BEHAVIOR OF REGULAR STEEL MOMENT RESISTING CONCENTRICALLY BRACED FRAMES (MRCBFs) IN SEISMIC ZONES

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ABSTRACT :

An equation is proposed to improve the design of medium-rise ductile steel moment-resisting concentrically braced frames. The equation was derived by defining the minimum strength ratio for the resisting columns of the moment frames and the bracing system to have consistent strong column – weak beam – weaker brace collapse mechanisms assumed in the design premises of international building codes. The expression is supported with the pushover analysis of 26 ductile steel MRCBFs, with different heights and two different bracing configurations. From the analysis of the research results a strong relationship is found between the collapse mechanism and the building height, beams stiffness and the conceptual design for the bracing system.

KEYWORDS: Steel, concentrically braced frames, collapse mechanisms, building codes

1. INTRODUCTION

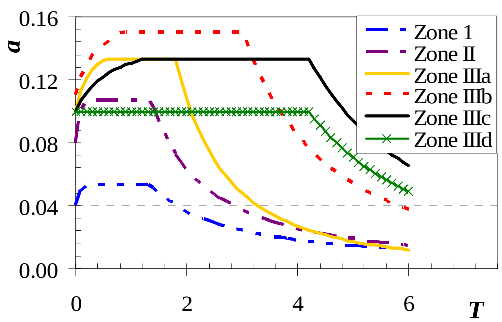
According with the guidelines of Mexico's Federal District Code (MFDC-04), that are similar to other international codes available, a concentrically braced frame (CBF) should be analyzed considering the shear resisting frame contribution and the bracing system contribution. All the stories should be able to resist, without the bracing system contribution, at least the 50% of the seismic force. It is implicitly assumed in MFDC-04 that following this simple recommendation, besides general design guidelines for the structural elements and connections, a consistent global inelastic behavior is obtained, that is, a strong column – weak beam – weaker brace collapse mechanism is insured.

Nevertheless, the results of recent research studies suggest that following general guidelines, steel CBFs do not adjust acceptably with the described collapse mechanism assumed in the design philosophy, as it is shown in Khatib *et al*. (1988), Remennikov and Walpole (1998), Elghazouli (2003) and Tapia (2005). In these studies it has been found that the behavior of the MRCBFs is dominated by the inelastic behavior of braced panels which is rather different from the behavior of conventional moment frames. The study presented in this paper focused its attention on the performance of MRCBFs buildings with chevron bracing, in order to improve current knowledge and to suggest design recommendations that may lead to consistent collapse mechanisms as assumed in building codes.

2. SUBJECT BUILDINGS

Twenty-six buildings from four to sixteen stories in height were studied with two different bracing configurations. Buildings were designed according with MFDC-04 for a soft soil site (zone *IIIb*) and a seismic response modification factor Q=3, the maximum allowed for these structures. The corresponding inelastic design spectra are depicted in Figure 1. Building models where designed according to MFDC-04 (i.e., Tena-Colunga 1999). Other design criteria for member sections of the resisting frames can be found elsewhere (Tapia-Hernández and Tena-Colunga 2007). The typical floor plan considered in the study is shown in Figure 2.

MRCBFs were designed for different shear resisting strength ratios between the bracing system itself and the corresponding columns of the moment frame (Fig. 3). Three different seismic shear contributions for the columns were taken for each considered building height. On the right side of Figure 3, the one hundred percent of the column contribution represents the resisting frame without the bracing system; whereas in the left side, the zero percent of column contribution represents a truss system. In addition, two different bracing configurations were studied for each five-bay building models proposed, that simulated the internal and external CBFs (Fig. 2 and 4).



