COMPARATIVE STUDY ON VARIOUS TYPES OF BRACING SYSTEMS IN HIGH RISE STEEL STRUCTURE CONSIDERING CONSTANT STEEL PLATE SHEAR WALL AS DUAL SYSTEM FOR LATERAL LOADS

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ABSTRACT

Dual system is generally preferable for high-rise steel structures to resist lateral loads as wind and earthquake loads. In dual system, if a constant steel plate shear wall is used in various types of bracing systems, which one is more economical and proper is the aim of this study. In this study, four types of bracing systems are chosen: X-bracing, Diagonal bracing, Inverted-chevron CBF, Inverted- chevron EBF. In each case, 40story steel structure are modeled, analyzed and designed by using ETABS2016. These are compared in different aspects such as maximum story displacement under seismic and wind loading and also from economical viewpoint by calculating the weight of the structure. It is found that Inverted- chevron EBF has a minimum top-story displacement. To optimize the amount of steel consumed and also to obtain a lightweight structure, X-bracing system is economical and flexible in steel frames.

Keywords:*Bracing systems, Dual system, High rise steel structures, Steel plate shear wall, Lateral Loads.*

1. INTRODUCTION

In present era, steel structure are most common choice for commercial building construction around the world. For high rise steel structure, it is the main concern to resist lateral loads in economical viewpoint. Generally dual system are used for high rise steel structure due to possibility to use common and lighter section of braces and beams as well as steel plate shear wall thickness. Concentric and eccentric braced frames are usually used and each type have specific characteristics and design requirements. In this study, various types of bracing including X, Diagonal, Inverted-chevron concentric and eccentric three dimensional braced frames with constant thickness of steel plate shear wall are modelled, analyzed and designed by using ETABS2016 and find out the most economical structure with flexibility. Though the process use to investigate does not met all the requirements but this analysis and design provides a good comparison between the chosen bracing types. The lateral loads are derived from BNBC1993, moreover the sections are the steel I/wide flange for beams and columns and also steel pipe (HSS) for brace, which are checked and designed by AISC provisions. Buildings were located in typical seismic zone and were analyzed by static equivalent method. At last, all of these are compared in different aspects and conclusions are obtained, that would be helpful for the designers.

2.METHODOLOGY

In this study, a rectangular grid of 44.19m with 6 bay along x-direction and 20.46m with 2 bay along y-direction is used (shown in figure 1). Three types of steel concentric braced frames as X, Diagonal, Inverted-chevron and one eccentric braced frames as Inverted-chevron (in which link beam indicated by e is assumed as ½ *l* and h is assumed as ½ *h*, where *l* is the length of beam and *h* is each story column height) with height of 40 stories shown in figure 2. In all these models the effect of moment frames are avoided by considering simple connections between beams and columns. P-delta effect is considered for linear static analysis. The frame responses are identified by using ETABS2016. According to AISC-LRFD design provision code, frame members are designed for the gravity loads and lateral forces. At design procedure, it is attempted to optimize the required sections to their minimum possible sizes by checking drift limitations to gain the desired strength and stiffness. So for beams and columns steel I/wide flange sections and for brace steel pipe (HSS) sections are assigned.
To prevent the undesirable phenomenon, the width to thickness ratios of the thin flange and the web plates are limited by AISC-LRFD code. Since the flange is continuously connected to the web, the shape is compact. All beams and columns are checked and resized defined by AISC code with the following equations [3]:

$$\frac{b_f}{2t_f} \leq 0.38\sqrt{\frac{E}{F_y}} \qquad (1)$$

To prevent local flange buckling, the limitation of width to thickness ration is shown in equation (1).

$$\frac{h}{t_w} \leq 3.76\sqrt{\frac{E}{F_y}} \qquad (2)$$

To prevent vertical web buckling, the limitation of height to thickness ration is shown in equation (2).
Since the shape is compact, so no need to check flexural FLB (flange local buckling) and flexural WLB (web local buckling).

where h, b_f, t_f, t_w, E and F_y are outside height of cross section of beams and columns, flange width, flange thickness, web thickness, modulus of elasticity for steel (29,000 *ksi*) and yield stress of steel (50 *ksi*). the type of steel is assumed as ST37.The thickness of steel plate shear wall (constant for each model) is assumed as 300*mm*. Selected types of sections for beam, column and brace are mentioned in Table 1 and 2.

