Behaviour of Different Bracing Systems in High Rise 2-D Steel Buildings under Wind Loadings

Dhanaraj M. Patil[1*] and Keshav K. Sangle[2]

[1,2]*Structural Engineering Department, VJTI, Mumbai 400019, India*
E-mal: [1]patil.dhanaraj@gmail.com, [2]dhanaraj_civil@yahoo.co.in

Abstract—*In this study, the behaviour of different bracing systems in high rise 2-D steel buildings under the application of dynamic wind load is investigated. For this purpose, a two dimensional dynamic wind analysis were carried out to on different braced high rise 2- D steel building frames of 10, 15, 20, 25, 30, and 35 storeys to capture the structural response. This research is carried out using five structural configurations of braced frames: moment resisting frames (MRF), chevron braced frames (CBF), V-braced frames (VBF), X-braced frames (XBF), and zipper braced frames (ZBF). Dynamic wind analysis is carried on total 30 high rise 2-D steel buildings using gust factor method. It is instructive to note that significant changes in structural behaviour of MRF high rise 2-D steel buildings is observed when compared with braced high rise 2-D steel buildings. Parameters such as the type of bracing and height of buildings significantly affect the structural performance of high rise buildings. In this study structural performance of different structural systems is compared on the basis of the fundamental time period, storey displacement, top storey displacement, and inter-storey drift ratio. It is observed that the CBF and ZBF are observed to be more efficient than other structural systems in high rise 2-D steel buildings.*

1. INTRODUCTION

In design of high-rise structures, strength and stiffness is an important criteria to control which measured in terms of inter storey drift and top storey displacements. In high rise buildings rigidity and the stability requirements become more important as the building height increases. Hence, high rise buildings must meet strength, stiffness, and stability requirements [1]. The moment resisting and concentrically braced frames have been widely used to resist lateral loadings. The moment resisting frames possess considerable energy dissipation characteristics, but it has limited stiffness. On the other hand concentric braced frames are excellent from a strength and stiffness considerations, but because of buckling of the diagonal brace its ductility is limited. Rigid or semi-rigid frames are not efficient for high rise buildings because the deflection produced by the bending of columns and girders causes the building drift to be too large. In such case the simple and economical way to increase the structure's lateral stiffness is by introducing bracings in the building.

Different concentric bracing such as typical diagonal bracing, X-bracing, chevron bracing and V-bracing configurations are excellent from a strength and stiffness considerations when lateral loads are caused by the wind. However, represent poor inelastic behaviour in seismic regions. To promote energy dissipation capability of a steel framed structure, new bracing is proposed, where brace is placed eccentrically to the beam-to-column joint called eccentrically braced frame system [2, 3]. In order to overcome the deficiencies with eccentrically braced frame, Aristazabal-Ochoa has proposed disposable knee braced frame [4]. The knee braced frame re-examined to propose modifications to control buckling of diagonal braces [5-7]. In eccentrically braced frame and knee braced frame stiffness is retained by eccentricity while ductility achieved through shear yielding of a short segment of beam/brace. As in eccentric braced frame yielding of main structural member and in knee braced frame yielding of knee element takes place so it difficult to retrofit [2-7]. Inverted V-braced frames (chevron frames) form a vertical truss system to resist lateral forces such as those produced by wind and earthquakes. Due to lateral loadings unbalanced vertical force acted on the frame, which results in very strong beams, much stronger than that would be required for ordinary loads [8]. This unbalanced force can be mitigated by adding zipper brace element. This frame with zipper called zipper braced frame [9]. Problems associated with the design of braced buildings have been investigated for cyclic loadings [10-13] and Problems associated with regions of strong wind, moderate seismicity and long period ground motion for the design of concentrically braced high rise buildings have been investigated [14-16].

In this study, an extensive analytical investigation of the behaviour of different braced high rise 2-D steel building has been undertaken by wind analysis using dynamic gust factor method. Most of the earlier studies focused on low and mid rise 2-D steel buildings. The seismic behaviour of differently braced 2-D and 3-D steel buildings is studied for different storey high rise buildings [17-18]. The overall aim is to assess the structural performance of five structural configurations: MRF, CBF, VBF, XBF and ZBF in high rise steel buildings of 10, 15, 20, 25, 30 and 35 storeys. Finally, the behaviour is compared based on parameters such as fundamental period of vibration, storey displacement, and inter-storey drift ratio.

