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Research Article

A Parametric Study on the Design of Multi-Storey Steel Buildings

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ABSTRACT

Buildings must withstand external loads by presenting limited displacements in order to ensure safety, stability, and functionality. This study focuses on the horizontal load bearing systems of multi-story steel buildings in consideration of lateral displacements. Seven different horizontal load-bearing systems were designed for an eleven-story steel building located in Bursa. The analysis of structures was performed applying same seismic parameter and capacity limits for all. The design procedure was conducted in accordance with current regulations of "Design, Calculation and Construction of Steel Structures-2016" and "Turkish Building Earthquake Code-2018". Fundamental model designed as moment resistant frames in both principal directions. Subsequently, vertical concentric braces were then added in six different configurations, varying in stiffness and layout, to explain their effect on lateral displacements. All models were designed as high ductile according to linear capacity-based analysis method. In this way, progress in horizontal displacement performances of the building under seismic and wind loads was examined using different vertical bracing systems and locations.

Introduction

Analysis methods and design criteria of structural systems evolve and improve continually. Despite of rapid change on design criteria, the fundamental goal of design remains same as to construct safe shelters for humanity. Throughout their lifespan, buildings are subjected to a various of external loads. These loads can be classified into two main categories those that act in vertical direction and those that act horizontally. Therefore, building must withstand not only vertical forces induced by gravity and variable loads but also lateral forces due to wind or seismicity. Lateral forces are quite critical for buildings because they can induce secondary stresses and lateral deformations, potentially leading to instability and collapse.

Earthquakes occur when stress accumulated along geological faults exceeds the strength of rocks, causing a sudden rupture and release of energy in the form of seismic waves. Seismic waves induce ground motion and energy carried by them is absorbed by buildings via deformations due to displacements. Excessive displacements due to earthquake causes uneven stresses, compromising structural integrity and potentially causing premature failure that prevents the building from reaching its intended performance level. Deformations and displacements should be limited taking account of their primary effects on strength and secondary effects on stability of structure. While excessive lateral displacement can compromise structural integrity under ultimate conditions. It can also cause several problems under service loads, from failure of mechanical equipment, to loss of function of non-structural elements, to negative effects on human psychology.

Almost every design code including Turkish Building Earthquake Code-2018 (TBDY-2018) limits lateral displacements and joint rotations in structures proposing some inter story drift limits [1]. But there is no precise value to limit top displacement of the structure under wind loads. As Design, Calculation and Construction of Steel Structures-2016 (ÇYTHYEDY-2016) states that in chapter 15.3 that in order to prevent damage to non-structural elements of the building, under wind loads, and to prevent their function from being adversely affected, horizontal displacements must be limited. The horizontal displacement limits of the structure may vary depending on the type of building and the type of cladding and partitions [2]. As indicated by Lawrence G. Griffis once damage thresholds are determined from tests or estimated, it remains only to establish an appropriate limit for different building components [3].

This study examines horizontal load bearing systems of multi-storey steel buildings such as vertical concentric braces to limit horizontal displacements. It elucidates the relationship between layout of vertical braces within overall structural system and horizontal displacement. In this context, a reference model consisted of moment resistant frames was used at first. Structural elements were designed using LRFD load combinations specified in ÇYTHYEDY-2016. For service load combinations, the serviceability limit state combinations used according to ÇYTHE-2016 Section 9. Furthermore, structural system elements are designed to withstand their strengths at mechanism state of loading. Ignoring horizontal displacements, it was designed within the limits of capacity. Then, attempts were made to reduce horizontal displacements by increasing the stiffness of the columns and beams separately. As result of these attempts, inadequacy of the structural system to carry horizontal loads was demonstrated and the importance of vertical stability braces was emphasized. Later, six different vertical brace layouts were assigned for the same building to determine the most appropriate layout under seismic and wind loads.

Horizontal Displacement Limits for Tall Buildings

Several codes interest with lateral displacement limits for structural systems in case of both seismic and wind loading. Seismic design codes usually restrict inter-storey drifts while limit values are given for top lateral displacement under wind loading by considering mostly serviceability limit state.

