RESEARCH ARTICLE OPEN ACCESS

# Seismic Analysis of Steel Frames Subjected to Braced Connections

**Matha Prasad Adari***

*Assistant Professor, Dept. of Civil Engineering. NSRIT
Visakhapatnam, Andhra Pradesh, INDIA.

## 1.INTRODUCTION

In the present time, Steel structure plays an important role in the construction industry. Previous earthquakes in India show that not only non-engineered structures but engineered structures need to be designed in such a way that they perform well under seismic loading. Structural response can be increased in Steel moment resisting frames by introducing steel bracings in the structural system. Bracing can be applied as concentric bracing or eccentric bracing. There are 'n' number of possibilities to arrange steel bracings, such as cross bracing 'X', diagonal bracing 'D', and 'V' type bracing.

Steel moment resisting frames without bracing, inelastic response failure generally occurs at beam and column connections. They resist lateral forces by flexure and shear in beams and columns i.e. by frame action. Under severe earthquake loading ductile fracture at beams and columns connections are common. Moment resisting frames have low elastic stiffness. P-Δ effect is an another problem associated with such structures in high rise buildings.

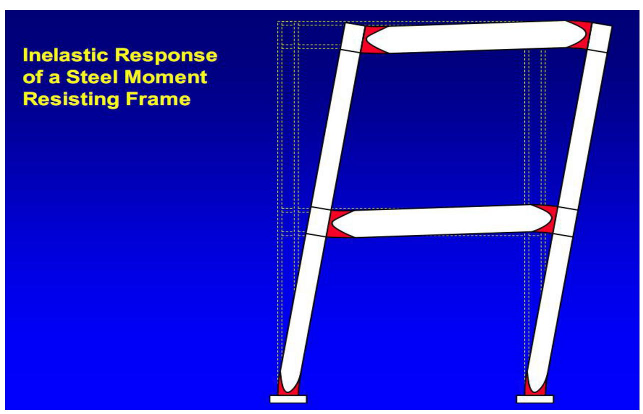



FIG 1.1 *(ref. Praval Priyaranjan,dspace@nitd.ac.in)*

So, to increase the structure response to lateral loading and good ductility properties to perform well under seismic loading concentric bracings can be provided. Beams, columns and bracings are arranged to form a vertical truss and then lateral loading is resisted by truss action. Bracings allow the system to obtain a great increase in lateral stiffness with minimal added weight. Thus, they increase the natural frequency and usually decrease the lateral drift. They develop ductility through inelastic action in braces. Failure occurs because of yielding of truss under tension or buckling of truss under compression. These failures can be compensated by use of Buckling Reinforced Braced frame (BRBs) or Self Centering Energy Dissipating frames (SCEDs).

The present study will clearly estimate the advantage of concentrically braced steel frames over Steel moment resisting frames. A simple computer based modeling in Staad Pro. Software is

performed for Equivalent static analysis, Response spectrum analysis, and linear Time history analysis subjected to earthquake loading.

## 2.1 INTRODUCTION

This chapter deals with a brief review of the past and recent study performed by researchers on seismic analysis of braced steel frames. A detailed review of each literature would be difficult to address in this chapter. The literature review focusses on concentrically braced frames, failure mode generally observed in moment resisting frames and bracings, brace to frame connections, local buckling and plastic hinge formation. The recent study of use of Buckling reinforced bracing (BRBs) and Self centered energy dissipating frames (SCEDs) is also mentioned.

## 2.2 LITERATURE REVIEWS

- ***Tremblay et al. ,(ASCE)0733-9445(2003)***
  Tremblay et al. 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 sequence 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.
- ***Khatib et al. (1988)***
  The failure mode generally observed in special moment resisting frames with bracing system is fracture of bracings at the locations of local buckling or plastic hinges. Significant story drift can occur at a single story and this research shows how the failure mode occurs and how the failure is concentrated entirely on single floor . So, this is one of the limitations of using moment resisting frames with bracing system.
- ***Seismic response of Steel braced reinforced concrete frames by K.G.Vishwanath in International journal of civil and structural engineering (2010)***
  A four storey building was taken in seismic zone 4 according to IS 1893:2002 . The performance of the building is evaluated according to story drift. Then the study is extended to eight story and twelve story. X type of steel bracing is found out to be most efficient.

- ***Seismic response assessment of concentrically braced steel frame buildings (The 14th World conference on earthquake engineering October 12-17, 2008, Beijing,***

***China)*** Improvement of performance based design and analysis procedure for better understanding of conventionally used concentrically braced frame and buckling restrained braced frames is discussed.

