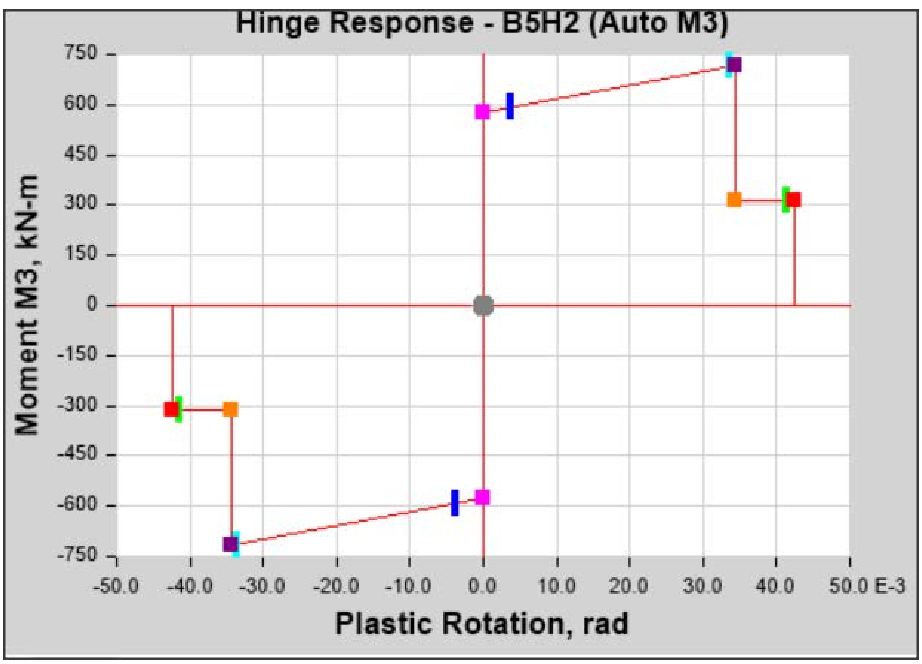
B5H34 (Auto M3)

2. For Beam B5 Storey 9:

B-5Hinge1 (Auto M3)

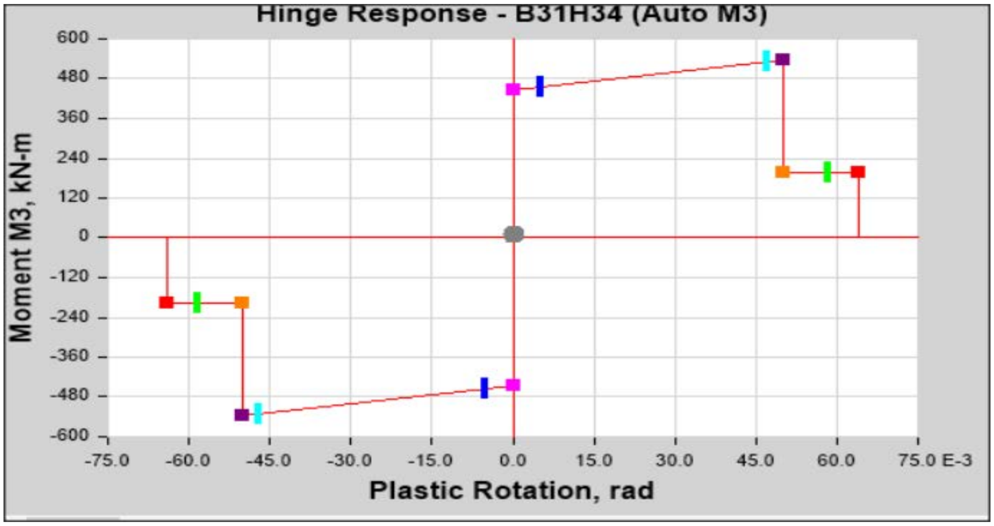


B-5Hinge 2 (Auto M3)