As per 4.9.1 clause of TBDY-2018 inter storey drifts and effective drifts obtained from non-reduced seismic loads are restricted obeying to equation 4.34b [1].

$$\lambda \times \frac{\delta_{imax}}{h_i} < 0.016\kappa \tag{1}$$

$$\delta_{lim} = \delta_{imax} \leq \frac{0.016\kappa \times h_i}{\lambda} \tag{2}$$

Because structure is made of steel κ is selected as 0.5 and λ is calculated as follow.

$$\lambda = \frac{S_{ae}(T)^{(DD-3)}}{S_{ae}(T)^{(DD-2)}} = \frac{0.0358}{0.0882} \cong 0.4058 \tag{3}$$

$$\delta_{lim} = \frac{0.008 \times 3800}{0.4058} = 74.91\text{mm} \tag{4}$$

In this study, a single limit value for all floors has been reached in terms of inter storey displacement limit since the floor height in the building is the same and 3800 mm.

As stated in the book of Tall Building Structures Analysis and Design "If a tall flexible structure is subjected to lateral or torsional deflections under the action of fluctuating wind loads, the resulting oscillatory movements can induce a wide range of responses in the building's occupants, ranging from mild discomfort to acute nausea" [4]. In case of wind loading, commonly used limits for top lateral displacement vary between $1/400$ and $1/600$ of the total building height, depending on the building type, facade cladding type, and material but the most common one is $H/500$ as mentioned in AISC 360-22 [5].

Structural systems for resisting lateral loads encompass a range of options, including moment resistant frames with varying ductility levels and braced frames. Ductility is a behaviour defined as the ability to undergo significant plastic deformation in structural members before failure. This enables buildings to absorb energy and redistribute stresses among sections of structural members, and is crucial for structural safety and design flexibility. But it is clear that the higher ductility leads the larger deformations.

Structural Analysis and Design Methods

In this study, an eleven-storey steel building was designed respect to provisions of TBDY-2018 [1] and ÇYTHYEDY-2016 [2]. The computer program, ETABS was utilized as addition to hand calculations [6]. Dead loads were obtained from material catalogues and TS-498 [7]. Wind loads are obtained from TS EN 1991-1-4 [8]. Seismic loads were calculated by modal analysis method and base shear force was increased respect to "Equivalent Earthquake Load Method".

Definition of Designed Building

The structure, which will be constructed as a residential building in Bursa, has eleven floors, each 3.8 metres high. There are seven openings of 7 meters in both principal direction at every floor level. Therefore, the overall height reaches 41.8 meters, and the plan dimensions are 49 meters. The composite deck supported by secondary beams is the same in all floors, and in all models. IPE270 secondary beams designed as simple beams, divide 7 meters opening into three equal parts.

The floors of the structure were composite deck supported by secondary beams. HEB profiles were used for columns, HEA profiles for beams and IPE profiles for secondary beams although dimensions of the elements were modified to design within capacity limits on every model basis. Members of concentric braces were formed by TUBO sections. As per ÇYTHYEDY-2016 Section 6.2 secondary effects of non-linearity were taken consideration by reducing moments of inertia of sections [2].

As per Table 3.1 of TBDY-2018, this building is classified as Type 3 according to purpose of use and it belongs to Class 4 according to height classification [1]. Seismic data in the site area with a ground class of Z_C was obtained through the AFAD interface [9]. In accordance with Table 3.4 of TBDY-2018, the building was designed considering strength, and taking account [DD-2] earthquake and [KH] damage level [1].

Base Model with Special Moment Frames

Special Moment Frames of the base model have HEB cruciform column sections. Because frames resist horizontal forces in both principal directions, all beam to column connections are fully moment-resisting connections. Section dimensions were selected to satisfy all SMF requirements except the limitation of lateral displacements and joint rotations.