## 3.1 EQUIVALENT STATIC ANALYSIS – AN OVERVIEW

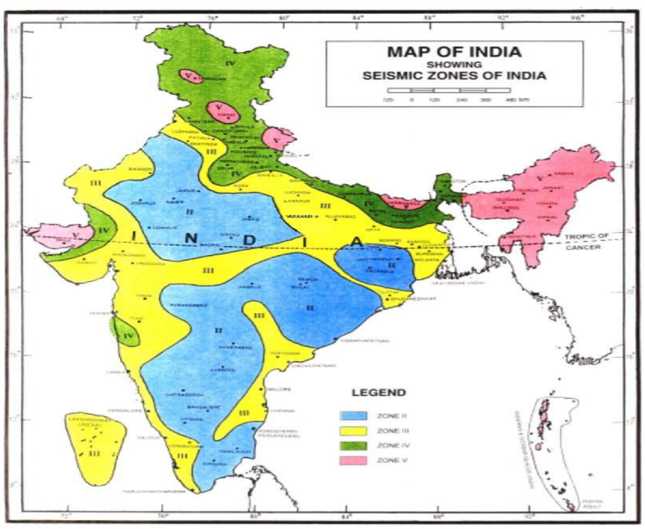
The equivalent static method is the simplest method of analysis. Here, force depend upon the fundamental period of structures defined by IS Code 1893:2002 with some changes. First, design base shear of complete building is calculated, and then distributed along the height of the building, based on formulae 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:

### Determination of Base shear

For determination of seismic forces, the country is classified in four seismic zones:

Fig 3.1 shows seismic zones of India *(ref., IS 1893:2002)*




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

$$V = AW \tag{3.1}$$

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 :

$$A = (ZIS_a)/ 2Rg \tag{3.2}$$

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, $S_a/g$ 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.

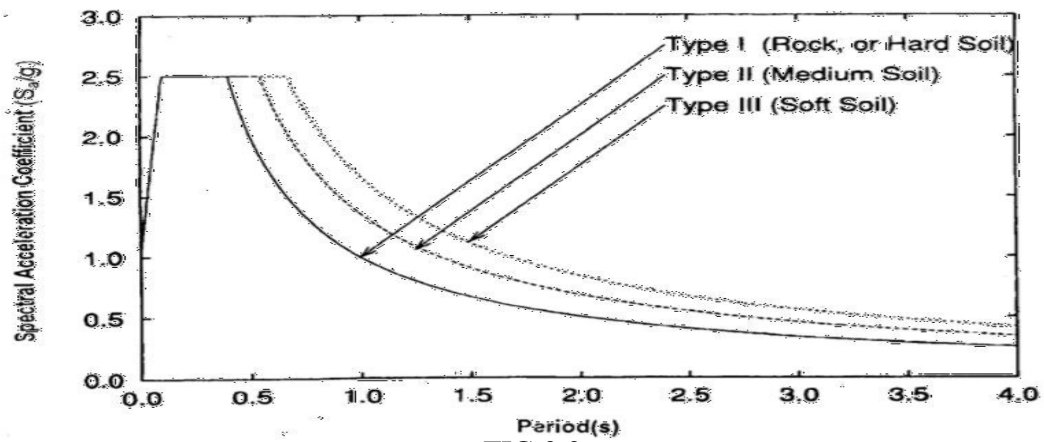



FIG 3.2

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

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

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

$T_a = 0.09h/\sqrt{d}$ 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.

**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 storyed 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

IS 1893:2002 (part 1) uses a parabolic distribution of lateral force along the height of the building. Distribution of base shear along the height is done according to this equation:

$$\mathbf{Q_i = W_i h_i^2 / \sum_{j=1}^{n} (W_j h_j^2)}$$

Where:

$Q_i$ = design lateral force at floor i

$W_i$ = seismic weight at floor i

$h_j$ = height of floor I measured from foundation

n = number of stories in the building or the number of levels at which masses are located.

## 3.2 RESPONSE SPECTRUM ANALYSIS - AN OVERVIEW

Response spectrum analysis is a procedure for calculating the maximum response of a structure when applied with ground motion. Each of the vibration modes that are considered are assumed to respond independently as a single degree of freedom system. Design codes specify response spectra which determine the base acceleration applied to each mode according to its period (the number of seconds required for a cycle of vibration).

Having determined the response of each vibration mode to the excitation, it is necessary to obtain the response of the structure by combining the effects of each vibration mode because the maximum response of each mode will not necessarily occur at the same instant, the statistical maximum response, where damping is zero, is taken as sum of squares (SRSS) of the individual responses.

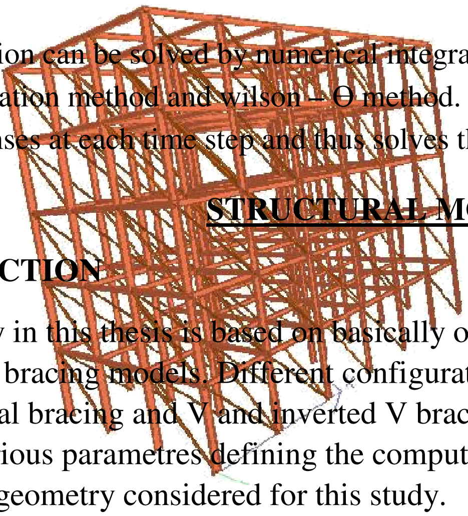
The results of response spectrum are all absolute extreme values and so they need to be combined as they do not correspond to any equilibrium state nor they take place at the same time. There are several methods to execute this, one of them being the (SRSS) method, Square root of sum of squares method. In this method , the maximum response in terms of given parameter, G (displacement, acceleration, velocity) may be estimated through the square root of sum of m modal response squares, contributing to global response:

$$\mathbf{G = \sum^{m}{}_{n=1} (Gn)^2}$$

## 3.3 TIME HISTORY ANALYSIS- AN OVERVIEW

It is a linear or non linear analysis of dynamic structural response under the loading which may differ according to specified time function. The basic governing equation for the dynamic equation for dynamic response of multi degree of freedom system is given by equation 3.4.