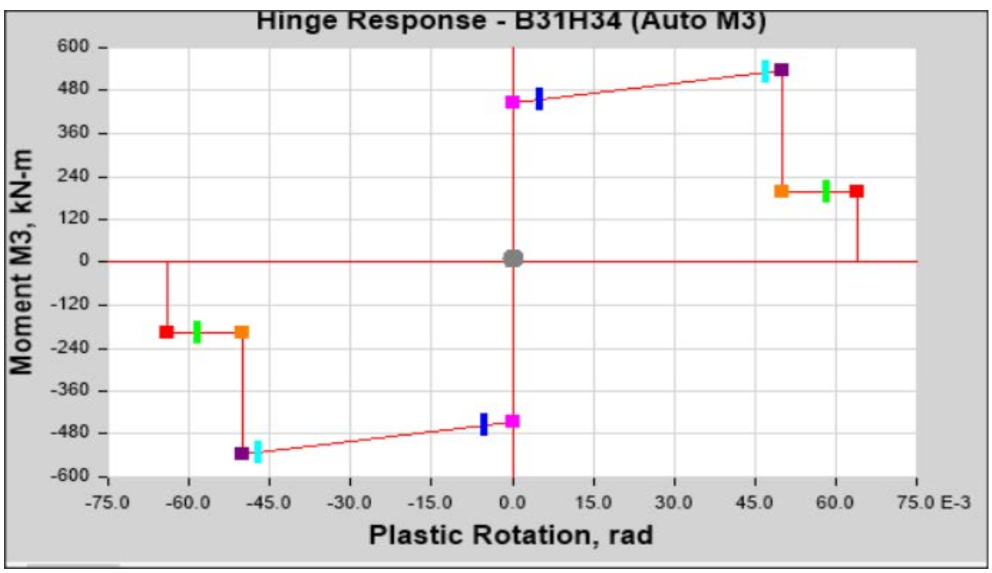


5.10.2 For Eccentric Braced Struture:

1. For Beam 35 at Storey 1:
   B35-Hinge 33 (Auto M3)

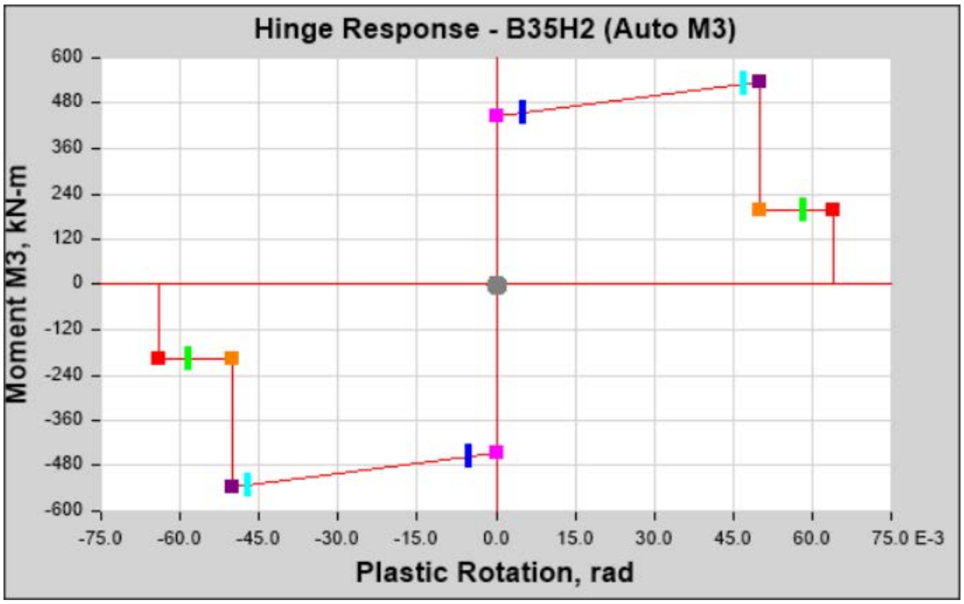


B35-Hinge 34 (Auto M3)

