Effect of Different Bracing Systems on the Performance of Metallic Tower

Cătălin BACIU and Marin LUPOAE

***Abstract*—Having multiple destinations, as observation facility, industrial bunkers, equipment bearer (for telecommunication, measurement instruments, etc.), leisure climbing walls or even for specific training of special forces, metallic towers are designed in various forms and dimensions, with different bracing systems. Usually these structures have limited footprint, but with considerable height. The efficiency of bracing systems refers to limiting lateral deformation, but also providing sufficient ductility in order to dissipate a larger energy quantity induced by severe wind gusts or seismic motions. This paper presents a comparative analysis on a steel tower in unbraced frames solution or with different bracing systems, under wind and seismic loadings.**

***Index Terms*—bracing systems, link, steel frame structure, storey drift, vibration mode.**

I. INTRODUCTION

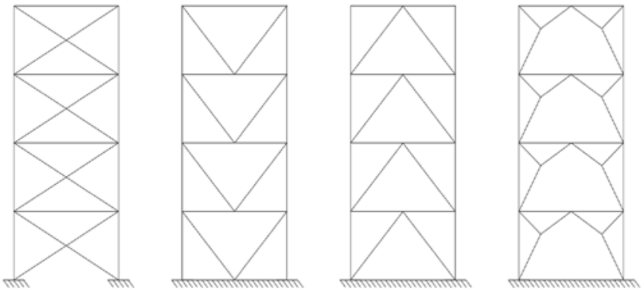
Metallic towers are light structures, basically executed in modules in special factories or workshops, then easily assembled on sites. The cross-section could be triangular (with 3 support pillars) or square (with 4 support pillars) and could be constant or variable on the vertical; for heights exceeding 25 meters reducing the horizontal cross-section of the cells or using tie rods fixed on the ground are efficient solutions.

Rational conformation of metallic towers, with different forms and dimensions, depends on the requirements imposed by the building destination, by specific demands and by the type of loads and their way of action. Thus, the response of a tall metallic structure to wind or seismic loads is strongly influenced by the presence and by the type of the bracing systems.

Multi-storey bare frame structures (without braces), named Moment Resisting Frames (MRF) are characterized by a relatively large number of rigid beam-column joints, that offer a good structural redundancy and a high capacity of energy dissipation. Nevertheless, these are flexible structures and, especially for tall towers, the lack of braces determines large deformations and efforts on elements. Introducing a proper bracing system, which works together with the frames, stiffens the structure and improves the state of stresses and deformations, determining in the end reducing the steel consumption.

In terms of braces currently used for metallic structures, there are classical solutions (Fig. 1) and modern, more efficient ones (Fig. 2), which provide a better dissipative behavior for the entire structure.

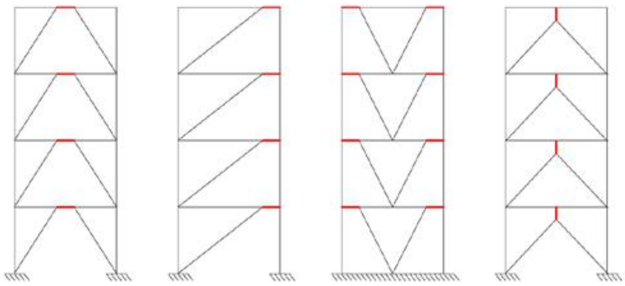
Concentrically Braced Frames (CBF) are the most commonly utilized bracing systems, having a high level of lateral stiffness and a low level of ductility. For CBFs to be utilized in high seismic regions, special detailing is required to ensure that the frames behave properly (in the prescribed manner) [1]. The dissipative elements are limited to braces under tension (Fig. 1a) or braces under tension and compression (Fig. 1 b,c,d).



a) b) c) d)

Figure 1. Bracing systems with CBF

Eccentrically Braced Frames (EBFs) have become a widely accepted form of seismic force resisting system in the last four decades. An EBF is a brace frame system in which one end of the brace is connected eccentrically to another brace end or to a frame corner. EBFs successfully combine the high level of ductility of MRFs and the high level of stiffness of CBFs. The cross brace of an EBF provides the elastic stiffness of CBF and the eccentricity of the cross brace (elements in red color in Fig. 2) creates a link that is responsible for ductility [1].