The given equation can be solved by numerical integration method such as Runge-kutta method, Newmark integration method and wilson – Θ method. The staad pro. Software calculates the structural responses at each time step and thus solves the governing time equation.



## STRUCTURAL MODELLING

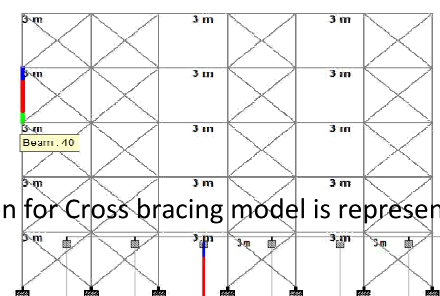
### 4.1 INTRODUCTION

The study in this thesis is based on basically on linear time history analysis of steel frames with concentric bracing models. Different configurations of frames are selected such as cross bracing, diagonal bracing and V and inverted V bracing and analyzed. This chapter presents a summary of various parametres defining the computational models, the basic assumptions and the steel frame geometry considered for this study.

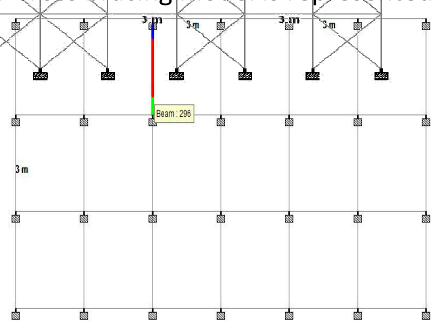
### 4.2 FRAME GEOMETRY

Model 1 is an asymmetric plan . Model 2 is a symmetric plan and hence a single plane frame is considered to be representative of building in one direction.

For Model 1, plan is represented in fig4.1.

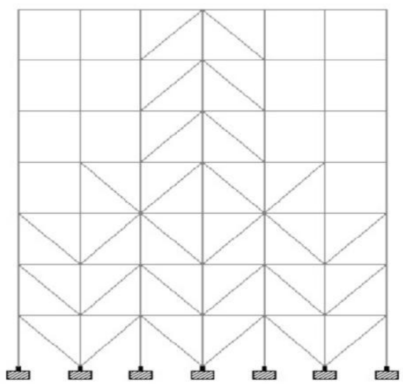



And the front elevation for Cross bracing model is represented in fig4.2.




A 3D view of the typical steel frame with diagonal bracing is represented as FIG 4.4

Model 2 is a symmetric plan, and hence plane frame used for analysis. Variation is both the brame is height of the building.



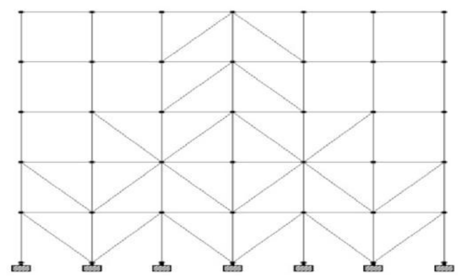


FIG 4.5 FIG 4.6 The bay width is 2m and the story

height is 3m at each floor level.

**4.3 FRAME DESIGN**

The building frame used in this study is assumed to be located in Indian seismic zone IV with medium soil conditions. Seismic loads are estimated as per IS 1893:2002 and design of steel elements are carried as per IS 800 (2007) standards. The characteristic strength of steel is considered 415 $N/mm^2$. The gravity loading consists of the self weight of the structure, a floor load of $3kN/m^2$ on every floor except the roof , the roof floor load is taken $2kN/m^2$. The design horizontal seismic coefficient ($A_h$ ) is calculated as per IS 1893:2002

$A_h = ZI/2R$,

Where, seismic zone factor, Z = 0.24, Importance factor I = 1.0, Response reduction factor, R = 3.0.The design base shear (VB) is calculated as per IS 1893:2002

$VB = A_h.S_a/g.W$

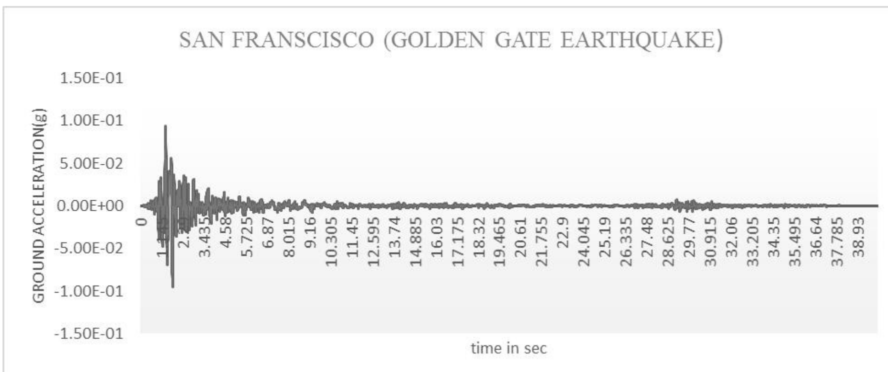
Period for analysis = $0.085H^{0.75}$, which is found to be 0.647 sec.