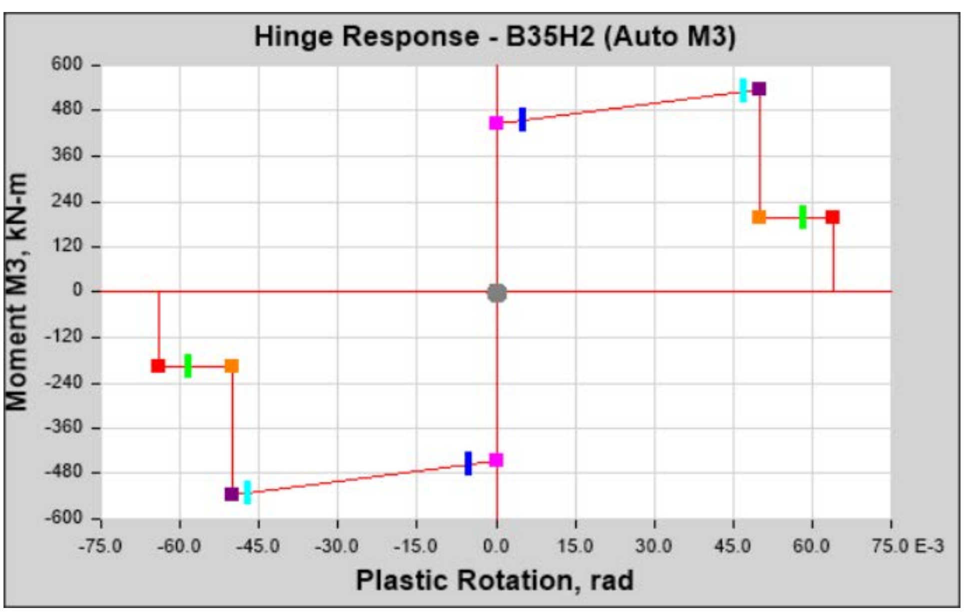


2. For Beam 35 at Storey 9:

B35-Hinge 1 (Auto M3)



B35-Hinge 2 (Auto M3)



These are graphs showing hinges results for concentric and eccentric bracing systems, they plot plastic rotation in radians to moments in Kn-m as the load on the beam goes on increasing the moments also increases and finally its reaches to it largest moment carrying capacity then our first plastic hinge is formed. There after moment carrying capacity Drastically decreases, remain constant some while and eventually becomes zero thus final failure happens. This phenomenon can been seen for different beams and column in the non linear analyses.

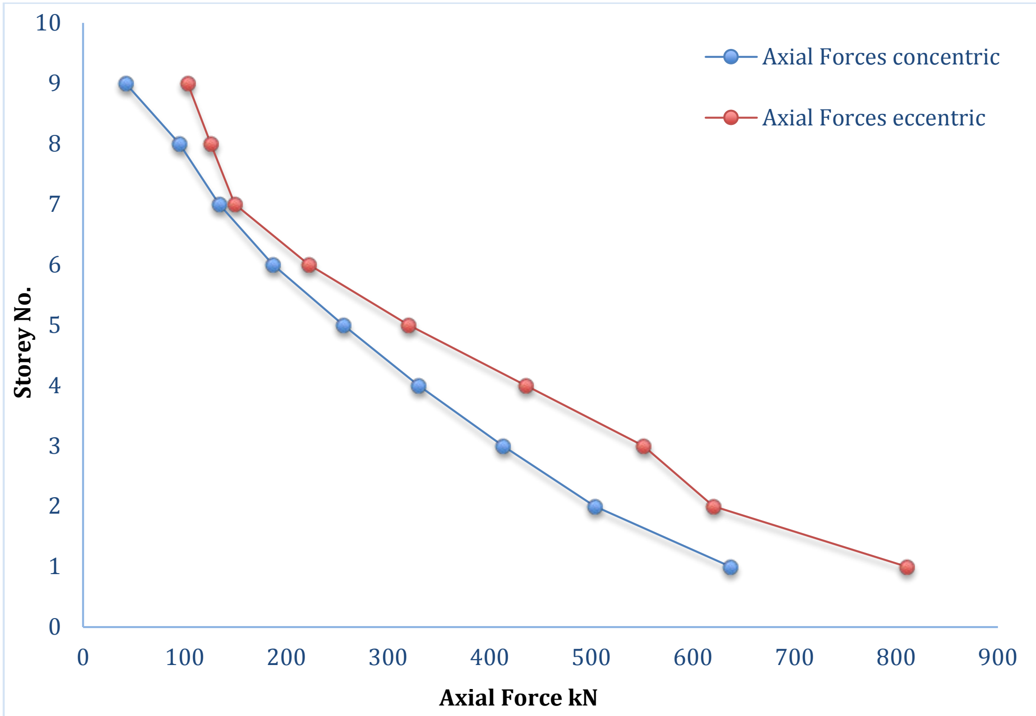
5.11 Axial Force Comparison on Corner Column (Floor Wise)