Inelastic spectra (Q= 3)

Figure 1. Design spectra according with MFDC-04

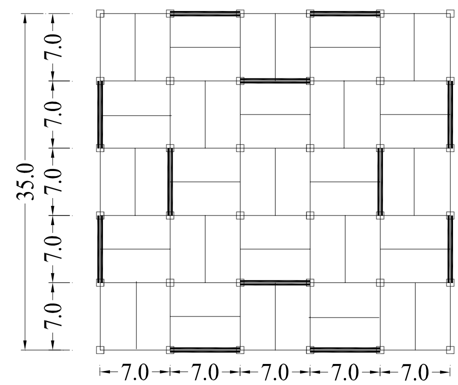


Figure 2. Floor plan of the building in meters

The designed member sections for the CBF modeled are presented in Table 1. For each building model, the same W beam section was used in all the stories. Columns were modeled with rectangular box sections made of A-50 steel using the same width and changing their thickness from bottom to top, whereas the box sections for the bracing system are made of A-36 steel and have the same width but different thicknesses for each model.

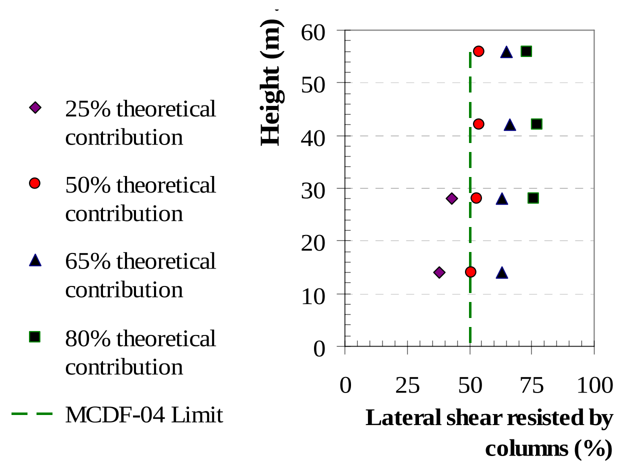


Figure 3. Lateral load contribution of columns

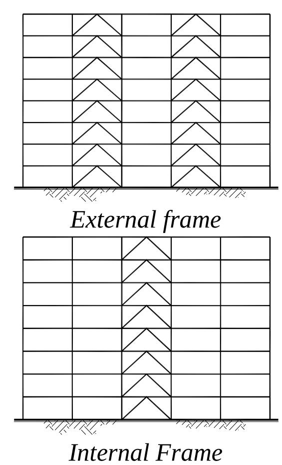


Figure 4. Elevation of the studied frames

3. NONLINEAR ANALYSIS

Pushover analyses of the frame models were carried out using the Drain-2DX computer program (Prakash *et al*. 1992). The lateral force distribution used was consistent with the seismic static method of analysis established in MFDC-04. The mechanisms presented in following sections show by a color scale the magnitude of inelastic deformations, normalized in relationship with the maximum yielding rotation in beams and columns or axial extension and axial shortening in braces. In the braced bays, the brace in the left side was in tension with axial extensions, while the brace in the right side was in compression with axial shortenings. In this paper, only the external frames are shown, the internal models could be consulted in Tapia-Hernández and Tena-Colunga (2007).

4.1 Models where columns resist near 25% of the total seismic shear force

Although models *Ch4p2*, *Ch4i2*, *Ch8p2* and *Ch8i2* were designed with slender column sections and stocky braces, it was not possible to obtain that the columns will only have a small contribution of near 25% in resisting the lateral seismic shear using commercial ("real") steel sections. These models started their nonlinear behavior with some yield rotations in beams. However, dominating failure mechanisms were soft-story mechanisms with plastic hinge rotations at both columns ends in the lower stories (Tapia-Hernández and Tena-Colunga 2007). The described collapse mechanism is not consistent with the assumed design collapse mechanism. Therefore, in this regard, these results suggest that it is correct the recommendation available in MFDC-04 of not allowed designing CBFs with such a small shear strength contribution for the columns to resist lateral seismic loads.

