Seismic vulnerability analysis of steel buildings in Bogotá, Colombia

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SUMMARY:
A number of mid-height steel buildings have been erected recently in Bogotá. Their seismic risk might be high, given the new microzonation of Bogotá and the lack of previous studies; remarkably, the response reduction factors were commonly obtained from general recommendations. The objective of this work is to investigate the seismic performance of these buildings. This study is carried out on eighteen representative buildings. All these edifices have plan symmetry and are uniform along their height. Span-length: 6 – 8 m; number of floors: 5, 10, 15. Earthquake-resistant systems: moment-resistant frames, concentrically-braced frames and eccentrically-braced frames (using chevron braces). For each building, eight seismic zones have been considered. The vulnerability has been evaluated by "push-over" analyses. In the moment-resistant frames and in the eccentrically-braced frames the nonlinearities are concentrated in the plastic hinges near the connections; in the concentrically-braced frames the nonlinearities are allocated in the braces.

Keywords: seismic vulnerability, steel buildings, push-over analysis, retrofit, Colombia

1. INTRODUCTION

A number of mid-height steel buildings have been erected in the last years in Colombia, principally in Bogotá; they are mostly intended for dwelling, administrative and commercial use. These constructions have been designed according to the former Colombian seismic code [NSR-98 1998], which was basically inspired by the American regulations, mainly those by FEMA, ATC, ASCE and AISC. The seismic risk of these buildings might be high, given that a new microzonation of Bogotá has been recently released [Decreto 523 2010] (Figure 1) and that no comprehensive theoretical studies about their vulnerability have been carried out; in particular, the values of the response reduction factor (R) are commonly obtained from general recommendations that not account for the individual characteristics of each building. The objective of this work is to investigate the seismic performance of these buildings in order to provide more accurate estimates of the response reduction factor and to be able to formulate design criteria; these recommendations might be incorporated to the Colombian seismic design code. As well, retrofit strategies will be proposed in further studies.

Figure 1 displays the abovementioned seismic microzonation of Bogotá. In Figure 1 "Cerros" corresponds to rock and stiff soil with top soft layers not exceeding 6 m ($v_{s,30}$ > 750 m/s, where $v_{s,30}$ accounts for the shear wave velocity in the top 30 m), "Piedemonte" refers to soft alluvial and colluvial soil (200 m/s < $v_{s,30}$ < 750 m/s) and "Lacustre" corresponds to very soft clay deposits ($v_{s,30}$ < 175 m/s). As well "Aluvial" refers to mid-quality alluvial deposits (175 m/s < $v_{s,30}$ < 300 m/s) and "Lacustre-Aluvial" shows intermediate characteristics in between "Aluvial" and "Lacustre". Finally "Depósito ladera" are unstable high slope soils, with relevant risk of land-sliding; the construction is restricted. In the categories "Lacustre", "Aluvial" and "Lacustre-Aluvial", the numbers indicate the depth (in m) of the soft deposit layers. In the category "Piedemonte", subcategories A, B and C do not differ deeply.