a) b) c) d)

Figure 2. Dissipative bracing systems with EBFs

The length of the beam between two braces or between brace and the frame node is known as a link, which acts like a seismic fuse, brace forces being introduced to the frame through shear and flexure in the link. Depending on the type of developed plastic mechanism, the dissipative links are classified as follows: short links (shear deformation dissipates energy), long links (bending deformation dissipates energy) and intermediate links (both shear and bending deformation dissipates energy) [2].

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The link, vertical or horizontal, representing the main dissipative part of the bracing system, is in a state of static equilibrium under axial force – N, shear force – V and bending moment – M (Fig. 3).

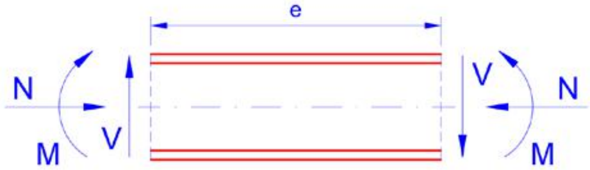

Figure 3. Static equilibrium of the link

For usual cases, the above classification is detailed depending on the value of eccentricity, e, as follows [2]:
a) short dissipative element, when

$$e < 1.6 \cdot \frac{M_{pl,link}}{V_{pl,link}} \tag{1}$$

b) long dissipative element, when

$$e > 3 \cdot \frac{M_{pl,link}}{V_{pl,link}} \tag{2}$$

c) intermediate dissipative element, when

$$1.6 \cdot \frac{M_{pl,link}}{V_{pl,link}} \le e \le 3 \cdot \frac{M_{pl,link}}{V_{pl,link}} \tag{3}$$

where $M_{pl,link}$ is the plastic bending moment and $V_{pl,link}$ is the plastic shear force, both values corresponding to hinge formation.

Interesting valuable scientific papers analyze the use of EBF systems for steel and reinforced concrete (RC) frame structures. Z. Khan, B. R. Narayana, S. A. Raza [3] studied the seismic behavior of a RC building equipped with different bracing systems by performing linear static and non-linear static analysis, comparing various parametric results such as Storey drift and Storey forces. Pushover curves obtained both in X and Y directions emphasized the contribution to structural rigidity of centrically X-shaped and V-shaped bracing systems.

A nonlinear pushover analysis is carried out by M. Mubeen, K. N. Khan, M. I. Khan [4] for high rise steel frame building with different patterns of eccentric bracing systems, resulting that the models with bracings have lesser vulnerability as compared to the frames without bracings. Compared with steel bare frame model, the eccentrically bracing systems (Eccentric Backward Brace, Eccentric Forward Brace, Eccentric V-Brace and Inverted V-Brace) reduced the maximum displacements up to 90%, but increased the base shear capacity up to 49%. In conclusion, the Eccentric Inverted V Brace model has increased the structural performance level as compared to other bracing models.

The significant improvement to the seismic response of RC structures equipped with dissipative bracing systems, such as eccentric braces (EBs) and buckling restrained braces (BRBs) is illustrated in the paper of Mazzollani, Della Corte and D'Aniello [5]. The experimental tests were carried out on two similar two-storey one-bay RC structures, respectively equipped with EBs and BRBs. For eccentric braces a Y-inverted bracing configuration, with a vertical steel link, was adopted. The experimental test has shown that the collapse was due to failure of link end connection and the lateral strength achieved by the tested EBs was significantly larger than the expected one (because of the shear over-strength exhibited by the tested steel links).

A representative paper for Romanian research in the domain of using of EBFs [6] presents numerical simulation and experimental tests on planar steel frame, where the connections beam-column are dog-bone type. Push-over and Time-History analyses are carried out for both fixed and removable links. The final results determine the influence of eccentric braces and dog-bone connections to the dissipative response of the analyzed structure. A further study was materialized in the paper [7], where composite steel-concrete eccentrically braced frames are used in experimental and numerical tests. Results show that the presence of concrete slab decisively influences the initiation and the development of plastic hinges in steel section.