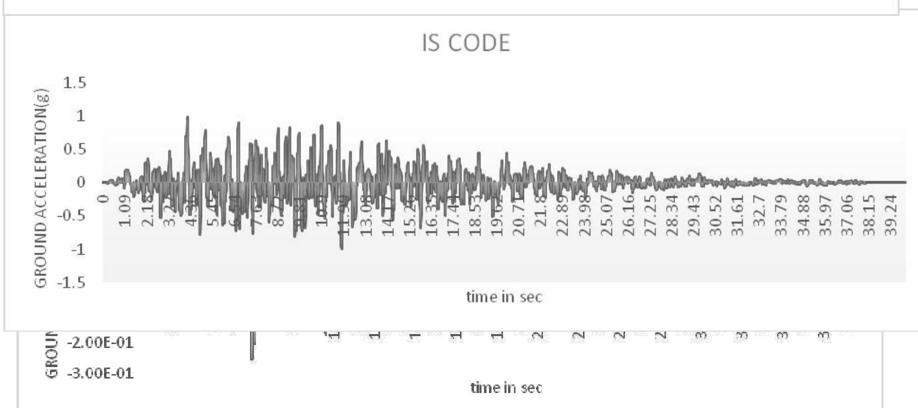
Estimated design base shear from above formula is found to be 18.12 kN for without bracing. By SRSS method, in response spectrum analysis, the total base shear was found to be 22.84 considering 6 modes of participation for without bracing. A comparison between them will be shown later.

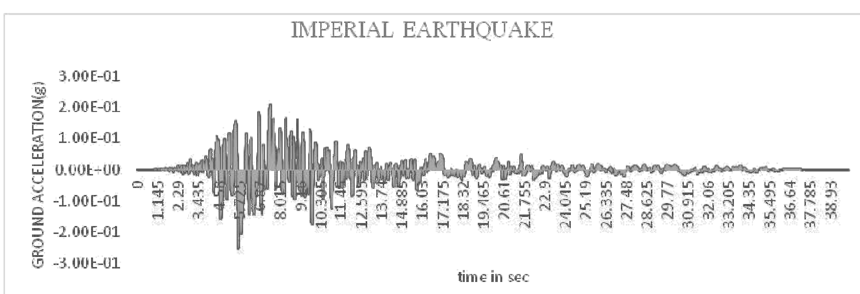
Every beam used in the both the models is ISMC 200. Every column used in the model is ISMC 300 and for bracings angle section are used. Every bracing is an angle section IS 75x75x5.

## TIME HISTORY LOADING

The three earth quake used for analysis are as follows:



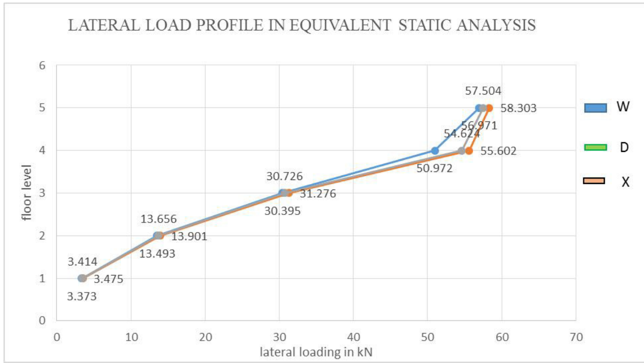




## SEISMIC RESPONSE OF STEEL FRAME UNDER DIFFERENT BRACING CONFIGURATION AND LOADING

## 5. 1 MODEL 1

### 5.1.1 LATERAL LOAD PROFILE




Cross bracing have the highest lateral stiffness as compared to diagonal bracing, and obviously to frame without bracing. A increase in stiffness attracts larger inertia force and this is evident from the graph.

### 5.1.2 BASE SHEAR COMPARISION OF MODAL ANALYSIS (RESPONSE SPECTRUM ANALYSIS) WITH IS CODE 1893:2002 CALCULATED DESIGN BASE SHEAR

| | | |
|---|---|---|
| WITHOUT BRACING | 18.12 | 22.84 |
| WITH DIAGONAL BRACING | 22.24 | 23.99 |
| WITH CROSS BRACING | 24.64 | 24.27 |

From table, it is evident that the design base shear provided by the code is less as compared to by modal analysis. A 26% increase in design base shear is observed in moment resisting frame without bracings. It can also be concluded that by increasing the lateral stiffness of the steel frame, base shear of the frame will obviously increase.