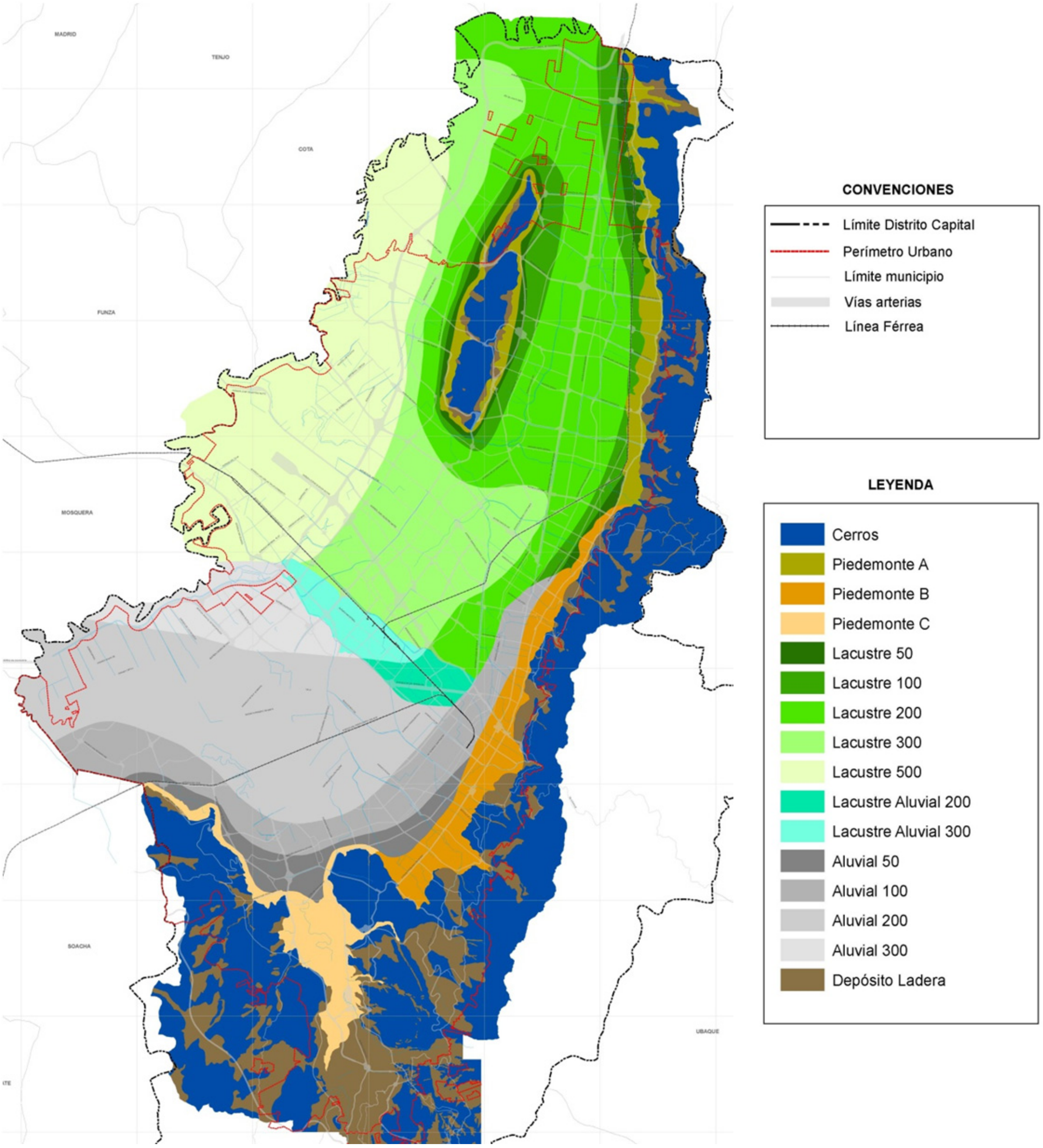


Figure 1. Seismic microzonation of Bogotá

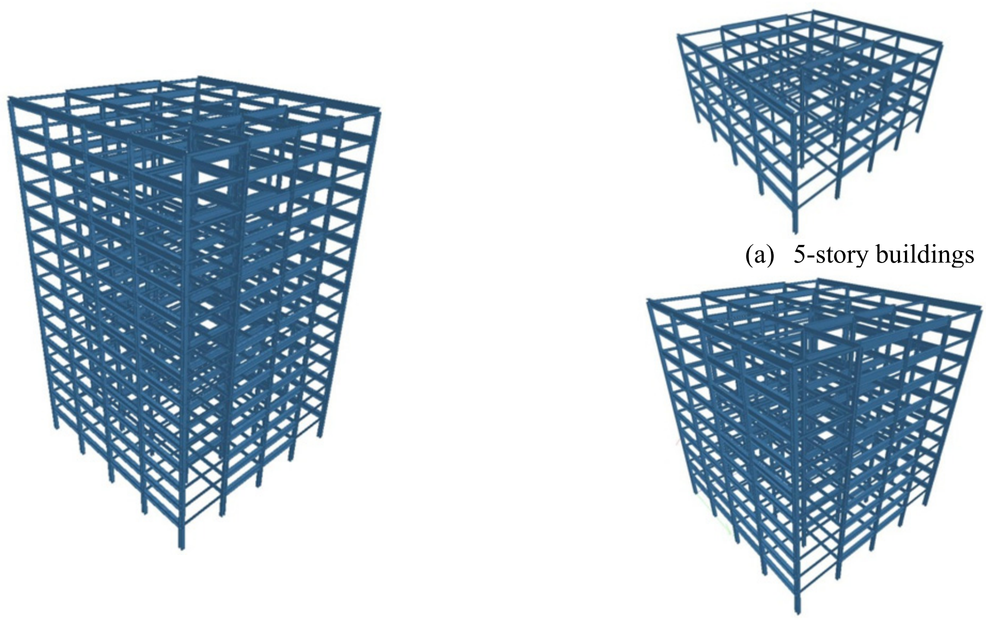
This study is carried out on eighteen buildings that have been selected to represent the vast majority of the existing ones. All these buildings have plan symmetry and are uniform along their height; in half of them the span-length is 6 m in both directions (this corresponds mainly to housing use), while in the others it is 8 m (mainly for commercial and administrative use). Three earthquake-resistant systems have been considered: moment-resistant frames (MRF), concentrically-braced frames (CBF) and eccentrically-braced frames (EBF); in these last two cases, chevron braces are contemplated. The numbers of floors are 5, 10 and 15. To obtain representative results, the structures of these buildings have been designed according to the current Colombian regulations [NSR-10 2010]. For each building, nine seismic zones (in Bogotá, see Figure 1) have been considered; they are “Cerros” (corresponding to Zone 1 in the previous seismic microzonation of Bogotá [Decreto 193 2006]), “Piedemonte A, B and C” (corresponding to Zone 2 in the previous seismic microzonation of Bogotá) and “Lacustre 50, 100, 200, 300 and 500” (corresponding to Zone 3 in the previous seismic microzonation of Bogotá). The other seismic zones indicated in Figure 1 are not considered since they contain only small numbers of steel buildings.