Table 1: Specifications of steel I/wide flange sections

Name	**Depth, d *(mm)***	**Flange width, b_f*(mm)***	**Flange thickness,t_f*(mm)***	**Fillet radius *(mm)***	**Web thickness,t_w*(mm)***
W10X12	250.7	100.6	5.3	7.6	4.8
W18X311	566.4	304.8	69.6	12.7	38.6
W21X182	576.6	317.5	37.6	12.7	21.1
W24X370	711.2	348	69.1	12.7	38.6
W27X539	825.5	388.6	89.9	20.1	50
W30X391	843.3	396.2	62	20.1	34.5
W33X387	914.4	411.5	57.9	20.1	32
W36X652	1043.9	447	89.9	24.1	50
W40X593	1092.2	424.2	82	30	45.5
W44X335	1117.6	403.9	45	20.1	26.2

Table 2: Specifications of steel pipe (HSS) sections

Name	Outer diameter *mm*	Wall thickness *mm*
HSS10.750X0.500	73	6.4
HSS14X0.625	127	12.7
HSS18X0.500	174.6	12.7
HSS2.875X0.250	219.1	15.9
HSS20X0.500	273.1	12.7
HSS5X0.500	355.6	15.9
HSS6.875X0.500	457.2	12.7
HSS8.625X0.625	508	12.7

For compact section with the width to thickness ratio of flange, the following requirements must be meet for link beams in eccentric braced frames as [4]:

$$\frac{b_f}{2t_f} \leq \frac{435}{\sqrt{F_y}} \tag{3}$$

$$M_p = Z.F_y \tag{4}$$

$$V_p = 0.55F_y.d.t_w \tag{5}$$

In these equations, d is the depth of web, Z is the plastic modulus, M_p is the plastic bending moment of links, V_p is the plastic shear and Q is the expected resistance of links.

When $e \leq 1.6\,\frac{M_p}{V_p}$ $\qquad Q = V_p$ (6)

When $e \geq 2.6\,\frac{M_p}{V_p}$ $\quad\quad Q = 2\,\frac{M_p}{e}$ (7)

In mass source, Dead and Live load multiplier is taken as 1 and 0.5 respectively. Total thickness of deck is taken 0.175*m* with deck depth 0.075*m*.

3. LOADING

In this study, selected loads are shown in Table 3. According to BNBC1993 sustained wind pressure and design wind pressure is calculated by the following equaions.

$$q_z = Cc.C_I.C_z.V_b^2 \quad (8)$$

$$P_Z = C_G.C_p.q_z \quad (9)$$

$$F = \sum P_Z.A_Z \quad (10)$$

where q_z is sustained wind pressure at height *z*, $C_I$isstructure importance coefficient=1, C_c is velocity to pressure conversion coefficient=47.2x10^{-6}, $C_z$is combined height and exposure coefficient, $V_b$is basic wind speed in km/h=210, P_z is design wind pressure at height z, C_g is gust coefficient, C_p is pressure coefficient, *F* is wind force on primary framingsystems acting normal to a surface, A_Z is area of the building surface. By using equation (8) (9) (10), design wind force is calculated manually and then inputting data in ETABS2016 by using user loads.

According to BNBC1993, the approximate period, T_a is calculated by the equation (11).