The reason these moment frames are called special systems, is because of additional limitations. They are detailed to the high-energy absorption capacity in other words high ductility, and achieve superior behaviour during strong earthquakes. Inelastic behaviour is accommodated by plastic hinges in beams at beam-column joints as well as at column bases [10]. TBDY-2018 allows the design of high-ductile frames if requirements are met, such as preserved connection rotation limit (0.04 rad) and strong column weak beam capacity [1]. In addition, because of column buckling is not a ductile phenomenon, columns are expected to remain stable under maximum compression and tension forces [11]. Design codes prefer different alternatives to capture these forces. TBDY-2018 uses overstrength factor to consider these effects, so column axial capacity is controlled with axial forces that produced by amplified earthquake forces. After the initial design, it is observed that section sizes are controlled by $1.2G+Q+0.2S+E_d^{(H)}+0.3E_d^{(Z)}$ load combinations, then the section dimensions are increased disregarding difficulties on the arrangements of details. In this context, first increasing column dimensions then beam dimensions separately were utilized to limit lateral displacements in wind and seismic cases. Because loading and geometry of structure are same in both principal direction, analysis results for them were nearly identical. Therefore, the study was carried out on X-direction displacements. In case of seismic loading, relationship of inter storey drift and column section are presented in Table 1 and Figure 1.

Table 1. Inter storey drift versus column moment of inertia.

Cruciform Column Profile	Beam Profile	Inertia Moment of Column[cm^4]	Δ_{ix} [mm]	Δ_{lim} [mm]
HE450B		91703	214.6	
HE500B		119918	198.3	
HE550B		149139	186.0	
HE600B	HE300A	183810	175.5	74.91
HE700B		271578	155.7	
HE800B		374396	144.2	
HE900B		510407	128.0	
HE1000B		661685	115.3	

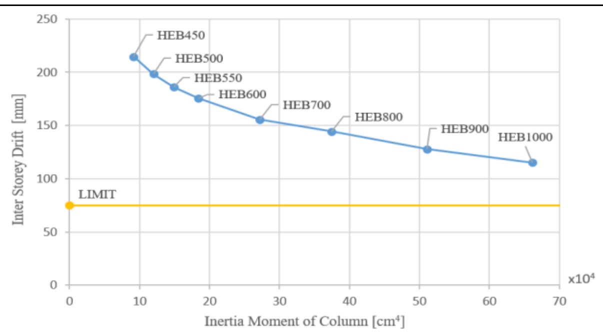

Figure 1. Relationship between column moment of inertia and inter storey drift.

Table 2 presents effects of change in beam sections on floor displacement. In case of seismic loading, relationship of inter storey displacement and beam section is shown in Figure 2.

Table 2. Inter storey drift versus beam moment of inertia.

Beam Profile	Cruciform Column Profile	Inertia Moment of Beam [cm^4]	Δ_{ix} [mm]	Δ_{lim} [mm]
HE300A		18260	214.6	
HE340A		27690	152.9	
HE360A		33090	133.1	
HE400A	HE450B	45231	105.1	74.91
HE450A		63720	83.9	
HE500A		86970	69.9	
HE550A		111900	60.9	
HE600A		141200	54.1	

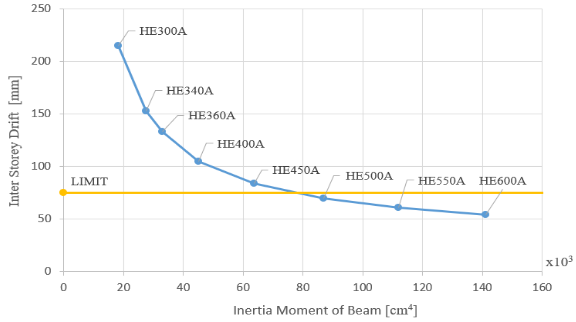

Figure 2. Relationship between beam moment of inertia and inter storey drift.

In case of wind loading, relationship of top lateral displacement with column sections are presented in Table 3 and Figure 3.

Table 3. Top displacement versus column moment of inertia.

Cruciform Column Profile	Beam Profile	Inertia Moment of Column [cm^4]	Δ_{ix} [mm]
HE450B		91703	93.4
HE500B		119918	88.6
HE550B		149139	84.9
HE600B	HE300A	183810	81.7
HE700B		271578	75.7
HE800B		374396	70.7
HE900B		510407	65.9
HE1000B		661685	61.9

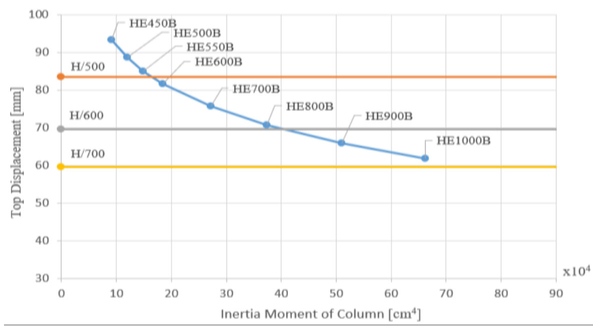

Figure 3. Relationship between column moment of inertia and top displacement.