### 5.1.4 STORY DRIFT OF THE MODEL

On an average, 87% decrement in story drift is observed by installing cross or diagonal bracing on the model as compared to that of the model without bracing. Now, cross bracing and diagonal bracing undergo almost same drift. This is because, one of the diagonal of cross bracing remains inactive during the analysis.

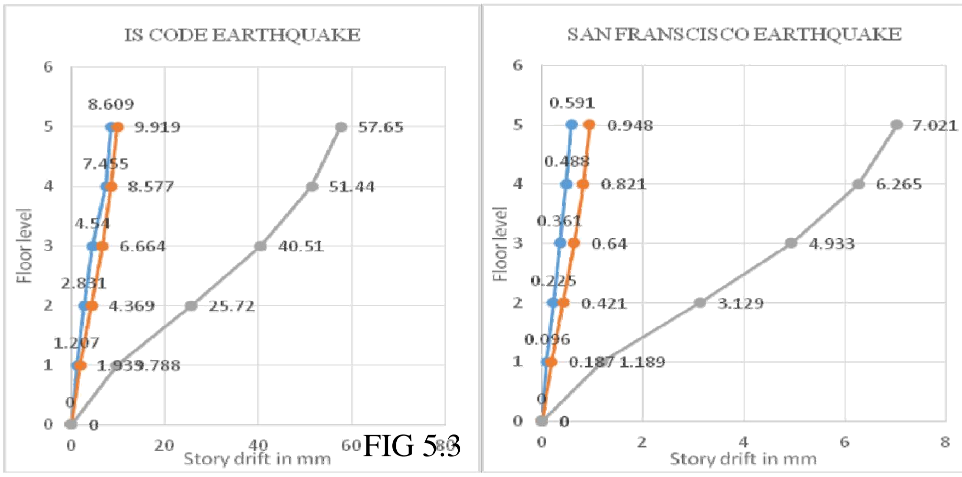
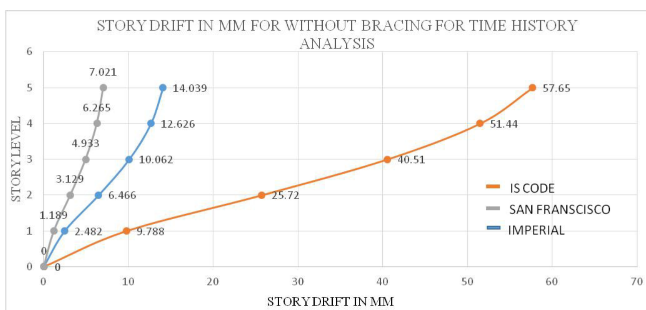


FIG 5.3

A decrease in the story drift is observed in both the analysis in upper floors. This can be infered from that the loading profile of the model. The roof load is lower as compared to the load on other floors. Hence the loading profile shows an increment till the 4th floor and then falls on the 5th floor leading to a decrement of drift on the upper floors.

On an average, 28% decrement is observed by installing cross bracing instead of diagonal bracing. Cross bracing is obviously more laterally stiffer than diagonal bracing, and hence the decrement is observed.



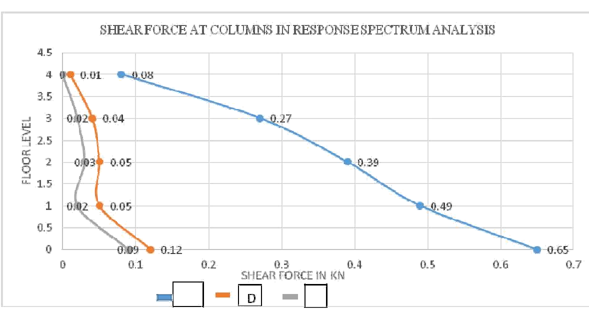

Time history is a linear analysis and hence the effect of decreased roof loading doesn't affect the final drift profile. IS code ground loading has the highest peak ground acceleration as compared to the other two earthquake loadings. Therefore, highest story drift is observed in IS Code as compared to the other two earthquake loading.

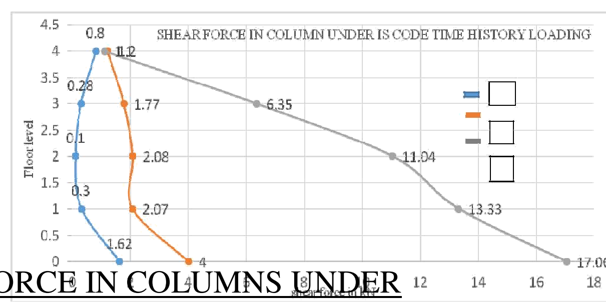
Cross bracing has the most lateral stiffness and hence in both the earthquake loading it shows least story drift.

### 5.1.5 <u>A COMPARISION OF SHEAR FORCE, BENDING MOMENT AND AXIAL FORCE AT THE CORNER COLUMNS</u>

Bracings change the stiffness of the moment resisting frames. Hence, it has a significant effect on the shear force and bending moment of columns as they take most of the lateral loading acting as a truss member i.e, they can take only tension or compression. Here, the values of shear force, bending moment and axial force of the corner columns at the 1st bay is observed and discussed.