$$T_a = 0.083\,(h_n)^{0.75} \text{ [for steel structure]} \quad (11)$$

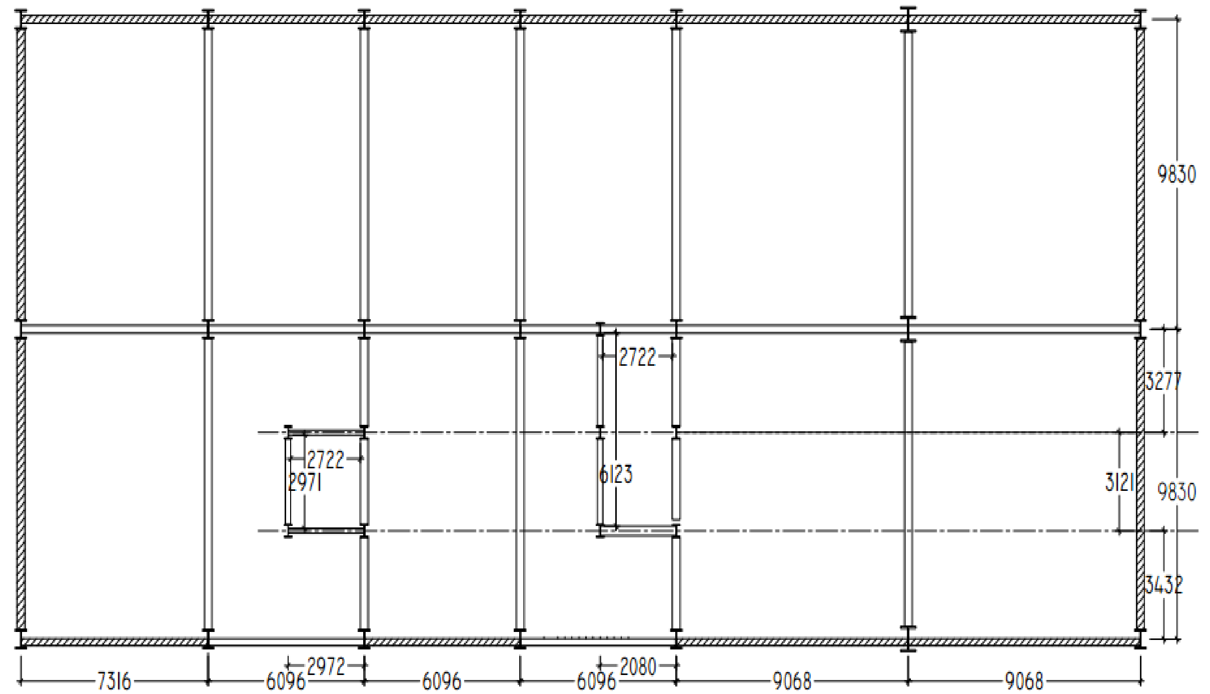
in accordance with equivalent static analysis method [5] baseshear is obtained by the equation (12).

$$V_b = A_h W \quad (12)$$

$$A_h = ZIS_a/2Rg \quad (13)$$

$$Q_i = V_b.\frac{Wi.hi}{\sum Wi.hi} \quad (14)$$

The base shear, v_b is distributed over the height of the building in accordance with equation given in (14). Where,h_n is total height of the steel structure in meters, $a_h$is the design horizontal spectrum



value, s_a/g is spectral acceleration coefficient=1, r is response reduction factor (8 for cbf and 10 for ebf), i is importance factor=1, w is seismic weight of the building (based on specified mass), q_i is portion of base shear applied to ith story level, v_b is base shear, $w_i$is weight of ith story level (based on specified mass), h_i is ith story height distance from base of building to story level, seismic zone factor, z=0.15. Mass source is assumed as 50% for live load, total dead load and others.