In case of wind loading, relationship of top lateral displacement with beam sections are presented in Table 4 and Figure 4.

Table 4. Top displacement versus beam moment of inertia.

Beam Profile	Cruciform Column Profile	Inertia Moment of Beam [cm^4]	Δ_{ix} [mm]
HE300A		18260	93.4
HE340A		27690	64.9
HE360A		33090	56.0
HE400A	HE450B	45231	43.7
HE450A		63720	34.1
HE500A		86970	27.8
HE550A		111900	24.0
HE600A		141200	21.1

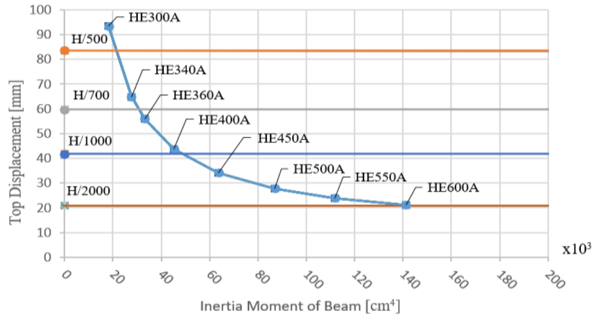

Figure 4. Relationship of beam moment of inertia and top displacement.

Inspections indicated that even though strong column weak beam principle was ignored, base model that designed with special moment frames did not succeed to remain under lateral displacement limit with reasonable sections. The top lateral displacement in seismic loading grows to impractical values as 5-20 times of the top displacement observed in wind loading. It is deduced that vertical braces supporting the building against lateral loads is compulsory for an economic design even the building is not so tall. Therefore, following models are constructed with vertical concentric braces.

Models with Concentric Braced Frames (CBF)

As can be seen from the first model, special moment frames are not economically feasible to limit displacement in high-rise buildings. As, the initial design, it is found that section sizes are controlled by $1.2G+Q+0.2S+DE_d^{(H)}+0.3E_d^{(Z)}$ load combinations. The following models have vertical concentric braces and the most reasonable bracing layout among the structural system is investigated.

Vertical bracing prevents horizontal displacements of frame and uses diagonal members having an angle from horizontal axis. These diagonal members carry axial and shear forces and they connect to beams and columns at their ends. Diagonal bracing is commonly found in three configurations: concentric X-bracing, V-bracing and inverted V-bracing. Of these, concentric X-bracing is the most commonly used option due to its efficiency and balanced resistance to two-way lateral forces. In order to ideal seismic behaviour, they must be designed as having appropriate strength and ductility [12]. CBFs typically have limited ductility compared to moment resistant frames. Because any instability occurrence (buckling) does not permit the diagonal compression members to reach their strength capacity. The direct load transfer mechanism of concentric braced members limits horizontal displacements effectively and offers stiffer frames.

Brace fracture is preferred failure initial failure mode, but it does not in itself trigger immediate collapse [13]. So, diagonal braces are designed as preserved members and until the intended behaviour of the system occurs, all other bearing members must retain their capacities. Tension and compression capacities of diagonal members directly affect design sizes of other components (columns and beams). Therefore, in order to capture lateral displacement and brace layout relation, all columns and beams are designed with LRFD load combinations at capacity limit. The change in the weight of the structure is recorded as dependent variable.

In models with concentric braced frames, bracing layout was same in both principal directions. However, only the vertical braces resisted to horizontal force in the direction X while the braces and planar frame with I section columns worked together against horizontal force in the direction Y. Therefore, lower displacements were observed in the direction Y. Bracing layout of six different models are presented below. In models from the first to the fourth, vertical braces are situated at outer faces or facades of the building, while in the fifth and sixth models braces are at the inner axes to model a stiffer core. Vertical bracing layouts are shown with the elevation views and braced axis names are given below.