The seismic vulnerability has been evaluated, in the framework of the Performance-Based Design [Priestley et al. 2007], by static nonlinear analyses (push-over). The structural behavior of the buildings is described by a finite element model with frame elements; the cooperation of the top concrete layer is neglected. In the moment-resistant frames and in the eccentrically-braced frames the nonlinearities are concentrated in the plastic hinges located in the connections; in the concentrically-braced frames the nonlinearities are concentrated in the braces. In the push-over the pushing forces are shaped as the first eigenmode. The target drifts (performance points) are determined, for each performance objective (IO, LS and CP) according to the recommendations of the FEMA [FEMA 356 2000].

The results of the push-over analyses provide estimates of the response reduction factor; it is concluded that, in some cases, the values recommended by the design code are unconservative. As well, some of the buildings exhibit inadequate behavior for several performance objectives.

2. CONSIDERED BUILDINGS

Eighteen buildings are selected to represent the vast majority of the steel edifices in Bogotá. All these buildings have plan symmetry and are uniform along their height; all the columns are continuous down to foundation and the influence of the basements is not considered. The main carrying-load system is composed of steel columns and of steel decks topped with a concrete layer. The plan floor of the buildings is square, with four bays in each direction; there are 25 columns, which are laid according to an orthogonal regular pattern. The buildings are distinguished by the span-length in both directions, by the number of floors and by the type of earthquake-resistant system. Two span-lengths and three numbers of floors are considered: 6 – 8 m and 5 –10 – 15, respectively. Three earthquake-resistant systems have been considered: moment-resistant frames, concentrically-braced frames and eccentrically-braced frames; in these last two cases, chevron braces are contemplated. The cooperation of the infill walls is neglected since, according to the common construction practices in Colombia, they are regularly separated from the main structure. Figure 2 shows overall views of the structures of the selected buildings.



(a) 5-story buildings

(b) 15-story buildings

(c) 10-story buildings

Figure 2. Selected representative buildings

To obtain representative results, these buildings have been designed according to the current

Colombian regulations [NSR-10 2010] by considering normal importance (given their dwelling, administrative and commercial use). The seismic design is based in the simplified method [NSR-10 2010], it implies assuming the same fundamental period in both directions (estimated from empirical expressions). In Bogotá the design acceleration is $A_a = 0.15$ g. The dead load has been assumed as 2.5 kN / m^2 (slab self-weight) + 1.5 kN / m^2 (partitioning walls) + 1 kN / m^2 (facilities) + 1.5 kN / m^2 (cladding system, distributed along the whole surface of the façade). Live load is L = 2 kN / m^2; according to the Colombian code, 50% of this load is considered to act simultaneously with the seismic action. The design input spectra are obtained from [NSR-10 2010]. In spite that the buildings are symmetric, the accidental eccentricities established by the [NSR-10 2010] are considered. The damping factor has been assumed equal to 5%. The Colombian code states a design inter-story drift equal to 1%; this condition is the most restrictive in most of the MRF buildings, being comparatively less restrictive in the CBF and EBF.

The columns are made of ASTM A-572 steel (f_y = 342 MPa) while the beams and joists are made of ASTM A-36 (f_y = 248 MPa) steel; this difference attempts getting earlier failures in the beams than in the columns. The compressive strength of the topping concrete is f_c' = 21 MPa; the depth of the steel deck is 50 + 70 mm (120 mm concrete depth). Remarkably, the beam-column connections have been designed by following the guidelines by [FEMA 350 2000]. Figure 3 shows a plan view of a typical floor slab of the selected buildings. Figure 3 shows that each building contains two seismic-resistant frames in the y direction (A and E frames) while in the x direction there are four seismic-resistant frames (inside 1, 2, 4 and 5 frames); as discussed previously, each of these resistant parts can be either a moment-resisting frame (MRF), a concentrically braced frame (CBF) or and eccentrically braced frame (EBF).