Figure 1: Plan view with bracing location

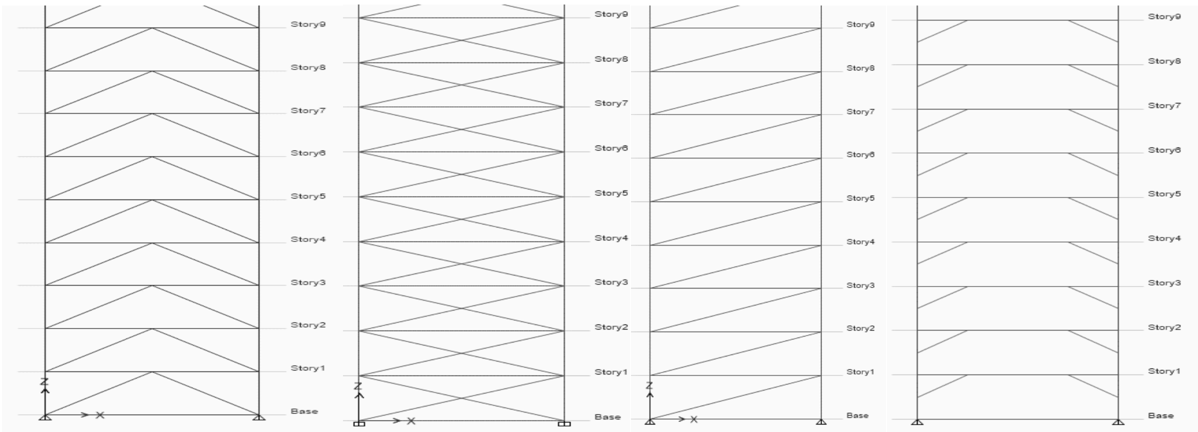


Figure 2: Typical geometry of bracing systems

Table 3: Selected load patterns

Name	Type	Self-Weight Multiplier	Auto Load	Applied Load
Dead load	Dead	1	----	-----
Partition wall	Super Dead	0	----	1.5*(kn/m^2)*
Live load	Live	0	----	4*(kn/m^2)*
Floor finish	Super Dead	0	----	1.5*(kn/m^2)*
Stair live	Live	0	----	5*(kn/m^2)*
Wall load	Super Dead	0	----	0.678*(kn/m)*
Wind along X	Wind	0	User Loads	----
Wind along Y	Wind	0	User Loads	----
Earthquake along X	Seismic	0	IS1893 2002	----
Earthquake along Y	Seismic	0	IS1893 2002	----

By using Auto Load (IS1993-2002) [IndianStandard, 1893] in ETABS2016 base shear for selected bracing systems are shown in Table 4.

Table 4: Calculated base shear

Bracing types	Period Used *(sec)*	W *(kN)*	V_b *(kN)*
X-brace	3.07	896648.79	3723.87
Diagonal brace	3.07	1024894.00	4256.49
Inverted-Chevron CBF	3.07	1009307.00	4191.75
Inverted-Chevron EBF	3.07	1020368.00	3390.15

According to BNBC1993, load combinations for steel structure are as $1.4D$, $1.2D+0.5L_r$, $1.2D+1.6L_r$, $1.2D+0.5L_r+1.3W$, $1.2D+1.5E$, $0.9D+(1.3W$ or $1.5E)$. Where D is all dead load, $L_r$is floor live load, W is wind load, E is earthquake load.

4. ANALYSIS, DESIGN AND RESULTS

By using load envelope, on which load combination case gives maximum value for displacement are identified. Figure 3 and Figure 4 show the maximum story displacement versus story number along X and Y axis induced by the lateral loads in different bracing systems. It indicates the displacement which resisted by each bracing design members. From Figure 3 and Figure 4, it is understood that Inverted- chevron EBF resisted less than others. Abrupt change occurred in story-41 (staircase roof) due to single frame members without continuation.Table 5 shows brace material schedule comparison. Total amount of steel brace weight required forInverted-Chevron EBF is less than other types of bracing and sequentially as Inverted-Chevron EBF < Diagonal Braced < Inverted-Chevron CBF < X-Braced.