#### 5.1.5.1 ***<u>Shear force comparision</u>***




***<u>Table 5.1.5.1</u>*** <u>A COMPARISION OF SHEAR FORCE IN COLUMNS UNDER TIME HISTORY ANALYSIS (values in kN)</u>

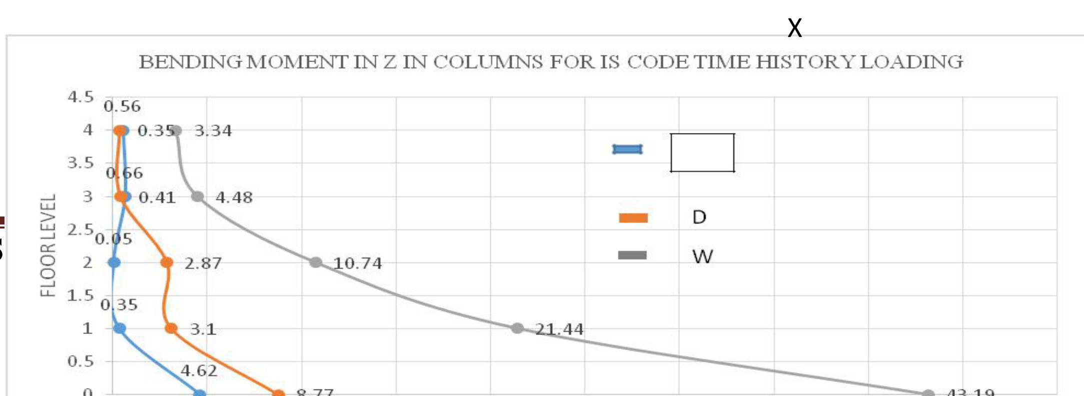
| EARTHQUAKE | IMPERIAL | | | IS CODE | | | SAN FRANSCISCO | | |
|---|---|---|---|---|---|---|---|---|---|
| BRACING | X | D | W | X | D | W | X | D | W |

| | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| COLUMN AT BASE | 1.02 | 1.67 | 3.73 | 1.62 | 4.00 | 17.06 | 0.51 | 0.70 | 1.51 |
| COLUMN@1ST FLOOR | 0.80 | 1.14 | 2.29 | 0.30 | 2.07 | 13.33 | 0.70 | 0.75 | 0.67 |
| COLUMN @2ND FLOOR | 0.87 | 1.15 | 1.61 | 0.10 | 2.08 | 11.04 | 0.72 | 0.77 | 0.37 |
| COLUMN@3RD FLOOR | 0.77 | 1.06 | 0.76 | 0.28 | 1.77 | 6.35 | 0.73 | 0.77 | 0.03 |
| COLUMN@4TH FLOOR | 0.89 | 0.89 | 0.68 | 0.8 | 1.20 | 1.10 | 0.87 | 0.93 | 0.87 |

Both the graphs, represent a lower value of shear force for cross bracing as compared to diagonal bracing and frame without bracing. For both the analyses, it can be concluded that by increasing the bracing, or by increasing the lateral stiffness shear force in columns tend to decrease.

***Table 5.1.5.2*** A COMPARISION OF BENDING MOMENT IN COLUMNS BY TIME HISTORY ANALYSIS

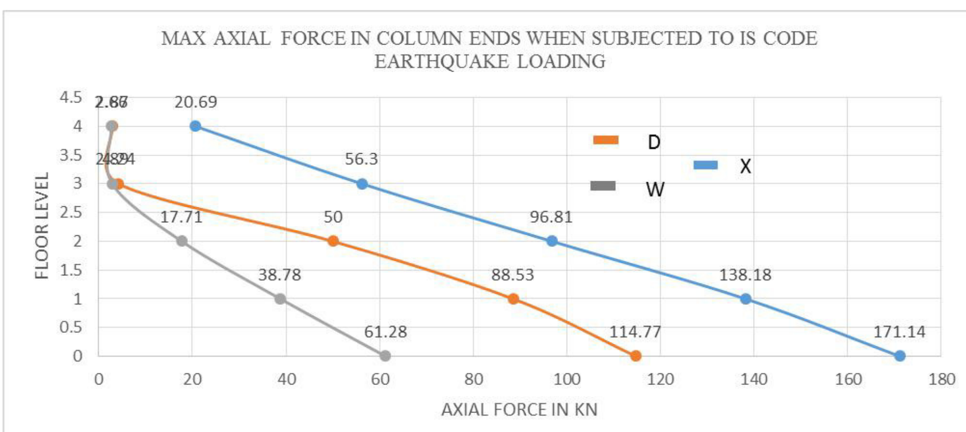
| EARTHQUAKE | IMPERIAL | | | IS CODE | | | SAN FRANSCISCO | | |
|---|---|---|---|---|---|---|---|---|---|
| BRACING TYPE | X | D | W | X | D | W | X | D | W |
| COLUMN AT BASE | 1.61 | 3.17 | 10.32 | 4.62 | 8.77 | 43.19 | 0.40 | 0.84 | 4.68 |
| COLUMN AT 1ST FLOOR | 1.16 | 1.64 | 3.69 | 0.35 | 3.10 | 21.44 | 1.00 | 1.04 | 1.15 |
| COLUMN AT 2ND FLOOR | 1.29 | 1.62 | 1.50 | 0.05 | 2.87 | 10.74 | 1.05 | 1.09 | 0.16 |
| COLUMN AT 3RD FLOOR | 1.22 | 1.40 | 0.30 | 0.66 | 0.41 | 4.48 | 1.11 | 1.14 | 0.75 |
| COLUMN AT 4TH FLOOR | 1.16 | 1.18 | 0.20 | 0.56 | 0.35 | 3.34 | 1.09 | 1.12 | 0.65 |