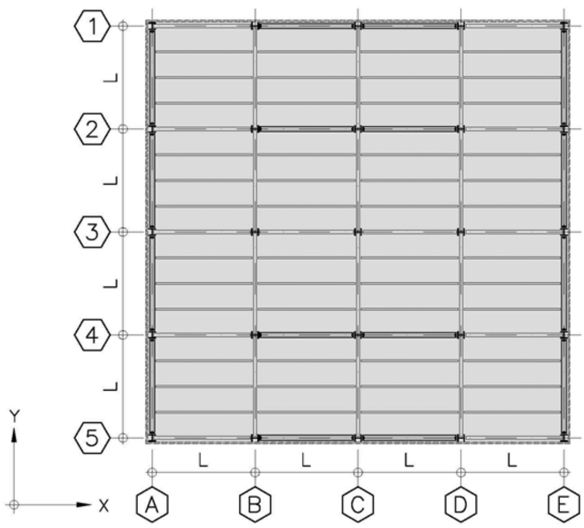
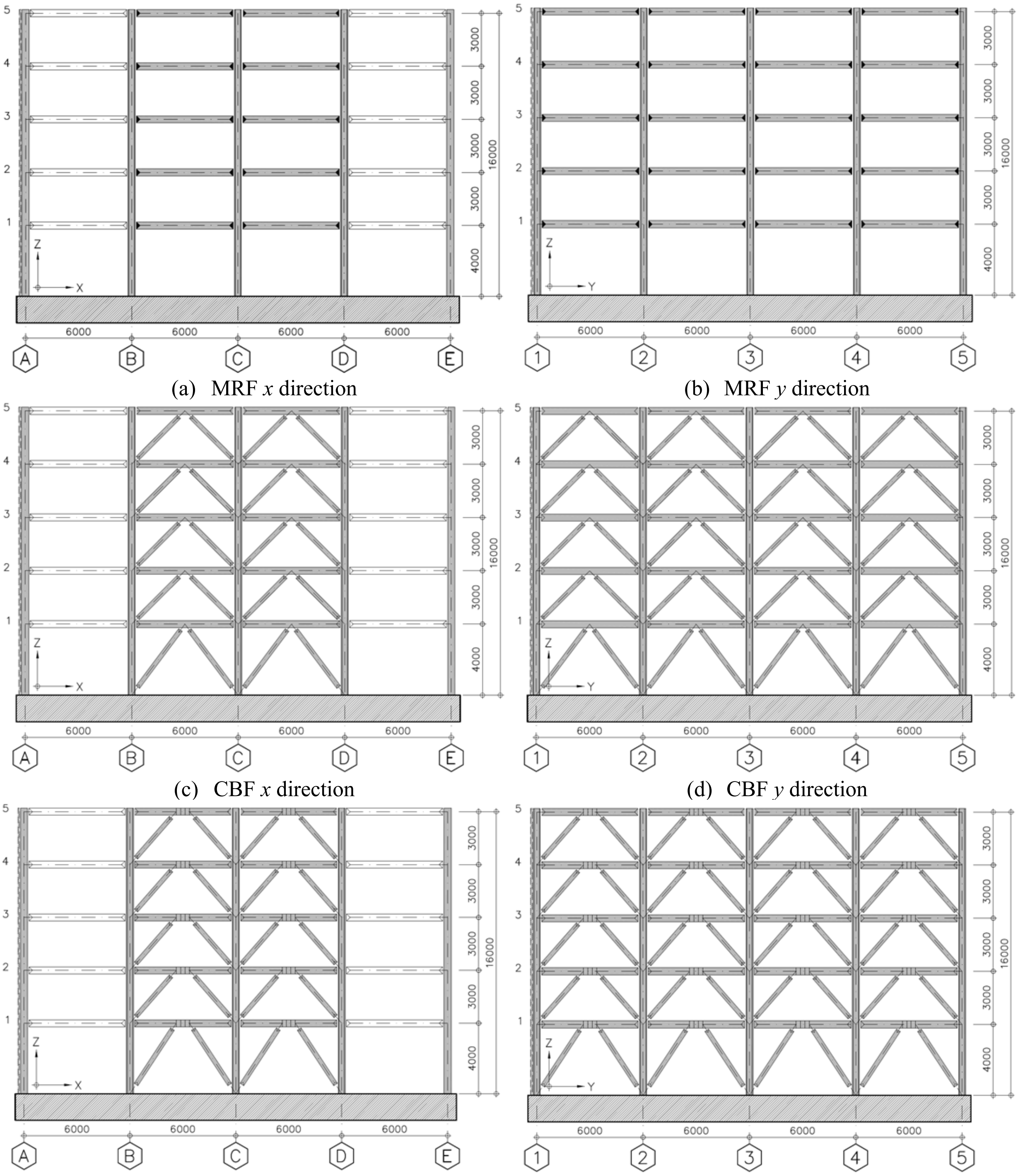