Table 5. Braced axes names

Model Number	Braced Axis Name
Model 1	A, H,1,22
Model 2	A, H,1,22
Model 3	A, H,1,22
Model 4	A, H,1,22
Model 5	C, F,7,16
Model 6	C, F,7,16

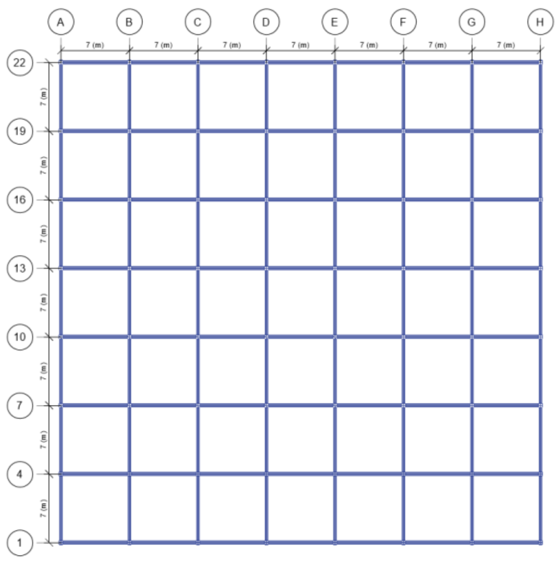

Figure 5. Grid plan of structure.

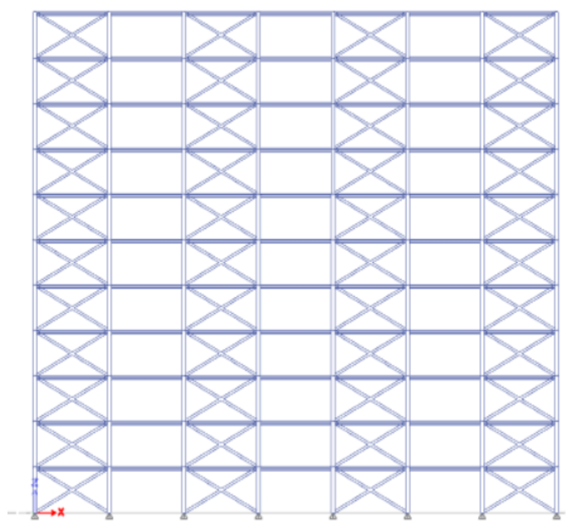

Figure 6. Elevation view of braced axes model 1.