Table 10 Column Axial Force

Storey	Axial Forces concentric	Axial Forces eccentric
9	41.8	102.9
8	94.58	125.6
7	133.89	149.12
6	186.65	222
5	255.9	320.1
4	329.8	435.2
3	413.12	551.2
2	503.3	620.1
1	636.9	810.256

In braced frames, beams and columns are combined to the point that they structure a truss framework and in this, we realize that forces are exchanged mostly by axial forces. In this way, in brace outlines, we can see a lot of increment in axial powers. It is noticed that the axial forces are more in brace frameworks then a framework without bracing. Axial forces at the ground floor level section for the framework with X propping is expanded by 27.22% when contrasted with that of unusual V supporting. In this way, we can say that Concentric X bracings, for the most part, exchanges power by axial forces so they should have lower shear powers and moments and this can be seen above aftereffects of shear power and moments examination corner segments.

Average expansion for all floor levels for the framework with X propping when contrasted with that of unusual V supporting 38.91%.

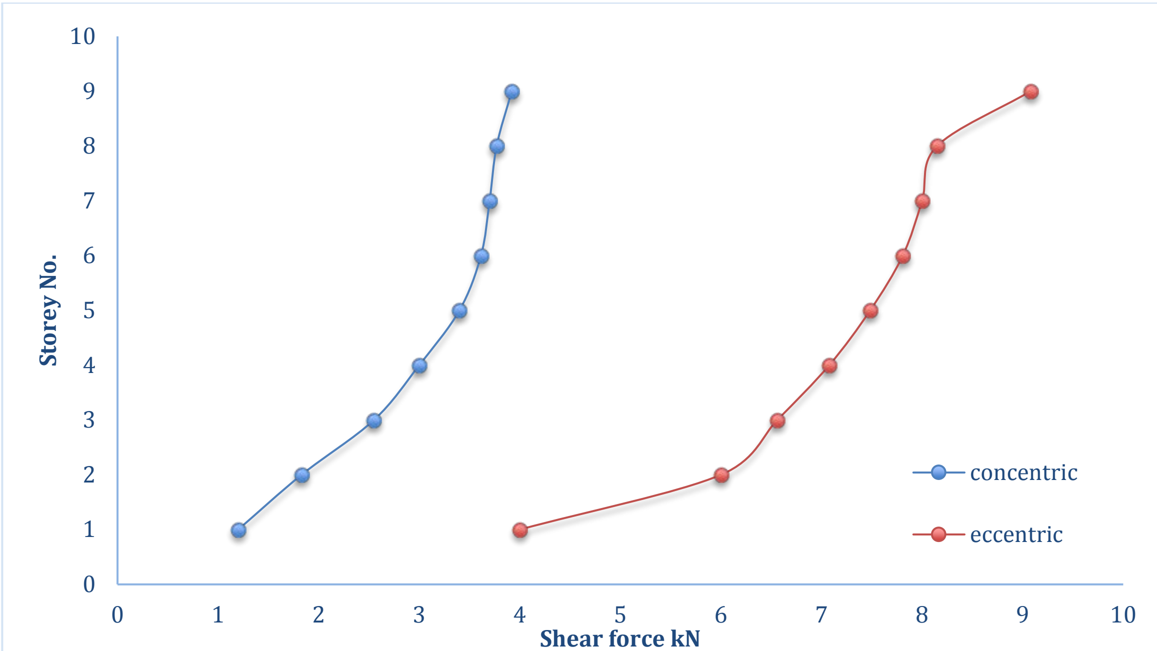


5.12 Shear Force Comparison on Corner Column (Floor Wise)

Table 11 Column Shear Force

Storey	Shear Forces concentric	Shear Forces eccentric
9	3.92	9.081
8	3.77	8.1502
7	3.7	8
6	3.614	7.808
5	3.4	7.4842
4	3	7.077
3	2.55	6.56
2	1.83	6
1	1.2	4