Figure 3. Floor slab layout

In the 6 × 6 buildings the height of the first floor is 4 m and the upper floors are 3 m high; in the 8 × 8 buildings those heights are 4.50 m and 3.50 m, respectively. The separation in between the joists is 1.50 m (L / 4) for the 6 × 6 buildings and 1.60 m (L / 5) for the 8 × 8 buildings. The columns, beams and joists are made of W sections [AISC 2010] and the braces are made with square hollow sections. The joists are W10×15 and W12×19 for the 6 × 6 and 8 × 8 buildings, respectively. Figure 4 shows overall elevation views of the selected 5-story buildings; the configurations of the buildings with 10 and 15 floors are similar.

Table 1 displays the main structural features of the eighteen selected buildings. In the notation 5 – 6 × 6 – MRF, "5" accounts for the number of stories, "6 × 6" refers to the span-length (in m) in both directions and "MRF" means Moment-Resisting Frame; analogously "CBF" and "EBF" corresponds to Concentric-Braced Frames and to Eccentric-Braced Frames, respectively. Given that the structural parameters depend on the soil conditions, the properties in Table 1 correspond to the most demanding terrain "Piedemonte-B", as indicated in Figure 1. The steel profiles [AISC 2010] correspond to the seismic parts of the structure (highlighted members in Figure 3 and in Figure 4).



(a) MRF x direction (b) MRF y direction

(c) CBF x direction (d) CBF y direction

(e) EBF x direction (f) EBF y direction

Figure 4. Elevation views of the 5-story buildings

Table 2 displays the main seismic design parameters for the representative buildings, for the "Piedemonte-B" zone. The weights correspond to the aforementioned D + 0.5 L load level. The periods T_0 have been determined from the empirical expressions suggested by the [NSR-10 2010]. The fundamental periods T_F have been derived from linear elastic modal analyses and, hence, refer to initial (undamaged) conditions; the comparison among the values of T_0 and T_F shows no major differences. The fundamental periods T_F listed in Table 1 show that the initial stiffness in both directions is similar; comparison among the MRF, CBF and EBF cases shows that the stiffening generated by the braces is important, mostly in the concentrically braced buildings. As well, the buildings spanning 6 m are significantly stiffer than those spanning 8 m. The response reduction factor (R) is obtained as indicated in the [NSR-10 2010]. Last two columns in Table 2 show the dimensionless spectral ordinates S_a / A_a where A_a = 0.15 g, as discussed previously. The constant-acceleration branch of the design spectrum ranges in between $T = 0$ and $T_C = 0.56$ s; the amplification factors accounting for the microzonation are F_a = 1.95 and F_v = 1.70 [NSR-10 2010, Decreto 523 2010]. The important values of S_a / A_a in the last two columns in Table 2 show that the input acceleration in the bedrock is significantly amplified in the top of the buildings; in the 5-story buildings this effect is contributed both by the soft soil and by the rather stiff building.