Both the graphs, represent a lower value of bending moment for cross bracing as compared to diagonal bracing and frame without bracing. So, by increasing the lateral stiffness of the moment resisting frame, increasing the bracing bending moment force applied at the columns tend to decrease.

5.1.5.3 ***Axial force comparision***

| EARTHQUAKE | IMPERIAL | | | IS CODE | | | SAN FRANSCISCO | | |
|---|---|---|---|---|---|---|---|---|---|
| BRACING TYPE | X | D | W | X | D | W | X | D | W |
| COLUMN AT BASE | 76.36 | 33.5 | 34.17 | 171.14 | 114.77 | 61.28 | 49.25 | 48.27 | 41.98 |
| COLUMN AT 1ST FLOOR | 60.62 | 28.7 | 23.09 | 138.18 | 88.53 | 38.78 | 40.97 | 40.50 | 34.70 |
| COLUMN AT 2ND FLOOR | 43.54 | 23.4 | 23.06 | 96.81 | 50.0 | 17.71 | 31.38 | 31.37 | 26.57 |
| COLUMN AT 3RD FLOOR | 26.07 | 16.5 | 15.86 | 56.30 | 4.24 | 2.89 | 20.28 | 20.86 | 17.41 |
| COLUMN AT 4TH FLOOR | 9.43 | 7.6 | 6.71 | 20.69 | 2.87 | 2.66 | 8.06 | 9.03 | 7.20 |



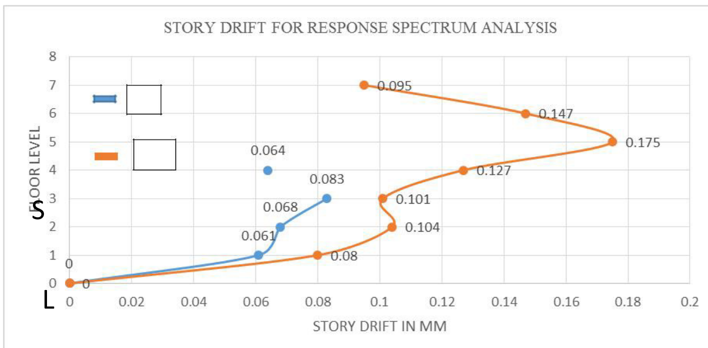

The graph represent a higher value for axial force at column ends for cross bracings followed by diagonal bracing and frame without bracing. So, with increase in bracing axial forces in the columns tend to increase.

**5.2 MODEL 2**

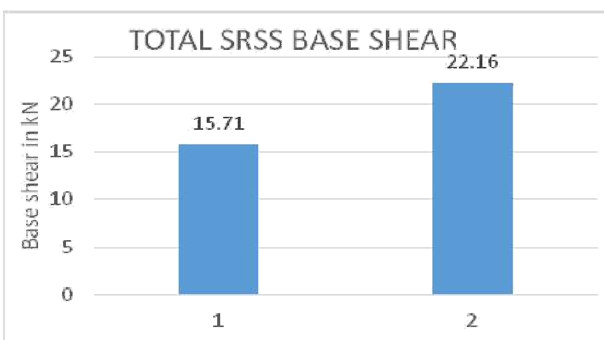
5.2.1 BASE SHEAR COMPARISION FOR BOTH THE MODELS WITH VARYING HEIGHT UNDER RESPONSE SPECTRUM ANALYSIS (SRSS METHOD)

1 represents smaller height of building and 2 represents larger height. The total base shear found out is smaller for smaller height building as compared to larger height. This can again be attributed to the fact that the larger ht. model is more stiffer than than the smaller ht. and hence the variation is expected.