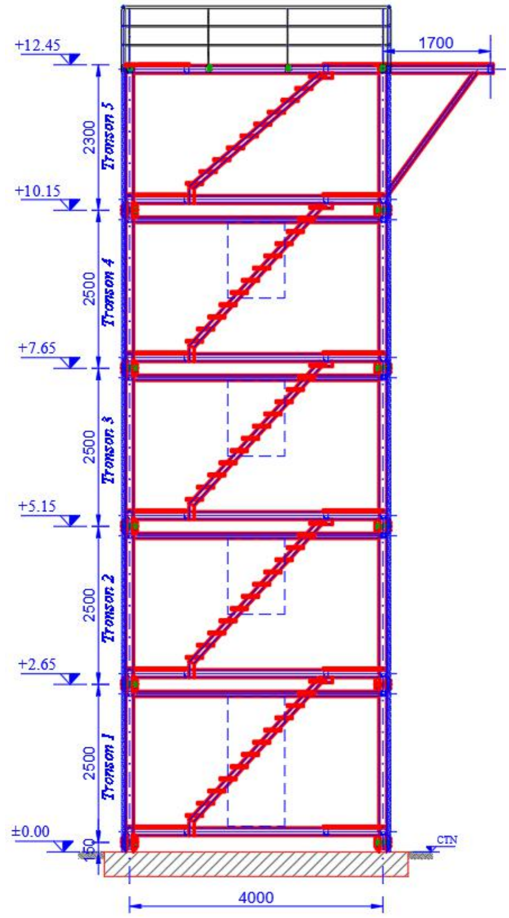

Figure 4. Metallic tower to be designed (lateral view)
Model A – Moment Resisting Frames

If all the above papers generally use I-beam and wide flange beam for the links of eccentric braces, the papers [8], [9] describe the development and results of a finite element parametric study of eccentrically braced frame links having hollow rectangular cross sections and the experimental verification of the proposed design requirements. A finite element parametric study consisting of over 200 models of EBF links with tubular cross sections has been conducted. Results of the parametric study lead to certain recommendations for compactness ratio limits for tubular cross sections used as links in EBFs [8]. In order to verify proposed design requirements, experimental tests were carried out [9]. Results indicated that tubular links satisfying the proposed compactness and stiffeness requirements can achieve

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the target plastic rotations for wide-flange links when subjected to the loading protocol.

II. Study Case

Starting from a list of certain requirements, a metallic tower is to be designed with a special destination (training troops for climbing, abseiling): 12.50 m total height, 3 × 4 m plan dimensions, 4 intermediate platforms and the fifth on top. A metallic staircase is used to access the upper floors (Fig. 4).

In order to facilitate the effective site execution of the tower, European profiles and rolled steel pipes were used, in a truss system solution, with columns, beams, braces and access stairs; the structure is modular, with sections with a maximum height of 2.50 m, so that they can be easily transported and assembled. On three lateral sides (one long and two short) three-layer sandwich panels are placed: on the long side there will be window openings, one of the short sides is prepared for climbing, and the opposite side, on the top platform, a console platform is arranged with a minimum length of 1.70 m for controlled assisted jumps. Metallic grind is placed on every platform and on stairs (Fig. 5).