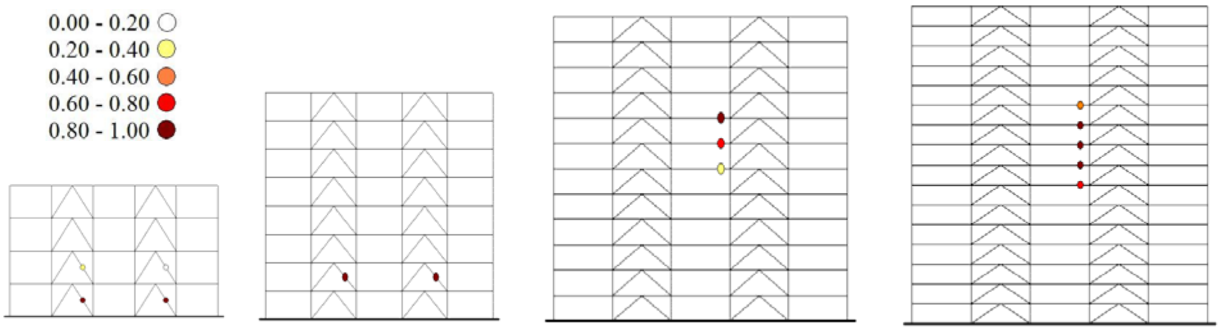
Table 1. Designed member sections for the CBF modeled

Models		Columns (cm.) Rectangular box section	Braces (cm.) Rectangular box section	Beams W-steel section
Ch4p2 and Ch4i2	Width	20 x 20	20 x 20	W 14"x90.7 kg/m
	Thick	t_1=1.27; t_2= 0.95	t= 0.64	
Ch4p5 and Ch4i5	Width	35 x 35	13 x 13	W 14"x101.3 kg/m
	Thick	t_1=1.59; t_2= 1.27	t= 0.64	
Ch4p7 and Ch4i7	Width	60 x 60	13 x 13	W 16"x132.7 kg/m
	Thick	t_1=2.22; t_2= 1.91	t= 0.64	
Ch8p2 and Ch8i2	Width	35 x 35	30 x 30	W 16"x99.8 kg/m
	Thick	t_1=1.59; t_2= 1.27; t_3= 0.95	t_1= 3.81; t_2= 3.18	
Ch8p5 and Ch8i5	Width	40 x 40	15 x 15	W 18"x112.9 kg/m
	Thick	t_1=1.91; t_2= 1.58; t_3= 1.27	t_1= 0.95; t_2= 0.64	
Ch8p6 and Ch8p6	Width	65 x 65	15 x 15	W 24"x125.1 kg/m
	Thick	t_1=2.22; t_2= 1.91; t_3= 1.58	t_1= 0.95; t_2= 0.64	
Ch8p7 and Ch8p7	Width	100 x 100	13 x 13	W 30"x137.4 kg/m
	Thick	t_1=4.12; t_2= 3.49; t_3= 3.18	T_1= 0.95; t_2= 0.64	
Ch12p5 and Ch12i5	Width	45 x 45	25 x 25	W 18"x144.3 kg/m
	Thick	t_1=2.22; t_2= 1.90; t_3= 1.58; t_3= 1.27	t_1= 2.22; t_2= 1.90; t_3= 1.58	
Ch12p6 and Ch12i6	Width	65 x 65	20 x 20	W 24"x217.8 kg/m
	Thick	t_1=3.49; t_2= 3.18; t_3= 2.86; t_4= 2.54	t_1= 1.27; t_2= 0.95; t_3= 0.64	
Ch12p8 and Ch12p8	Width	100 x 100	12 x 12	W 27"x240.1 kg/m
	Thick	t_1=4.45; t_2= 4.13; t_3= 3.82; t_4= 3.49	t_1= 1.27; t_2= 0.95; t_3= 0.64	
Ch16p5 and Ch16i5	Width	50 x 50	35 x 35	W 21"x150.9 kg/m
	Thick	t_1=2.54; t_2= 1.91; t_3= 1.58; t_4= 1.27	t_1=3.49; t_2= 3.18; t_3= 2.86	
Ch16p6 and Ch16i6	Width	60 x 60	20 x 20	W 24"x217.8 kg/m
	Thick	t_1=2.86; t_2= 2.54; t_3= 2.22; t_4= 1.91	t_1=1.58; t_2= 1.27; t_3= 0.95	
Ch16p8 and Ch16i8	Width	120 x 120	15 x 15	W 24"x217.8 kg/m
	Thick	t_1=4.45; t_2= 4.13; t_3= 3.81; t_4= 3.49	t_1=1.27; t_2= 0.95; t_3= 0.64	