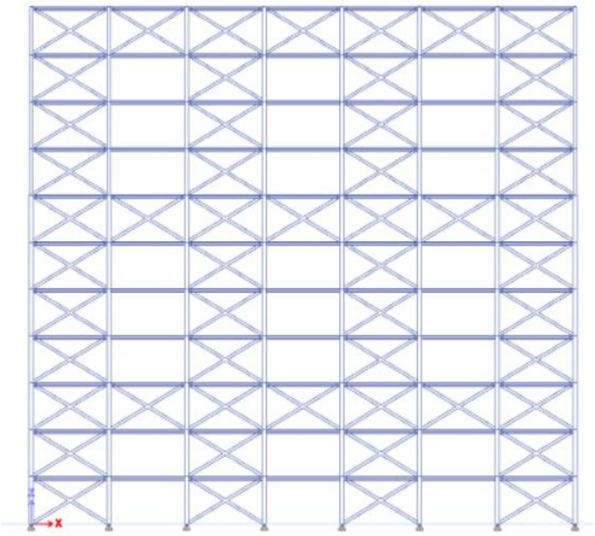

Figure 7. Elevation view of braced axes model 2

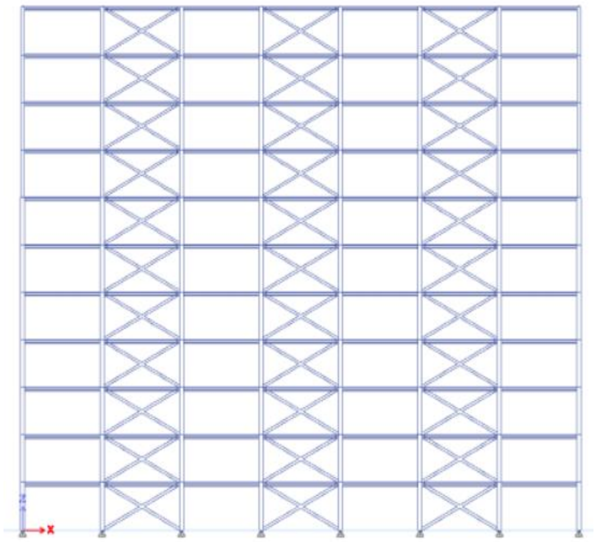

Figure 8. Elevation view of braced axes model 3

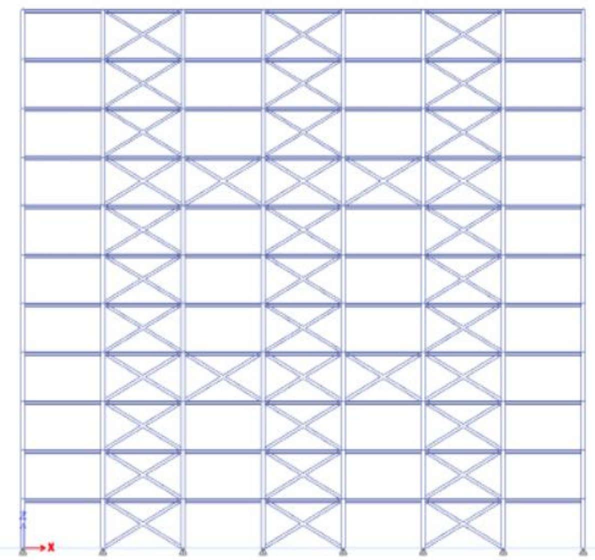

Figure 9. Elevation view of braced axes model 4

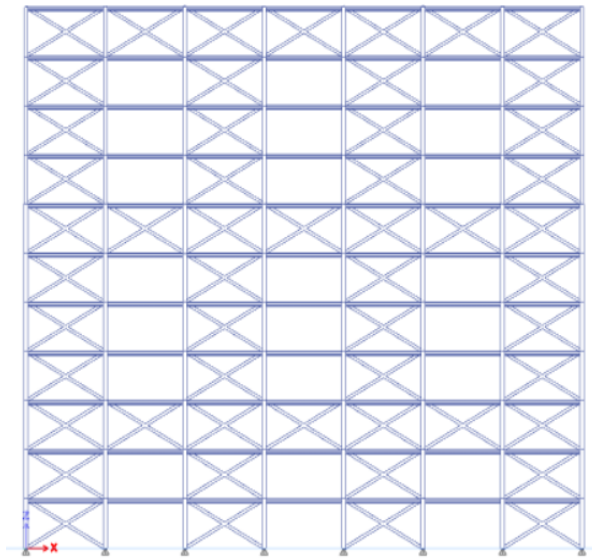

Figure 10. Elevation view of braced axes in model 5

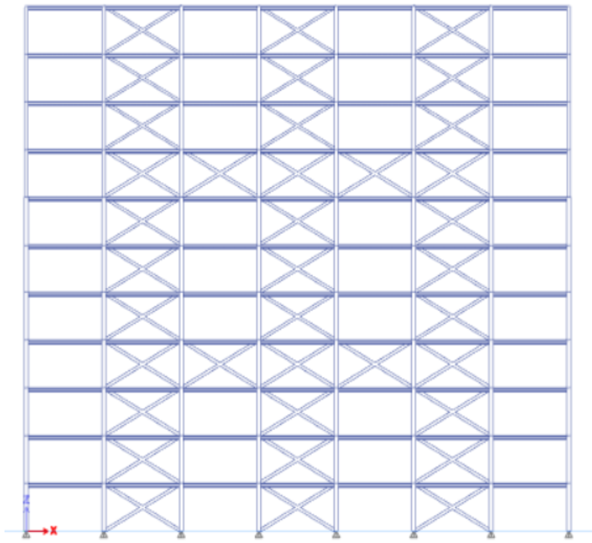

Figure 11. Elevation view of braced axes in model 6.