2. MODELLING OF EXAMPLE HIGH RISE 2-D STEEL BUILDINGS

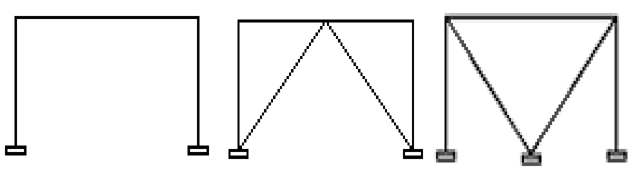
Modelling of example steel buildings illustrates the different structural configuration of high rise steel building. Fig. 1 illustrates the different structural systems MRF, CBF, VBF, XBF and ZBF. Example building of two bays of 6m span and a central bay of 4 m span is used for study as shown in Fig. 2 [17]. In two dimensional models, outer frame is considered for analysis as shown in Fig. 2. In braced frames, outer frames are modelled as CBF, VBF, XBF and ZBF configuration as in Fig. 1.

2.1 Description of example 2-D steel buildings

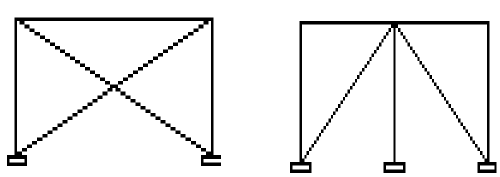
Total 30 high rise 2-D steel buildings have been used for analytical investigation. Patil et al investigated the same example buildings subjected to seismic loadings for 2-D and 3-D analysis. High rise steel buildings of different heights 10, 15, 20, 25, 30 and 35 storeys, with the plan and outer frame configuration as shown in Fig.1 and Fig. 2, are used for investigation [17-18]. The five different structural configurations of braced buildings used for study (Fig. 1).

2.2 Loadings considered for 2-D steel buildings

Example buildings are designed using IS1893 (Part-I): 2002, IS875 (Part 2): 1987, IS 875 (Part 3): 1987, and IS800:2007 [19-22]. In the design, structural steel sections of nominal yield strength of 345 MPa are used for beams, braces, and columns. The imposed load is considered as 4 kN/m^2 and floor finishes, partitions loads both are assumed to be 2 kN/m^2 [20].



a) MRF b) CBF c) VBF



d) XBF e) ZBF

Fig. 1: Structural configuration of different braced systems in elevation [17].

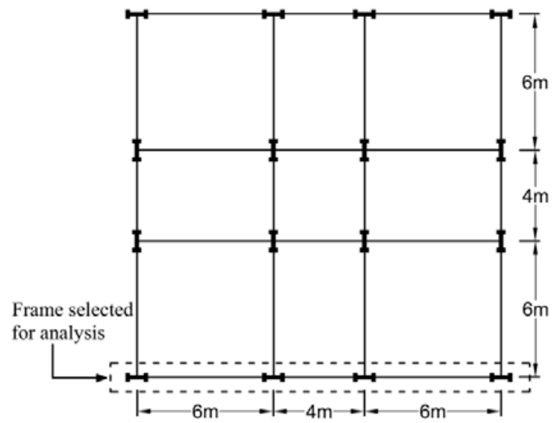


Fig. 2: Plan of the example buildings [17]

3. WIND ANALYSIS: GUST FACTOR METHOD

Gust factor method of calculating wind forces on structure is used in this study for wind analysis. Gust factor method is considered the dynamic effect of the wind on the structure [21]. These dynamic wind forces (F_z) given by Eq. 3 are applied to study example buildings by using SAP2000v16 [23]. Dynamic wind forces calculated by the gust factor method are applied to 2-D and 3-D steel high rise buildings of different configuration to find out the structural response. These dynamic wind forces are calculated as follows

Design Wind Speed (V_z): [21]

$$V_z = V_b K_1 K_2 K_3 \tag{1}$$

Where Basic Wind Speed (V_b) - 50 m/s

Probability factor/risk coefficient (K_1) - 1.08

Topography factor (K_3) – 1

Terrain Category- 2

Terrain, height and structure size factor ($\overline{K_2}$) – can be calculated as per clause 8.2 and 8.2.1 of IS 875 (Part 3): 1987 for different buildings [21].

Hourly Mean Wind ($\bar{V}z$): [21]

$$\bar{V}z = V_b K_1 K_2 K_3 \tag{2}$$

Along wind load (Fz) on a structure on a strip area (Ae) at any height (z) is given by:

$$Fz = Ct\ Ae\ \bar{P}z\ G \tag{3}$$

The force coefficient for the building, (Ct) – can be calculated as per clause 6.3.2 and Fig. 4 of IS 875 (Part 3): 1987 for different buildings [20].