Table 1. Representative buildings as designed for the "Piedemonte B" zone

Building	First floor beams	Top floor beams	First floor columns	Top floor columns	First floor braces	Top floor braces
5 – 6 × 6 – MRF	W36×159 / W36×176	W36×109 / W36×159	W14×342 / W14×342	W14×257/ W14×257	-	-
5 – 8 × 8 – MRF	W36×232 / W36×260	W36×150 / W36×230	W14×500 / W14×500	W14×370/ W14×370	-	-
5 – 6 × 6 – CBF	W36×150 / W36×160	W30×99 / W36×150	W14×257 / W14×257	W14×193/ W14×193	HSS5"×½"	HSS4"×½"
5 – 8 × 8 – CBF	W36×130 / W36×245	W36×135 / W36×210	W14×370 / W14×370	W14×283/ W14×283	HSS6"×¾"	HSS4"×½"
5 – 6 × 6 – EBF	W36×135 / W36×135	W30×90 / W30×132	W14×283 / W14×283	W14×211/ W14×211	HSS5"×⅝"	HSS4"×½"
5 – 8 × 8 – EBF	W36×232 / W36×260	W30×124 / W33×201	W14×426 / W14×426	W14×311/ W14×311	HSS6"×½"	HSS4"×½"
10 – 6 × 6 – MRF	W36×311 / W36×311	W36×233 / W36×233	W14×500 / W14×605	W14×370 / W14×426	-	-
10 – 8 × 8 – MRF	W36×439 / W36×439	W36×328 / W36×328	W14×730 / W14×730+PL¾"	W14×550 / W14×605	-	-
10 – 6 × 6 – CBF	W36×280 / W36×280	W36×210 / W36×233	W14×270 / W14×426	W14×283 / W14×311	HSS7"×½"	HSS5"×½"
10 – 8 × 8 – CBF	W36×328 / W36×328	W36×300 / W36×300	W14×550 / W14×730	W14×398 / W14×426	HSS8"×¾"	HSS6"×¾"
10 – 6 × 6 – EBF	W36×260 / W36×260	W36×233 / W36×233	W14×500 / W14×605	W14×370 / W14×426	HSS7"×⅝"	HSS5"× ⅝"
10 – 8 × 8 – EBF	W36×439 / W36×439	W36×328 / W36×328	W14×730 / W14×730+PL¾"	W14×550 / W14×605	HSS8"×½"	HSS6"×½"
15 – 6 × 6 – MRF	W36×342 / W36×342	W36×233 / W36×233	W14×550 / W14×605	W14×283/ W14×283	-	-
15 – 8 × 8 – MRF	W36×485 / W36×485	W36×328 / W36×328	W14×730+PL1" / W14×730+PL1½"	W14×500/ W14×500	-	-
15 – 6 × 6 – CBF	W36×280/ W36×280	W36×194 / W36×194	W14×500 / W14×550	W14×342/ W14×342	HSS8"×½"	HSS7"×½"
15 – 8 × 8 – CBF	W36×393 / W36×393	W36×230 / W36×230	W14×730 / W14×730	W14×370/ W14×370	HSS10"×¾"	HSS8"×¾"
15 – 6 × 6 – EBF	W36×328 / W36×328	W36×210 / W36×210	W14×730 / W14×730+PL¾"	W14×550 / W14×605	HSS8"×⅝"	HSS7"× ⅝"
15 – 8 × 8 – EBF	W36×439 / W36×439	W36×300 / W36×300	W14×550 / W14×605	W14×283/ W14×283	HSS10"×½"	HSS8"×½"

Table 2. Design parameters for the representative buildings. "Piedemonte B" zone							
Building	Weight (kN)	T_0 (s)	T_F (x) (s)	T_F (y) (s)	R	S_a / A_a (x)	S_a / A_a (y)
5 – 6 × 6 – MRF	17285	0.627	0.662	0.629	4.5	4.109	4.324
5 – 8 × 8 – MRF	29429	0.726	0.743	0.706	4.5	3.660	3.853
5 – 6 × 6 – CBF	17285	0.362	0.392	0.372	4.5	4.873	4.873
5 – 8 × 8 – CBF	29429	0.421	0.437	0.415	4.5	4.873	4.873
5 – 6 × 6 – EBF	17285	0.523	0.584	0.555	5.4	4.658	4.873
5 – 8 × 8 – EBF	29429	0.621	0.651	0.618	5.4	4.178	4.401
10 – 6 × 6 – MRF	34496	1.215	1.123	1.067	4.5	2.422	2.549
10 – 8 × 8 – MRF	58760	1.211	1.266	1.203	4.5	2.148	2.261
10 – 6 × 6 – CBF	34496	0.635	0.644	0.611	4.5	4.224	4.452
10 – 8 × 8 – CBF	58760	0.717	0.720	0.684	4.5	3.778	3.977
10 – 6 × 6 – EBF	34496	1.001	0.959	0.911	5.4	2.836	2.986
10 – 8 × 8 – EBF	58760	1.173	1.073	1.019	5.4	2.608	2.669
15 – 6 × 6 – MRF	51707	1.525	1.540	1.463	4.5	1.763	1.859
15 – 8 × 8 – MRF	88091	1.726	1.738	1.651	4.5	1.565	1.647
15 – 6 × 6 – CBF	51707	0.816	0.865	0.822	4.5	3.145	3.309
15 – 8 × 8 – CBF	88091	0.972	0.969	0.921	4.5	2.807	2.953
15 – 6 × 6 – EBF	51707	1.287	1.289	1.222	5.4	2.110	2.226
15 – 8 × 8 – EBF	88091	1.424	1.444	1.372	5.4	1.884	1.983