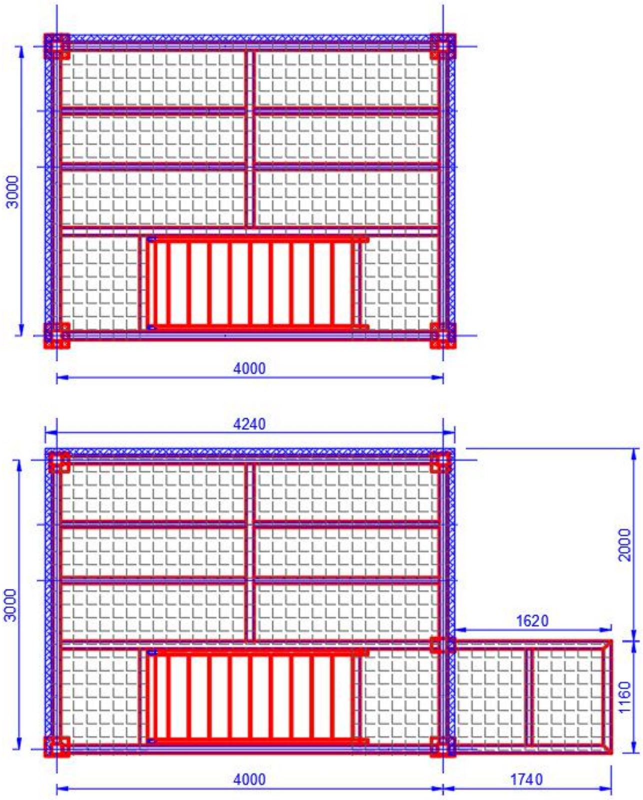

Figure 5. Plan views of the current platform (upper view) and of the top platform (lower view)

For the pre-sizing of the tower elements and for the subsequent detailed analysis, the various possible scenarios during the construction life were taken into account. The main types of loads considered in the analyses, according to the regulations in force, are as follows:

- self-weight of structural and non-structural elements;
- payload (weight of personnel in the phases of execution, operation and maintenance of the tower); for the operation phase, the establishment of the calculation value for the personnel weight loads will also take into account the dynamic factor generated by the specific actions of the trainings;
- wind, seismic, snow and hoarfrost loads.

The metal columns, placed on every corner of the tower, are made of square hollow section - Tp120×6. Perimeter horizontal beams are fixed between the columns, both at the bottom of each section (rectangular pipe Td120×80×6) and at the top of them (square pipe Tp80×6). Between the lower secondary beams there are arranged support elements (square pipe Tp60×5 at maximum distances of 600 mm) for the floor grilles.

All the tower elements are made of S235 structural carbon steel. The welds are continuous (corner joints), with the thickness equal to at most 0.7 of the minimum thickness of the elements to be joined. High-strength M12 pretightening screws - group 8.8 are used to connect the modules (8 screws on each flange), Fig. 6.

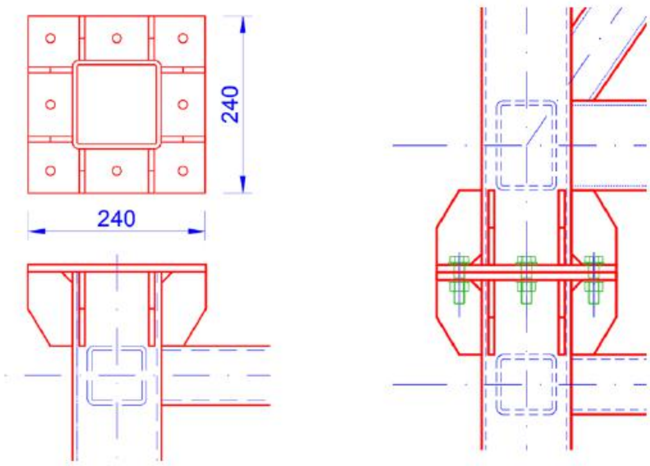

Figure 6. Plan views of the current platform (upper view) and of the top platform (lower view)

The proper solution to connect the modules using flanges is according to the actual regulations and is based on the valuable experimental and numerical results presented by Y. Chen et al. in the paper [10]. Providing the sufficiently strong ribs, welds, and flange plates, the capacity of the flange was found to be mainly predominated by the bolt strength. Both the experimental and the numerical results show a linear load interaction curve, in terms of the ultimate capacity.

III. Analyzed Models

The tower was modeled using the automatic calculation program ETABS, software dedicated to structural analysis, thus determining the efforts in the components, dynamic characteristics and displacements of the structure.