Design pressure ($\bar{P}z$)

$$\bar{P}z = 0.6\ \bar{V}z^{\,2} (N/m^2) \tag{4}$$

Gust factor (G):

$$G = 1 + g_f r \sqrt{[B\,(1+\phi)^2 + \frac{SE}{\beta}]} \qquad (5)$$

Where,

g_{fr} - can be calculated as per clause 8.2 and Fig. 8 of IS875 (Part 3): 1987.

B - can be calculated as per clause 8.2 and Fig. 9 of IS875 (Part 3): 1987.

S - can be calculated as per clause 8.2 and Fig. 10 of IS875 (Part 3): 1987.

E - can be calculated as per clause 8.2 and Fig. 11 of IS875 (Part 3): 1987.

β - can be calculated as per clause 8.2 and Table 34 of IS875 (Part 3): 1987.

$\frac{SE}{\beta}$ - measure of the resonant component of the fluctuating wind load

$\emptyset = \frac{g_f r \sqrt{B}}{4}$ and is to be accounted only for buildings less than 75 m high in terrain Category 4 and for buildings less than 25 m high in terrain Category 3, and is to be taken as zero in all other cases [21].

4. RESULTS AND DISCUSSIONS

4.1 Fundamental Time period

In this investigation, the natural periods of vibration, evaluated from empirical equations given in IS 1893:2002 for steel buildings with infill are shown in Table 1, similar to Patil et al [17]. The fundamental period of vibration of the example buildings is found out by using modal analysis using eigenvalue method. The fundamental period is the first elastic mode of the longest time period of vibration. Obtained fundamental period of vibration from eigenvalue analysis of 2-D buildings are reported in Table 2 which are similar to Patil et al [17].

Table 1: The natural period of vibration (s) by empirical expression [17, 18]

Buildings storey	10	15	20	25	30	35
Period (s)	0.6750	1.0125	1.3500	1.6875	2.0250	2.3625

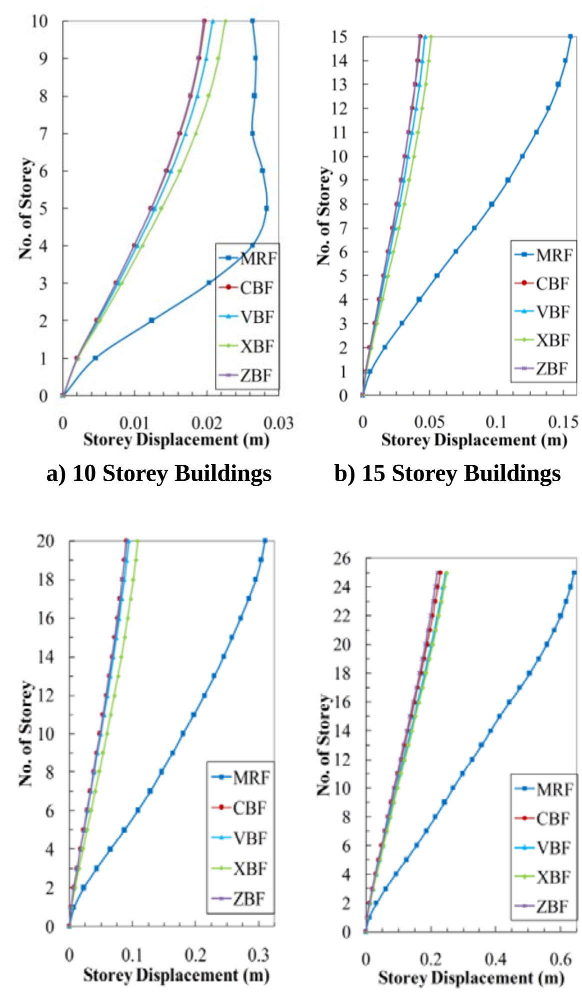
Table 2: Fundamental period of vibration(s) by modal analysis of 2-D building frames [17]

Buildings storey	MRF	CBF	VBF	XBF	ZBF
10	1.1447	0.6501	0.6670	0.6935	0.6502
15	1.6319	0.9011	0.9335	0.9766	0.8999
20	2.1913	1.2532	1.2927	1.3474	1.2525
25	2.9477	1.8303	1.8830	1.8901	1.8251
30	3.7718	2.3232	2.3899	2.4774	2.3152
35	5.3050	3.3367	3.4242	3.6635	3.3187

It is observed from Table 1 and Table 2 that the fundamental periods obtained from eigenvalue analysis are 70%, 61%, 62%, 86%, and 124% higher than the values derived by codal empirical equations for MRF of 10, 15, 20, 25, 30, and 35 storey 2-D buildings, respectively [17]. It is seen that the fundamental period obtained from the eigenvalue analysis is nearly close to that of empirical equations for braced buildings. The significant difference is seen in periods of vibration obtained from codal empirical and modal analysis for MRF buildings. It is observed that the seismic codes tend to underestimate the period of vibration.