In the case of seismic loading, comparison of the models in terms of structural weight and lateral displacements is given in Figure 12 (as X-direction) and Figure 13 (as Y-direction).

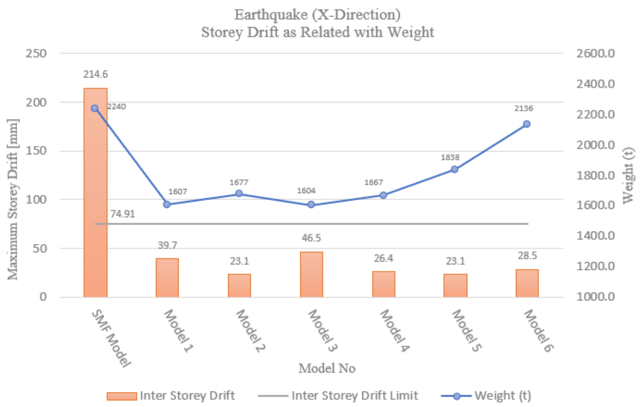

Figure 12. Comparison of models considering by earthquake loads in X-direction

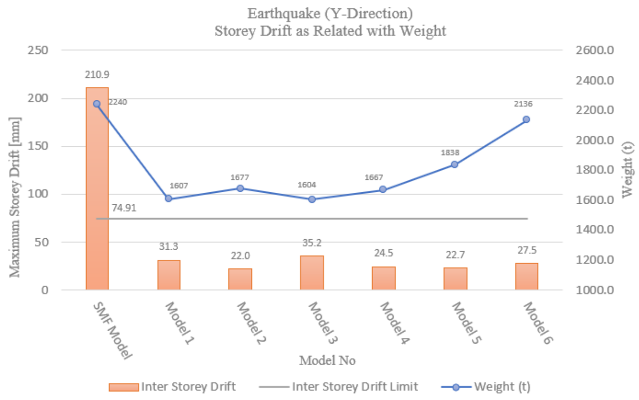

Figure 13. Comparison of models considering by earthquake loads in Y-direction

In the case of wind loading, comparison of the models in terms of structural weight and lateral displacements is given in Figure 14 (as X-direction) and Figure 15 (as Y-direction).

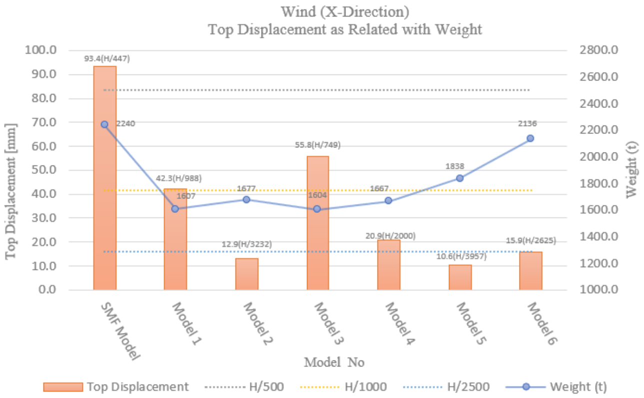

Figure 14. Comparison of models considering by wind loads in X-direction

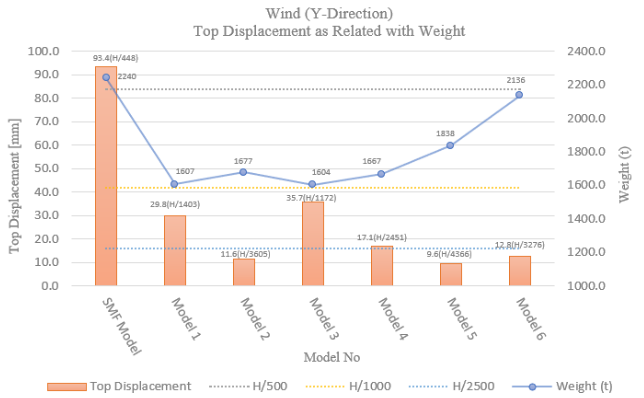

Figure 15. Comparison of models considering by wind loads in Y-direction

Finally, Figure 16 and Figure 17 present comparisons of top lateral displacements produced by seismic load and wind load for models with braced frames.