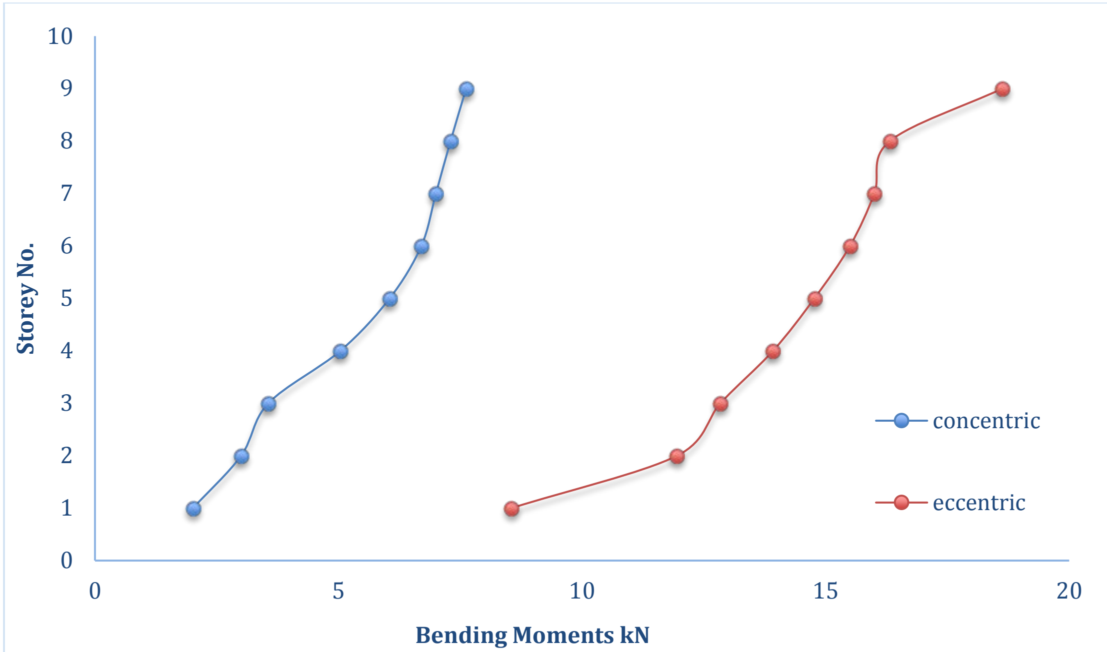
It tends to be seen that the shear power delivered on each floor corner section are least for Concentric X kind of bracing frameworks, while these are greatest for the framework with no supporting. Shear powers at the sections for the arrangement of concentric X supporting are diminished by 58.98% (average reduction for all floor) when contrasted with that of Eccentric V bracings.



5.13 Bending Moments Comparison on Corner Column (Floor Wise)

Table 12 Column Bending Moments

Storey	Bending Moments concentric	Bending Moments eccentric
9	7.62	18.63
8	7.3	16.32
7	7	16
6	6.7	15.5
5	6.05	14.77
4	5.038	13.92
3	3.56	12.84
2	3	11.95
1	2.012	8.55

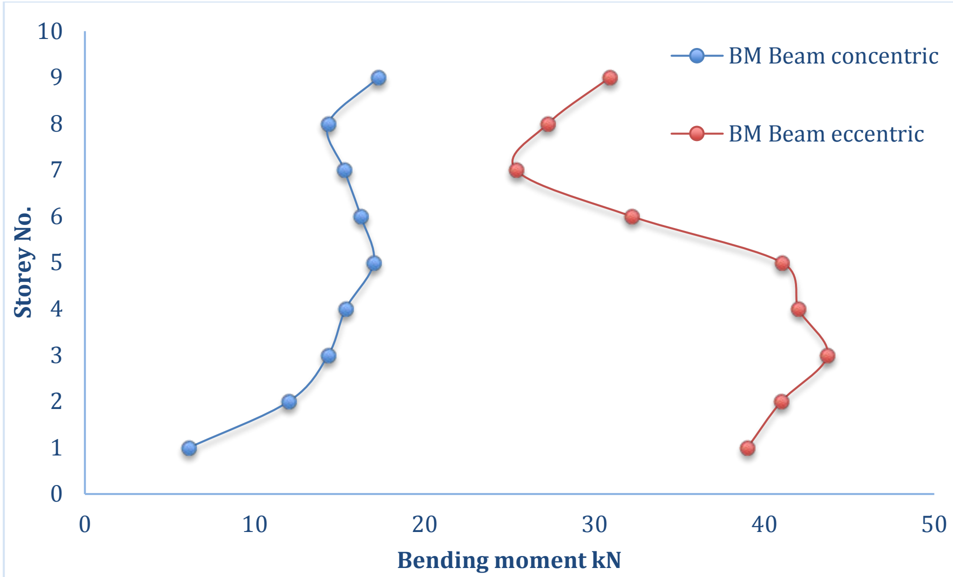


It can be observed that the bending moment at corner column of each floor are minimum for X type of bracing systems, while these are maximum for the system without bracing.Bending moment at the beam for the system with X bracing is decreased by 63.76% as compared to that of Eccentric V bracings.