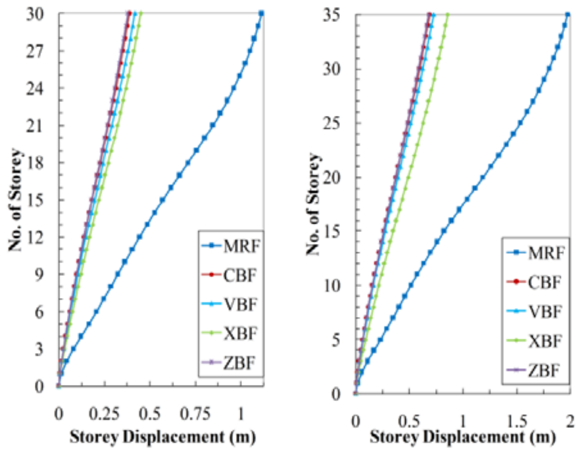
4.2 Storey displacement

The storey displacement discussed herein on the basis of the effect of different braced buildings on the structural response of high rise buildings. The storey displacements of different high rise 2-D steel buildings are illustrated in Fig. 3 (a-f). However, following observations could be made from the overall interpretation of storey displacements. Table 3 depicts the top storey displacement of different structural systems for all storey 2-D high rise steel buildings.



a) 10 Storey Buildings **b) 15 Storey Buildings**

c) 20 Storey Buildings **d) 25 Storey Buildings**



e) 30 Storey Buildings **f) 35 Storey Buildings**

Fig. 3: Storey displacement of differently braced 2-D steel buildings

It is revealed from Fig. 3 (a-f) that storey displacement of CBF and ZBF systems show a lower value than other systems for all storey 2-D buildings. It is noted that storey displacement are significantly reduced in differently braced 2-D buildings than MRF building. It is seen that VBF also shows nearly the same values of storey displacement for 20, 25, 30 and 35 storey 2-D buildings. XBF shows considerable differences in storey displacement than CBF, VBF and ZBF of 2-D buildings. Strength and stiffness of MRF buildings are increased due to the addition of braces. MRF buildings are more flexible than other systems as the fundamental period of all MRF buildings is higher, than other systems (Table 2). The MRF buildings, as depicted in Fig. 3 (a-f), show higher storey displacements than other systems. CBF, VBF, and ZBF show lesser storey displacement of almost all storey buildings. It is depicted from study that stiffness of high rise buildings increases due to addition of braces in MRF buildings. Storey displacement of different braced buildings is different represent particular trend in different height buildings to indicate the most suitable bracing system.

Table 3: Top storey displacement (mm) of 2-D building frames

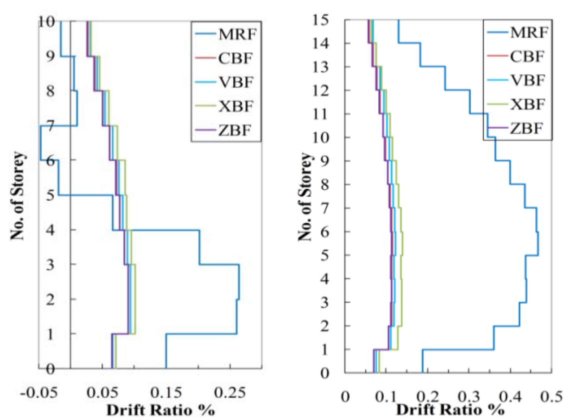
Buildings Storey	MRF	CBF	VBF	XBF	ZBF
10	26	20	21	23	20
15	155	43	46	51	42
20	310	90	94	108	90
25	641	228	245	249	221
30	1112	388	417	449	379
35	1975	691	726	858	680

It is observed from Table 3 that top storey displacement is significantly reduced due to the addition of braces in 2-D steel buildings. It is depicted from table 4 that ZBF and CBF show less top storey displacement than VBF, XBF and MRF for all storey 2-D buildings. It is seen that 23%, 72%, 71%, 64%, 65% and 65% reduction in top storey displacement of 10, 15, 20, 25, 30, and 35 storeys in CBF buildings than MRF 2-D buildings respectively.