4.2 Models where columns resist near 50% of the total seismic shear force

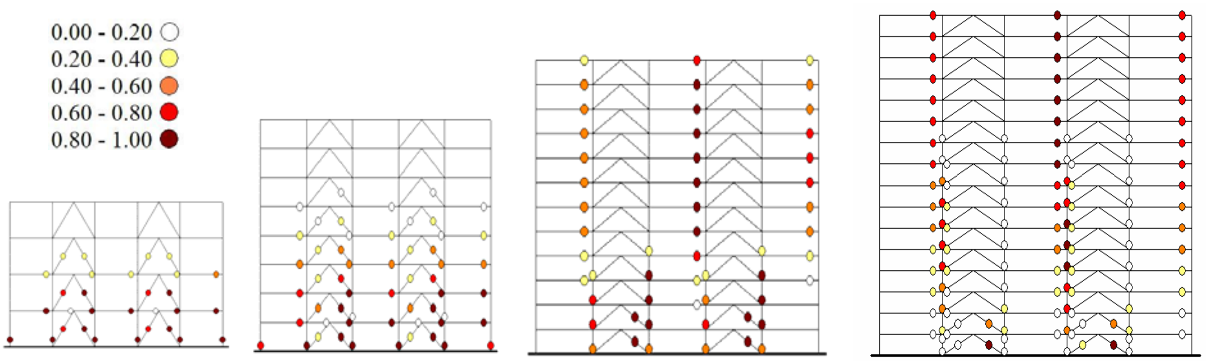
The initial yielding for four, eight, twelve and sixteen-stories models where columns resist near 50% of the total seismic shear force are depicted in Figure 5. This strength contribution is the minimum currently required for the columns of CBFs by MFDC-04. For low-rise models (*Ch4* and *Ch8* models) the inelastic response starts with incipient buckling for braces under compression. In contrast, for taller models, initial yielding occurs in the beams.

Final collapse mechanisms are mapped in Figure 6. Collapse mechanisms for low-rise models reasonably agree with the initial design assumptions behind MFDC-04: considering the buckling of the compression brace or their yielding in tension, the plastic hinge rotation in beams and the plastic hinges in the columns at base level (unavoidable for the fixed-base modeling assumption). However, the collapse mechanism changes as the models become taller. Besides the plastic hinges at the beams, some plastic hinges develop at the column ends, particularly for *Ch16* models, where plastic hinges form at both column ends in the first story. In contrast, fewer braces buckle. The results for *Ch16* models show a near soft story collapse mechanism with no uniform distribution of yielding within the height. The collapse mechanism is completely different from the one assumed in the design process.



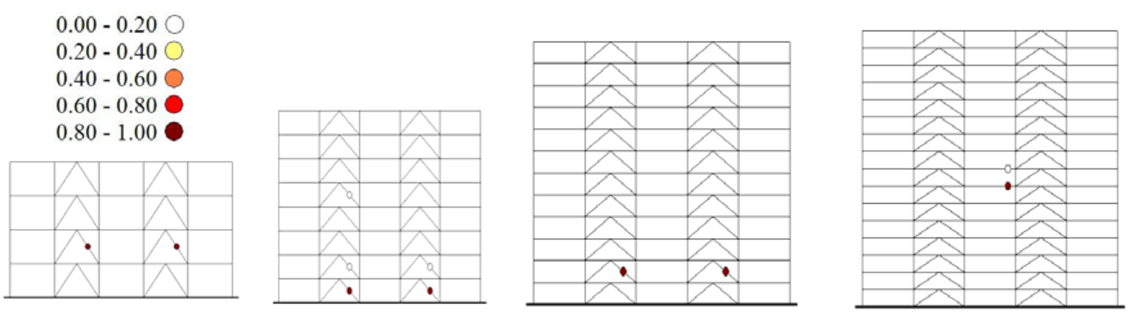
Ch4p5 *Ch8p5* *Ch12p5* *Ch16p5*

Figure 5**.** Initial yielding for models where columns resist near 50% of the total seismic shear force