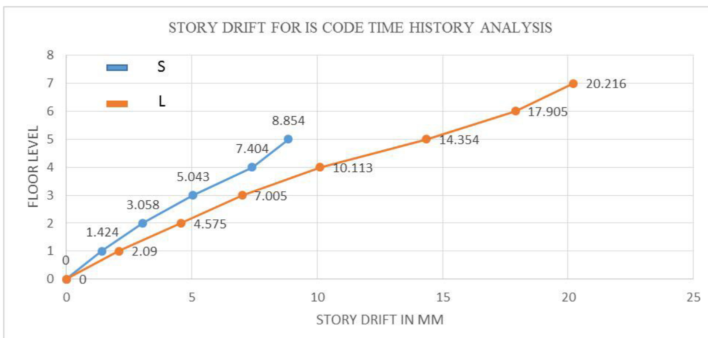
5.2.2 STORY DRIFT COMPARISION FOR RESPONSE SPECTRUM ANALYSIS




***ble 5.2.2*** STORY DRIFT FOR TIME HISTORY ANALYSIS



| MODEL | SMALLER HEIGHT | | | LARGER HEIGHT | | |
|---|---|---|---|---|---|---|
| EARTHQUAKE | IS | IMPERIAL | SAN | IS | IMP | SAN |
| BASE | 0 | 0 | 0 | 0 | 0 | 0 |
| 1 STORY | 1.424 | 0.262 | 0.146 | 2.090 | 0.093 | 0.225 |
| 2 STORY | 3.058 | 0.562 | 0.311 | 4.575 | 0.200 | 0.490 |
| 3 STORY | 5.043 | 0.928 | 0.503 | 7.005 | 1.463 | 0.745 |
| 4 STORY | 7.404 | 1.366 | 0.718 | 10.113 | 2.106 | 1.063 |
| 5 STORY | 8.854 | 1.636 | 0.718 | 14.354 | 2.977 | 0.346 |
| | | | | 17.905 | 3.710 | 0.435 |
| | | | | 20.216 | 4.172 | 0.498 |




From the graph it is evident that at the same floor level the story drift of larger height model is found to be greater than that of the smaller. This can also be attributed to the fact that the larger ht. model is more stiffer than the smaller one, and hence the variation.

## SUMMARY AND CONCLUSION

### SUMMARY

The selected frame models were analysed using response spectrum and non linear time history analysis. The 1st model was an asymmetric plan with a without braced moment resisting frame and then it was braced with diagonal bracing and cross bracing. The bracings increased the stiffness and the frequency of the frame. Cross bracing is more stiffer than diagonal bracing. Hence, for cross bracing maximum base shear was obtained as compared to diagonally braced model and model without bracing. Bracing decrease the lateral displacement of the moment resisting frame. More stiffer the frame least is the story drift. Bracings also increase the shear force and bending moment capacity of the columns. In a laterally more stiff frame, the columns are subjected to less shear force and bending moment and an increased axial force at their ends. Model 2 was a symmetric plan and a plane frame was used for analysis was performed. The frame had same V and inverted V bracing configuration but varied in height. A larger height model was more stiffer as compared to smaller one and hence had more base shear. Also at the same story, it was observed that the story drift in the larger height building was much more compared to smaller height. Larger height building is more stiffer and hence the variation. So, as the height of the model is increased, a bracing system will decrease the story drift but an increased height will increase the story drift leading to the problems like P-Δ effect.

## CONCLUSION

- Braced steel frame have more base shear than unbraced frames.
- Cross bracing undergo more base shear than diagonal bracing.
- Bracings reduce the lateral displacement of floors.
- Cross bracing undergo lesser lateral displacement than diagonal bracing.
- Cross braced stories will have more peak story shear than unbraced and diagonal braced frames.
- Axial forces in columns increases from unbraced to braced system.
- Shear forces in columns decrease from unbraced to braced system. Diagonal braced columns undergo more shear force than cross braced.
- Bending moment in column decreases from unbraced to braced system. Diagonal braced column undergo more bending moment than cross braced frame.
- Under the same bracing system and loading, system with larger height or more number of storys will have more base shear than the smaller one.
- Under the same bracing system and loading, system with larget height or more number of storys will undergo large lateral displacement on the same storys than the smaller one.

## REFERENCES


- Tremblay, R.; et al., Performance of steel structures during the 1994 Northridge earthquake, Canadian Journal of Civil Engineering, 22, 2, Apr. 1995, pages 338-360.
- Khatib, I. and Mahin, S., Dynamic inelastic behavior of chevron braced steel frames, Fifth Canadian Conference on Earthquake Engineering, Balkema, Rotterdam, 1987, pages 211-220.
- AISC (American Institute of Steel Construction), Seismic Provisions for Structural Steel Buildings, Chicago, 1997.
- AISC (American institute of Steel Construction).(1999), load and resistance factor design specification for structural steel buildings, chicago.
- A. Meher Prasad: “Response Spectrum”, Department of Civil Engineering, IIT Madras.
- David T. Finley, Ricky A. Cribbs: “Equivalent Static vs Response Spectrum – A comparision of two methods”.
- IS 1893 (Part 1):2002, “Criteria for Earthquake Resistant Design of Structures”.
- Hassan, O.F., Goel, S.C.(1991).”Modelling of bracing members and seismic behaviour of concentrically braced steel frames”.
- Tremblay, R.,Timler,P.,Bruneau,M.,and Filiatrault,A. (1995). “Performance of steel structures during 17 january,1994 Northridge earthquake.”