Four different models were analyzed:

Model A – Moment Resisting Frames, without braces (Fig. 4, 7a);
Model B – frames with cross braces on every side (Fig. 7b);
Model C – frames with chevron (Lambda type) braces on every side (Fig. 8a);
Model D – frames with eccentric braces on every side, (Fig. 8b).

All models are designed with linear elements such as columns, beams and braces, the last ones being considered articulated at both ends. For Model D, the length of the link is 1.00 m, falling into the category of long dissipative element.

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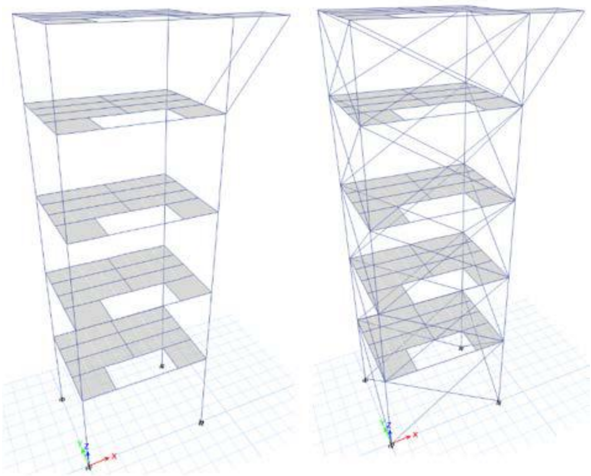
Gravitational loads are represented by the own weight of structural and non-structural elements (P), by the payload ($L = 1\ kN/m^2$ - value established depending on the destination of the tower). Lateral loads are generated by wind (W) and seismic input (E); the action of the wind took into account the provisions of the design code [11], the reference value of the dynamic wind pressure being 0.5 kPa. The seismic loading was taken into account in accordance with the specifications of the design code [2], with the peak value of the terrain acceleration $a_g = 0.25\ g$, and with the control period $T_c = 1\ s$. The response spectrum method was applied to determine the response of the structure for all four models. For final loading scenarios, snow or hoarfrost loads are not taken into account because the tower should be cleared before being used in troop training.



a) b)

Figure 7. 3D views of Model A – Moment Resisting Frames (a) and of Model B – Frames with cross braces (b)

The load combinations adopted in the analysis are:

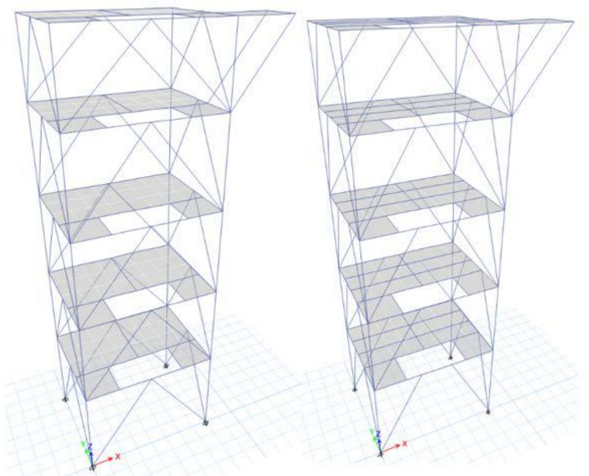
a) Fundamental Combination (FC): 1.35P+1.5L;

b) Fundamental Combination with Wind pressure on long façade, along Oy axis (FCW1): 1.35P+1.5L+1.5W_long;

c) Fundamental Combination with Wind pressure on short facade, along Ox axis (FCW2): 1.35P+1.5L+1.5W_transv;

d) Special Combination with seismic load along Ox axis (GSX): P+0.3L+E_x;

e) Special Combination with seismic load along Oy axis (GSY): P+0.3L+E_y.



a) b)

Figure 8. 3D views of Model C – Frames with chevron braces (a) and of Model D – Frames with eccentric braces (b)

Base shear is the estimation of maximum expected lateral force which will occur at the base of a structure due to ground motion during the earthquake. When seismic base shear is estimated, specific behavior factor (q), which accounts for the ability of a structure to dissipate energy, is applied for every type of structure, according to actual regulation [2]: for bare frame (Model A) $q = 6$; for frames with cross braces (Model B) $q = 4$; for frames with chevron braces (Model C) $q = 2.5$ and for frames with eccentric braces (Model D) $q = 6$.