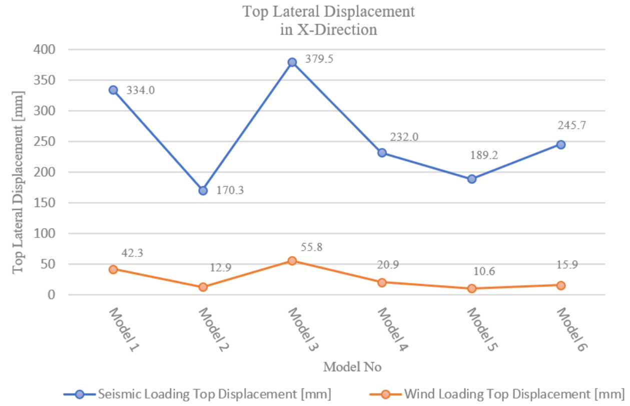

Figure 16. Comparison of X direction top displacements in seismic and wind loading.

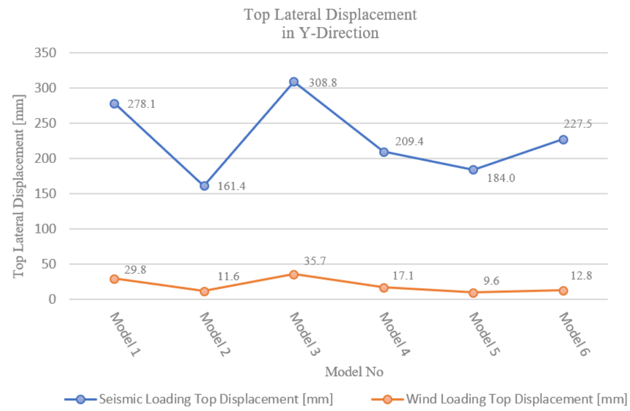

Figure 17. Comparison of Y direction top displacements in seismic and wind loading.

Conclusions

Under horizontal loadings such as earthquake and wind, multi-storey steel buildings produce severe lateral displacements. Horizontal displacements induced by wind loading is much smaller than inter storey drifts observed in earthquake at floor levels, in which weights concentrate. Structural analysis and design of the model with special frames proved that increasing beam rigidity have significant effect on limiting lateral displacements compared to increasing column rigidity. However, it can be seen that use of moment-resistant frames do not assure to limited displacements and is not an economical solution when strong column-weak beam principle is applied in design.

Structural system of high-rise buildings requires vertical braces evidently under horizontal earthquake loads. Lateral displacements reduce if vertical braces continue along intermediate floors in the load direction, even though these additional braces cause a slight increase in weight. Despite of, Model 4 was 3.9 percent heavier than Model 3 its maximum storey drift produced by seismic load in the direction X was 43.2 percent less than Model 3. Reduction percentage of maximum storey drift was 30.4 percent in the direction Y. When the performances of Model 1 and Model 3 are compared, it is deduced that bracing at the outer spans is an optimum choice. Maximum storey drift of Model 1 was 14.6 percent less than ones of Model 3 in the direction X seismic loading even though their weights are nearly same.

It is also observed that use of vertical braces in the outer axes or facades has an advantage relative to use them in inner axes. Here, it should be remembered that studied building is symmetric and regular in plane. Therefore, torsional effects of earthquake on structural system are not observed. Use of vertical braces in the inner axes in a way that they build a rigid core, could be necessary if planar irregularities and torsional effects exist. Although they limit horizontal displacements, they can also cause over-designed members and increase in gravitational loads depending on earthquake design provisions.

Results of wind loading support the advantages of continuous braces along intermediate floors. Despite being 4.4 percent heavier, Model 2 exhibited 69.5 percent less wind-induced top displacement in the X direction compared to Model 1. In case of Y- direction wind loading, Model 2 performed better than Model 1 with the less top displacement of 61.1 percent. Nevertheless, effect of bracing on lateral top displacement is not significant in wind loading as much as in seismic loading. Earthquake loading governs the design of multi-storey buildings in the context of limited lateral displacements.

Ethics committee approval and conflict of interest statement

There is no need to obtain permission from the ethics committee for the article prepared.

There is no conflict of interest with any person / institution in the article prepared.

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