Ch4p5 *Ch8p5* *Ch12p5* *Ch16p5*

Figure 6. Final collapse mechanisms for models where columns resist near 50% of the total seismic shear force

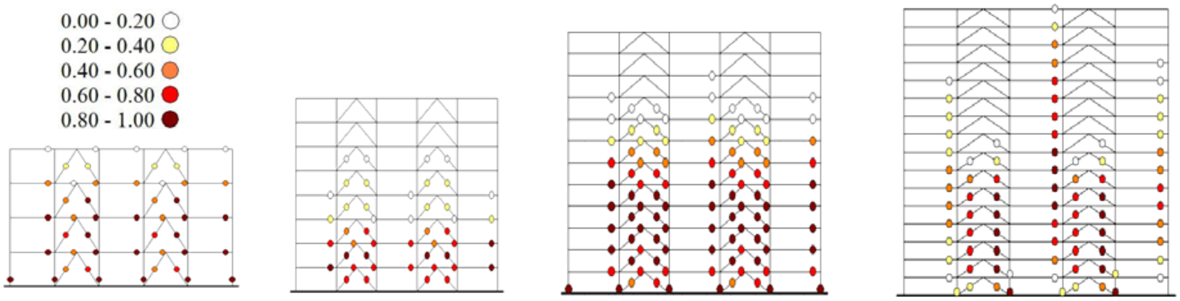


Ch4p6 *Ch8p6* *Ch12p6* *Ch16p6*

Figure 7. Initial yielding for models where columns resist near 65% of the total seismic shear force.

4.3 Models where columns resist near 65% of the total seismic shear force

It can be observed that the assumed initial incipient brace buckling is now extended up to the twelve-story models (Figure 7). This tendency continues until the final collapse mechanism is reached (Figure 8). Low-rise models exhibit a more uniform distribution of yielding along their height than the taller frames. On the other hand, *Ch16* models do not completely develop the collapse mechanism assumed in MFDC-04. Inelastic yielding predominantly occurs in the beams without any buckling of braces in compression in the upper stories. Inelastic yielding and plastic hinges at column ends start to trigger in the first two stories.

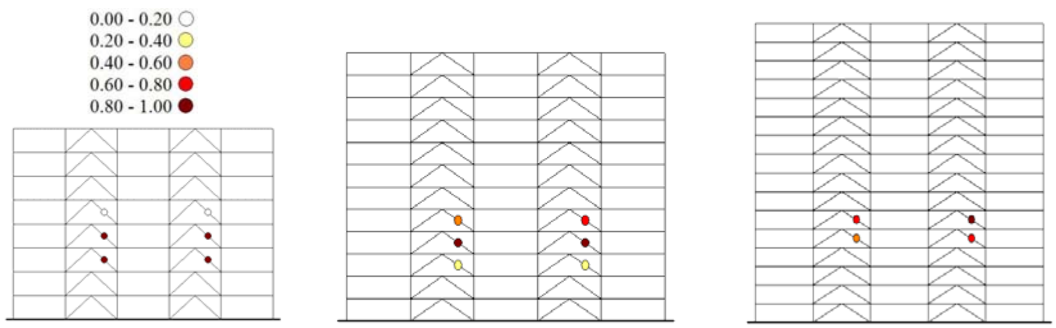


Ch4p6 *Ch8p6* *Ch12p6* *Ch16p6*

Figure 8. Final collapse mechanisms for models where columns resist near 65% of the total seismic shear force