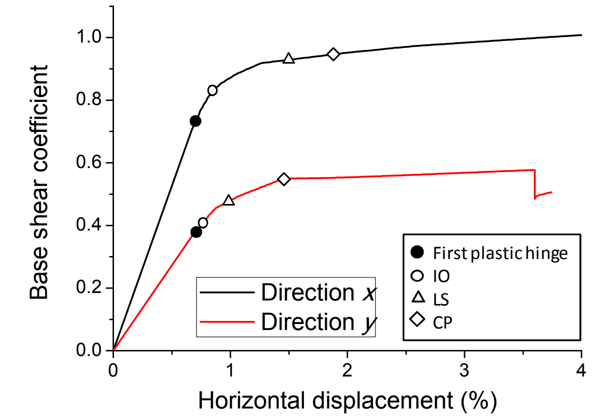
3. NUMERICAL MODELLING OF THE STRUCTURAL BEHAVIOR

The structural behavior of the selected buildings is described with 2-D finite element models with frame elements; the cooperation of the top concrete layer is neglected. In the MRF buildings the behavior is considered basically linear while the nonlinearities are concentrated in plastic hinges located near the beam-column connections. In the CBF buildings [Tapia, Tena 2008, Mahmoudi, Zaree 2011] the first failure arises when the compressed braces reach their critical buckling forces, accounting for the initial imperfections; once these braces are disengaged, next failures correspond to plastic hinges in the section of the main beams that are adjacent to the beam-brace connections. In the EBF buildings the behavior is considered basically linear while the nonlinearities are concentrated in plastic hinges located in the connections between the braces and the main beams. The moment-curvature laws of the plastic hinges are derived from the structural parameters of the steel as suggested in [FEMA 356 2000].

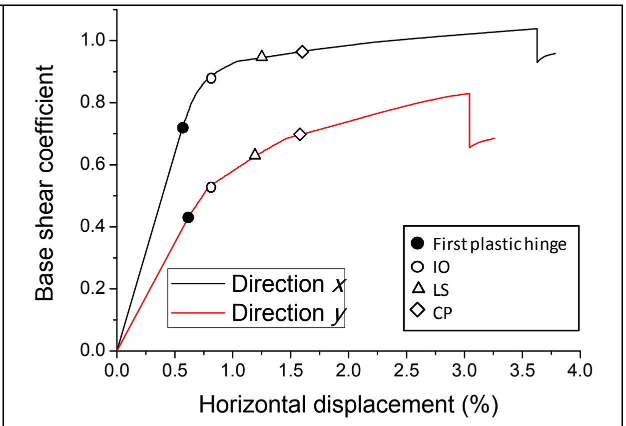
4. PUSH-OVER ANALYSES

This section displays some preliminary results of the 2-D nonlinear static (push-over) analyses. The demanding spectra are obtained from the current Colombian design code [NSR-10 2010] and from the recently-issued microzonation for Bogotá [Decreto 523 2010]. For LS (Life Safety) such spectra are intended to correspond to 475 years return period and for IO (Immediate Occupancy) and CP (Collapse Prevention) they correspond to 43 and 975 years, respectively [FEMA 356 2000]. The target drifts are determined, by intersecting the capacity curves and the demand spectra, as indicated in [FEMA 356 2000]. Given the rather lateral flexibility of these buildings, the soil-structure interaction is not accounted for and second-order analyses are performed; in most of the cases the differences with the first-order analyses are small.

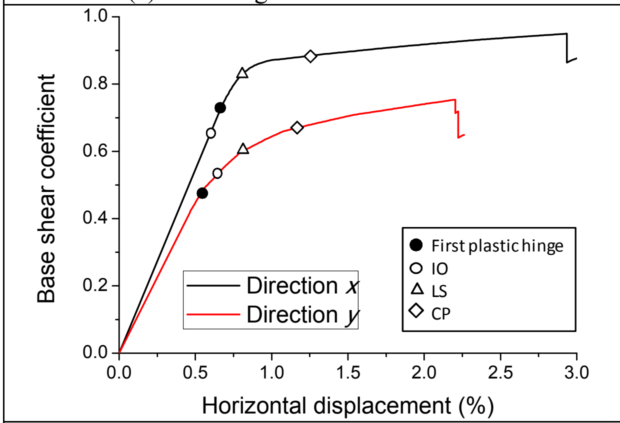
Figure 5 displays the capacity curves of the buildings with Moment-Resisting Frames. Each Figure contains the curves corresponding to the x and y directions (Figure 3 and Figure 4). In each capacity curve, point "•" indicates the onset of the first plastic hinge, which is coincident with the end of the linear branch; points "○", "△" and "◇"correspond to Target Drifts IO, LS and CP, respectively. Plots from Figure 5 show that, except in one case, the capacity curves for the x direction are clearly above those of the y direction; this difference is due to the irregular influence of the aforementioned design inter-story drift (1% [NSR-10 2010]). Comparison among the left and right pairs of curves shows that the capacity curves of the buildings with span-lengths 6 and 8 m are analogous. For a given building, the comparison among the Target Drifts IO, LS and CP, shows that they are rather similar; this implies comparable levels of damage in both directions. Comparison among points "•" and "O", indicates that in most of the cases the level of damage that correspond to the Target Drift IO is rather moderate; this suggest a satisfactory behavior. Conversely, for Target Drifts LS and CP the situation is unclear; ongoing studies are being carried out to further clarify this issue. Following the classic equal-displacement approach [Priestley et al. 2007], the ductility is determined as the ration between the collapse and the yielding displacements; the obtained results range in between 3.22 (building 10 – 8 × 8 – MRF, *x* direction) and 5.8 (building 15 – 8 × 8 – MRF, *y* direction). The comparison among these results and the response reduction factors listed in Table 2 shows that, in many cases, the analyzed buildings do not possess the required ductility.