5.14 Bending Moments Comparison on Beams (in braced frame at each level)

Table 13 Beam Bending Moments

Storey	BM Beam concentric	BM Beam eccentric
9	17.28	30.90
8	14.32	27.25
7	15.27	25.38
6	16.24	32.18
5	17	41.039
4	15.35	42
3	14.32	43.71
2	12	41
1	6.12	39

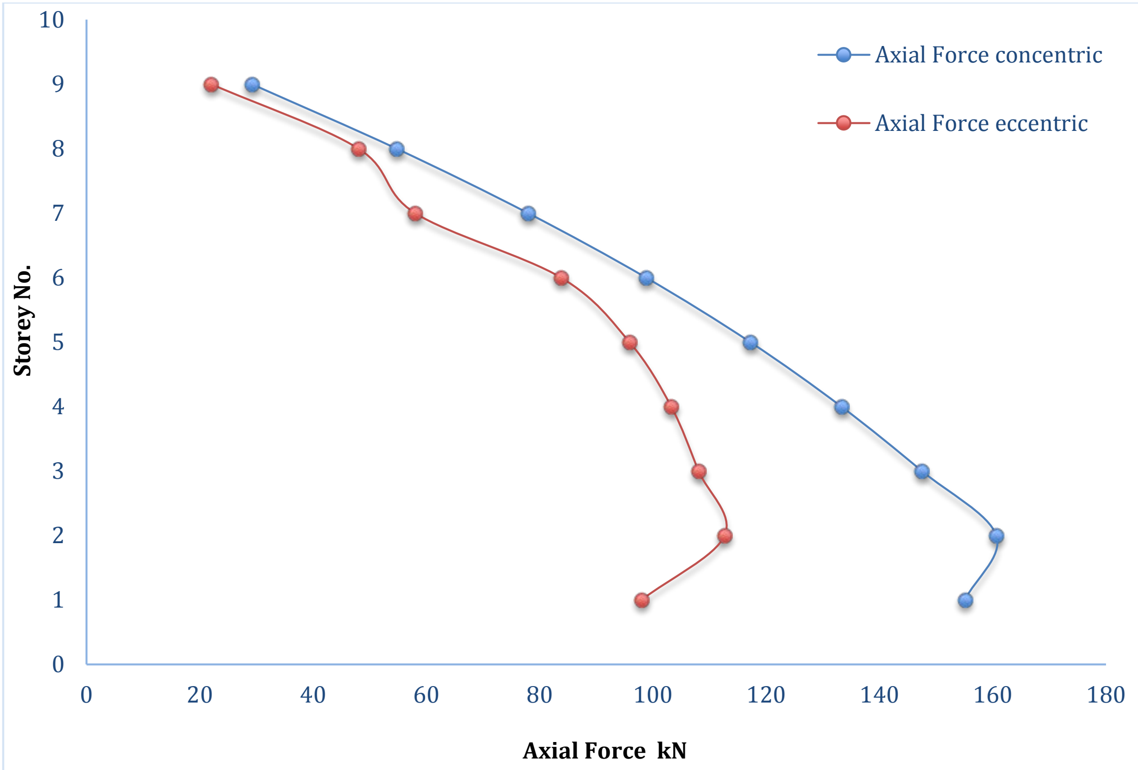


It is seen that the moment is least for X sort of bracing systems, while these are most extreme for the framework without bracing. It is because of that reality that braced frames for the most part exchange forces by axial because of the vertical truss framework along these lines, this outcome in lesser moments and lesser shear.Bending moment at the pillar for the framework with X propping is diminished by 58.36% contrasted with eccentric V bracing.