In order to determine the design spectrum for each model starting from elastic spectrum, the critical damping for this type of steel structure is ξ = 3% and the correction factor, according to the design code [2], becomes:

$$\eta = \sqrt{\frac{10}{5+\xi}} = 1.118 \qquad (4)$$

The elastic and design spectra used for each type of model are displayed in Fig. 9. The same value of behavior factor for Model A and D leads to the same design spectrum. As expected, a higher energy dissipation capacity determines a lower base shear value.

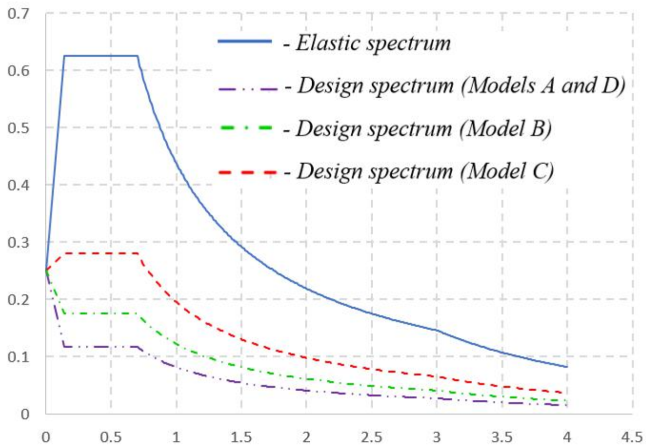


Figure 9. Elastic and design spectra for considered models

All models are analyzed for each combination, above mentioned, in order to determine their response in terms of displacements / drifts and efforts. Also modal characteristics of structures will offer further knowledge of typical behavior of models.

IV. Results and Discussions

Fundamental vibration periods of the models, shown in Table I – second column, are strongly influenced by structural stiffness; introducing braces into a bare frame structure increases the rigidity. The most flexible model is obviously A, and models B and C have almost the same fundamental periods. As a consequence, the displacements and rotations are larger for flexible structures.

TABLE I. Fundamental Vibration Periods and Base Shear

Model	Periods [s]	Base Shear [kN]
A	1.26	9 **(7%)**
B	0.20	21 **(12%)**
C	0.19	31 **(18%)**
D	0.47	17 **(10%)**

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In Fig. 10, lateral drifts generated by seismic loads for all four models are compared and the first conclusion is that for all four, the maximum drifts are below the admissible value (Ultimate State Limit). The reduced mass of the building generates low values of inertial lateral loads from earthquake action, meaning low values of shear base and further, limited deformations even for MRF. Examining Table I – third column, it results that the highest value of shear base is obtained by the system with chevron braces, which have the lowest value of behavior factor, while the lowest value of shear base corresponds to the bare-frame system. On the third column in the Table I, the percentage in the brackets represents the ratio between the shear base and the total weight of the building, or in other words the global seismic coefficient; the highest flexibility of MRF also brings the lowest value of this coefficient.

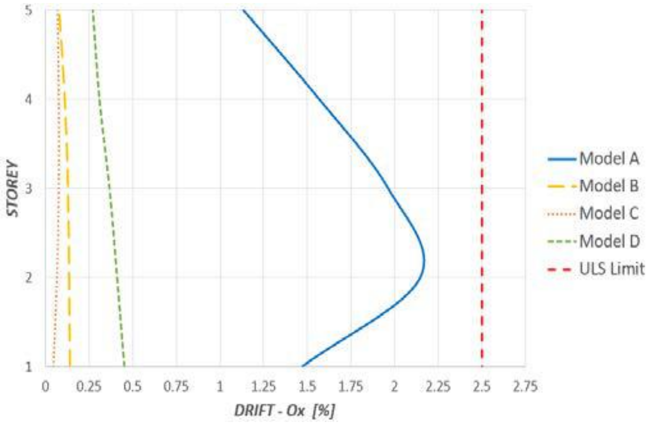

Figure 10. Drifts on Ox direction for all models

On the other hand, the shape of the graph is firmly changing when braces are added to MRF; if for the bare-frame structure the maximum drift is formed at one-third above the ground and then the values gradually decrease, for the structures with braces, drift distribution is changing, becoming more uniform on the height of the building.