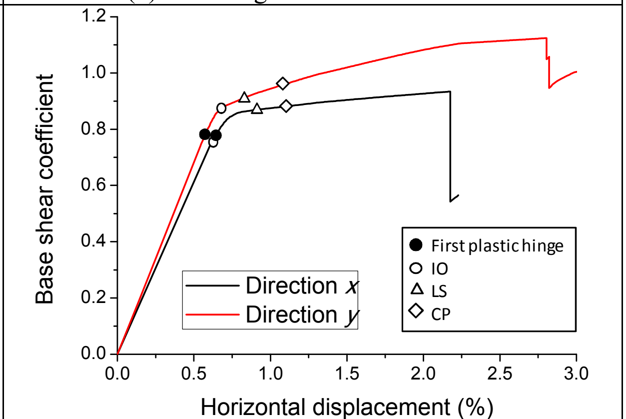
(a) Building 5 – 6 × 6 – MRF



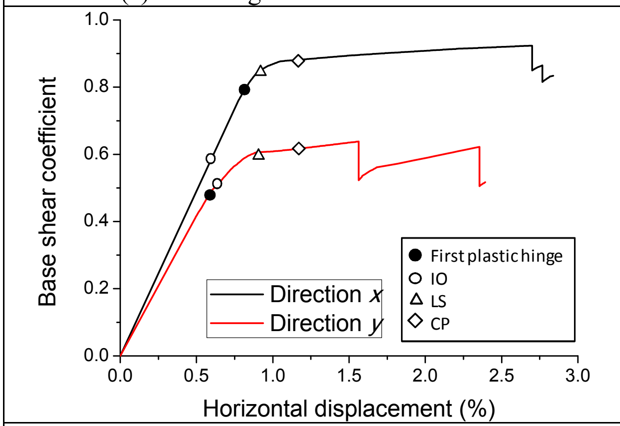
(b) Building 5 – 8 × 8 – MRF



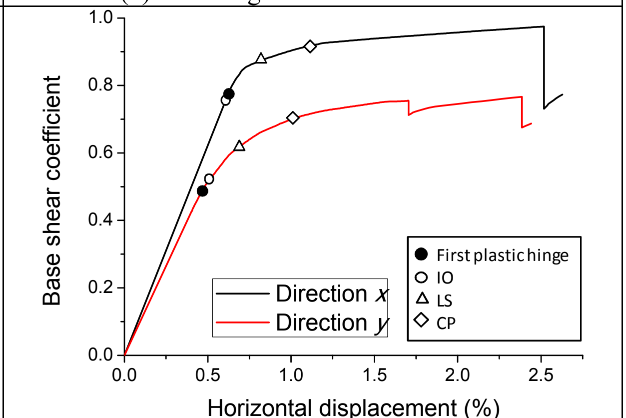
(c) Building 10 – 6 × 6 – MRF



(d) Building 10 – 8 × 8 – MRF



(e) Building 15 – 6 × 6 – MRF



(f) Building 15 – 8 × 8 – MRF

Figure 5. Capacity curves and Target Drifts of buildings with Moment-Resisting Frames. Zone "Piedemonte-B"

5. CONCLUSIONS

This work presents a numerical vulnerability assessment of eighteen 5, 10 and 15-story steel buildings. These buildings are selected to represent the vast majority of similar buildings erected recently in Bogotá, Colombia. The span-lengths are 6 m and 8 m and three lateral resistant systems are considered: moment resisting frames (MRF) and concentrically and eccentrically braced frames (CBF and EBF); only chevron braces are considered. The vulnerability is estimated by static push-over analyses. The preliminary results of these analyses provide estimates of the response reduction factor; it is concluded that, in some cases, the values recommended by the design code are unconservative. As well, some of the buildings apparently exhibit inadequate behavior for several performance objectives.

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