5.15 Axial Force Comparison on Braces (Floor Wise)

Table 14 Brace Axial Force

Storey	Axial Force concentric	Axial Force eccentric
9	29.16	22
8	54.72	48
7	77.93	58
6	98.77	83.76
5	117.17	95.86
4	133.26	103.23
3	147.48	108.07
2	160.63	112.67
1	155.08	98

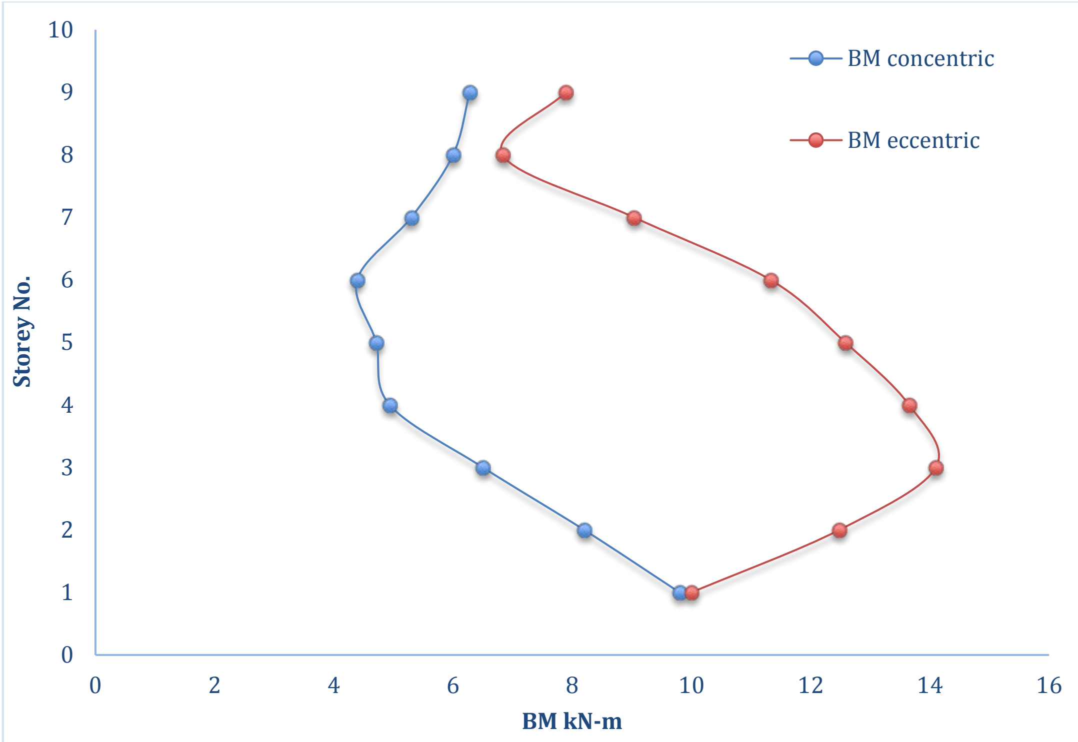


Axial force on braces in eccentric bracing reduced by 23.52% when compared to the same in concentric bracing.

5.16 Bending Moments Comparison on Braces (Floor Wise)

Table 15 Beam Bending Moments

Storey	BM concentric	BM eccentric
9	6.28	7.89
8	6	6.83
7	5.3	9.034
6	4.39	11.325
5	4.71	12.58
4	4.936	13.65
3	6.5	14.1
2	8.2	12.48
1	9.801	10



Bending moment on braces increases in eccentric bracing by the average amount of 86.96% when compared to that of concentric bracing.

CONCLUSION

- With the utilization of bracings in the steel edge structure, a critical increment in base shear is seen which shows that stiffness of the structure is expanded.
- Concentric X braces have the most elevated base shear and most noteworthy horizontal stiffness.
- A decrease in the drift is seen by the utilization of Concentric X type steel supporting framework which is more than the Eccentric X propping when both these models are contrasted against the unbraced edge.
- A decrease of in Bending moments is seen on corner segments by the utilization of Concentric X type steel supporting framework which is more than the Eccentric X propping when both these models are contrasted against the unbraced edge.
- A decrease in shear force is seen on corner segments by the utilization of Concentric X type steel propping framework which is more than the Eccentric X supporting when both these models are contrasted against the unbraced casing.
- Axial force on base sections increments in braced structure as a contrast against unbraced structures. Concentric X has the most astounding Axial force following with Concentric V, Eccentric X and Eccentric V.
- Concentric X kind of steel bracings give the best outcomes in steel outlines under seismic stacking.
- Concentric propping gives more firmness to the structure while unusual supporting gives greater malleability in the structure. In this way, we can say Eccentrically propped edges are the blend of unbraced and concentrically supported edges.
- By utilizing a comparable amount of steel we have planned an all the more horizontally hardened structure by utilizing bracings in the steel outlines.

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