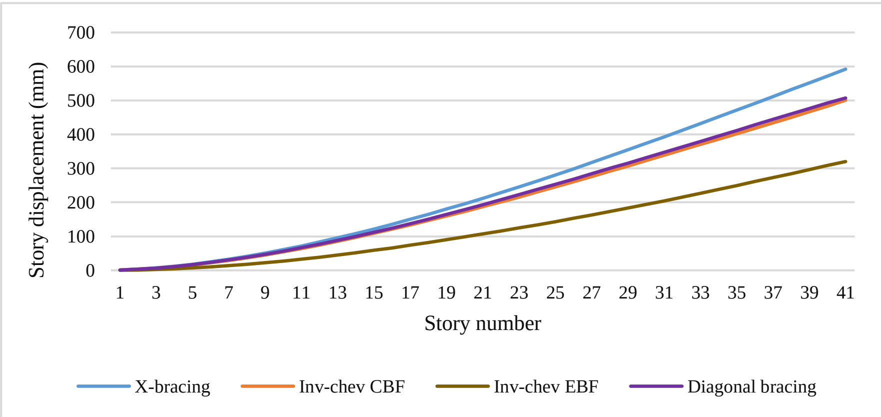


Figure 3: Maximum story displacement along Y axis vs story number

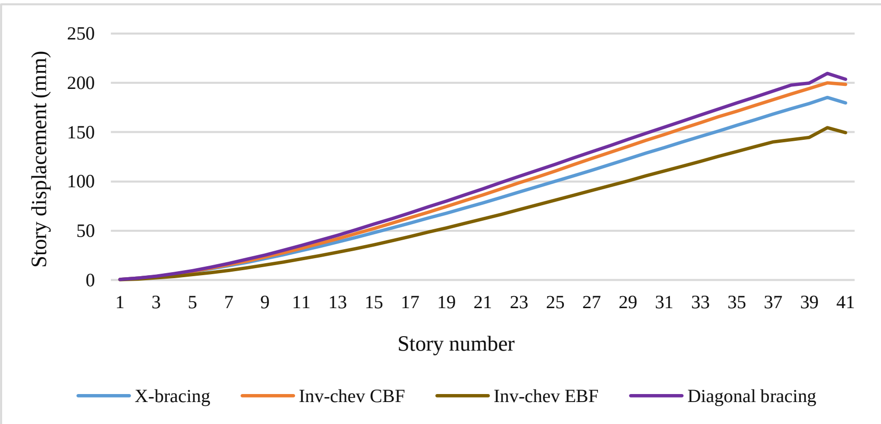


Figure 4: Maximum story displacement along X axis vs story number

Table 5:Steel brace comparison

Type of Bracing	**Total Weight *(tonne)***
X-Braced	311,246.95
Inverted-Chevron CBF	256,802.90
Inverted-Chevron EBF	155,194.91
Diagonal braced	228,161.15

Table 6 shows the steel beam (including secondary beam) schedule comparison. Total amount of steel beam weight required for Inverted-Chevron EBF is less than other types ofbracing and sequentially as Inverted-Chevron EBF< Diagonal Braced < Inverted-Chevron CBF < X-Braced.Table 7 shows the steel column schedule comparison. Total amount of steel column weight required for X-Braced is less
than other types of bracing and sequentially as X-Braced< Inverted-Chevron CBF < Inverted-Chevron EBF < Diagonal Braced.

Table 6:Steel beam comparison

Type of Bracing	**Total Weight *(tonne)***
X-Braced	2,170,245.79
Inverted-Chevron CBF	1,902,649.60
Inverted-Chevron EBF	1,681,795.00
Diagonal braced	1,880,765.99

Table 7:Steel column comparison

Type of Bracing	**Total Weight *(tonne)***
X-Braced	1,028,945.73
Inverted-Chevron CBF	1,594,467.86
Inverted-Chevron EBF	1,827,938.67
Diagonal Braced	2,009,553.59

Table 8:Whole steel structure comparison

Type of Bracing	Total Weight *(tonne)*
X-Braced	3,583,889.59
Inverted-Chevron CBF	3,827,371.48
Inverted-Chevron EBF	3,738,379.70
Diagonal braced	4,191,931.85

Table 8 shows whole steel structure schedule comparison.Total weight of the model steel structure for X-Braced is less than other types of bracing and sequentially as X-Braced < Inverted-Chevron CBF < Inverted-Chevron EBF < Diagonal Braced.

5. CONCLUSION

Inverted-chevron EBF model are laterally displaced less than others bracing model shown in Figure 3 and Figure 4. But total rigidity of Inverted-chevron EBF is less than other systems as compensate for the lack of stiffness in bracing members, for these reasons stronger sections (heavier weight) are required for beams and columns in the braced spans that are shown in Table6 and Table 7. Though the total amount of steel brace is maximum for X-braced also for steel beam, the total amount of material required for the whole structure is minimum which indicates that X-braced is economical. On the other way X-braced structure also resist maximum story displacement which indicates its flexibility and stiffness. From the above discussion, it can be concluded that applying X-Bracedbracing system is economical and proper for the steel braced frames.

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