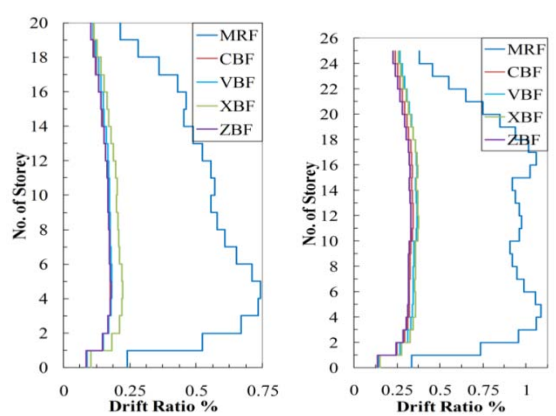
4.3 Inter-storey drift ratios

The inter-storey drift ratios discussed herein on the basis of the effect of different braced buildings on the structural response. The inter-storey drift ratios of high rise 2-D steel buildings corresponding to wind load are illustrated in Fig. 4 (a-f). However, following observations could be made from the overall interpretation of inter-storey drift ratios.

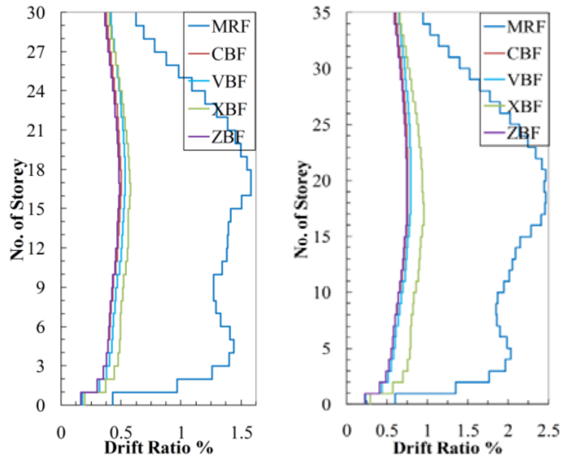
It is observed from Fig. 4 (a-f) that the distribution of inter-storey drift ratio over the building height is non-uniform. Inter-storey drift ratios of MRF 2-D buildings are higher than CBF, VBF, XBF, and ZBF braced 2-D buildings. MRF buildings show three time higher inter-storey drift ratio than braced buildings. MRF buildings are more flexible than braced systems as it show higher inter-storey drift ratio. It is highlighted that as the number of storeys of building increases, inter-storey drift ratio increases. It is seen from Fig. 4 (a-f) that inter-storey drift ratio of all buildings increases upto 1/3 to 2/3 of building height then it is decreases. Higher percentage of inter-storey drift ratio is observed in the middle and lower height of buildings. CBF, VBF and ZBF show less inter-storey drift ratio with few exceptions.



a) 10 Storey Buildings **b) 15 Storey Buildings**



c) 20 Storey Buildings **d) 25 Storey Buildings**



e) 20 Storey Buildings **f) 25 Storey Buildings**

Fig. 4: Inter-storey drift ratio of differently braced 2-D steel buildings

5. CONCLUSION

In this study an attempt is made to assess the performance of different braced systems under wind loadings. For this purpose, differently braced 2-D is studied for different storey height; and the performance is compared. An extensive analytical investigation of the different braced frames has been undertaken by wind analysis using the gust factor method.

The conclusions of this study can be highlighted as follows.

1. MRF high rise 2-D buildings show higher storey displacement and inter-storey drift ratios representing that MRF building are more flexible than CBF, VBF, XBF and ZBF systems.
2. Behaviour of CBF and ZBF under dynamic wind load is nearly similar in terms of storey displacement and inter-storey drift ratio for different heights 2-D high rise buildings.
3. CBF and ZBF show lower top storey displacements than other systems in all storeys 2-D high rise steel buildings under wind loads.
4. Storey displacement and inter-storey drift ratio is significantly reduced for ZBF and CBF than other systems representing these systems are stiffer than other systems. VBF also shows nearly similar storey displacement and inter-storey drift ratio.
5. In high rise 2-d steel buildings, ZBF, VBF and CBF are more efficient in terms of different parameters such as fundamental time period, storey displacement, and inter-storey drift ratio than XBF and MRF.
6. Strength, stiffness and stability requirements are main criteria to control in high rise 2-D steel buildings, so from this study one can chose a bracing system so as to increase strength, stiffness and stability of the MRF high rise steel buildings.

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