In terms of lateral displacements, the maximum values determined at the upper floor of the tower, Table II, indicates that for bare-frame structure (Model A) wind loads generate larger deformations than seismic loads, while for each model with braces the displacements are quiet similar. Setting the limit value of maximum lateral displacement at 2.5% of building height, representing 310 mm for analyzed tower, the cases of MRF under wind loads are the only ones with exceeded values (highlight cells of Table II). There can also be seen an important difference between Model A and all the other models, the presence of braces offering higher rigidity. We expected Model D to be less efficient in terms of displacement than Models B and C, knowing that the main advantage of EBF lays in the higher energy dissipation capacity.

TABLE II. MAXIMUM LATERAL DISPLACEMENTS [MM]

	Load Combination			
Model	**GSX**	**GXY**	**FCW1**	**FCW2**
A	198	156	530	431
B	14	20	14	7
C	8	13	13	7
D	42	70	73	35

Comparing the axial forces in columns at the base of the tower, Table III, also demonstrates that wind loads generate higher structural efforts than seismic loads for all the models. Moreover, adding braces into the frames drastically reduces the bending moments in columns and beams, offering the possibility to use lighter structural elements in a more efficient manner. Rising prices in recent times requires the adoption of proper structural configurations that lead to the lowest possible quantities of materials and labor.

TABLE III. MAXIMUM AXIAL FORCES, N [KN] AND BENDING MOMENTS, M [KN·M] IN COLUMNS

	Load Combination							
Model	**GSX**		**GXY**		**FCW1**		**FCW2**	
	N	**M**	**N**	**M**	**N**	**M**	**N**	**M**
A	47	5	50	5	225	91	149	70
B	60	0.7	66	0.6	224	3.7	145	3
C	55	0.3	63	0.4	192	4.7	128	3.4
D	49	0.8	51	1.1	147	13	111	8.5

In Table IV, axial forces in the braces are compared, resulting once again that higher wind loads determine bigger efforts in the structural elements (in this case in braces) than in the case of seismic scenarios. The cross braces are less loaded, almost half compared to the eccentric braces.

TABLE IV. MAXIMUM AXIAL EFFORTS IN BRACES [KN]

	Load Combination			
Model	**GSX**	**GXY**	**FCW1**	**FCW2**
B	12	15	75	48
C	16	20	102	65
D	14	19	150	80

V. CONCLUSION

The results obtained in this study proved that the presence of braces improves the response of structures under lateral loading, but the proper solution has to be in accordance with the type and level of loads and with the specific requirements.

Braces increase structural rigidity, reduce displacements and efforts in elements (columns, beams), but increase the level of shear base. The shape of the graph for drifts on the height of the tower is firmly changing from MRF to the systems with braces (it becomes more uniform, with lower values). For this kind of structure, the cross braces are the most efficient.

The lower mass of the building reduces the influence of seismic loads compared to wind loads. Using in this case a dissipative system, like EBF, tends to be less efficient than for the case of office or residential buildings, for example.

A further research direction, which completes the image of tower-type structures is to determine the capacity of the structure using a Push-over method for a system with an important mass.

Design of the braces, usually long and slender, has to consider their behavior under compression, when the possible loss of stability reduces the strength capacity. A solution for this matter is the usage of Buckling Restrained Braces (BRB). Another direction to develop the study presented in the paper is to analyze the structural response when BRB system is used.

Choosing the proper brace systems for the buildings to be designed or for existing structures to be rehabilitated determines an efficient usage of steel elements according to the required loads and specific conditions. Reducing the steel consumption determines, as a consequence, the reducing of CO_2 emissions.

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