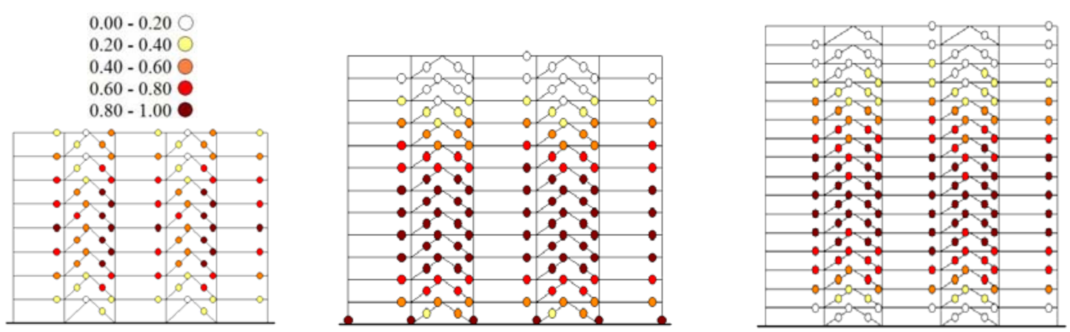
4.4 Models where columns resist near 75% of the total seismic shear force

The initial yielding for the models where columns resist near 75% of the total seismic shear force are depicted in Figure 9 and the collapse mechanisms are illustrated in Figure 10. These models represent the CBFs closer to a special moment-resisting frame (SMRFs) studied in this work; therefore, it is expected *a-priori* that the bracing system will indeed constitute the weakest link for the system.



Ch8p7 *Ch12p8* *Ch16p8*

Figure 9. Initial yielding for models where columns resist near 75% of the total seismic shear force.



Ch8p7 *Ch12p8* *Ch16p8*

Figure 10. Final collapse mechanisms for models where columns resist near 65% of the total seismic shear force

It can be observed in Figure 9 that, indeed, the inelastic response starts with the incipient buckling of the braces for all models. In general, for the collapse mechanism, the energy dissipation is mostly obtained by the buckling in compression of the braces or their yielding in tension and the subsequent formation of plastic hinges in the beams. All the models exhibit a more uniform distribution of yielding along the height which it is associated with the strong column – weak beam – weaker brace mechanism initially assumed in the design process. Columns remain essentially elastic with the exception of some inelastic yielding and plastic hinges at the base in the *Ch12* models. The inelastic behavior in columns is related to the assumed fixed end condition and with the stiffness increment required by these frames in the design process in favor of the stability of the models.

Peak plastic deformations in the structural elements for all the models studied are presented in Tapia-Hernández and Tena-Colunga (2007). These magnitudes are related to the time-step where the collapse mechanism formed taking into account the theoretical ultimate deformation capacities for beam columns and braces. For the braces, the magnitude of the axial shortening that defines the failure of the bracing due to buckling was assessed using the criteria proposed in Kemp (1996).

5. DESIGN REFLECTIONS

Based upon the analytical results from the study of some low-rise models (Bruneau *et al*. 1998), final collapse mechanisms agree reasonably well with the assumed strong-column, weak-beam, weaker brace mechanism currently considered in the design philosophy of building codes, as it was also observed with the low-rise models presented here. However, recent studies have shown that rather different collapse mechanism could be developed for CBFs for medium rise buildings (Khatib *et al*. 1988, Elghazouli 2003, Remennikov and Walpole 1998, Tapia-Hernández and Tena-Colunga 2004, Tapia-Hernández, 2005).

The results suggest that a relationship between the height of the structure, the lateral shear strength ratio between the bracing system and the frame system with the developed collapse mechanism seem to exist. Considering the results, a design strategy is proposed to define the minimum shear strength ratio for the resisting columns of the moment frames and the bracing system to have consistent strong column – weak beam – weaker brace collapse mechanism. This strategy takes into account the slenderness aspect ratio from the building and the yield stresses of the different structural elements. Therefore, from the results obtained, the minimum percentage of the total lateral seismic shear force that the columns of the frame should take in a CBFs system can be roughly assessed as:

$$V_{R\,Col} = 50 + \frac{h^2}{250}\sqrt{\frac{F_{y\,Col}}{F_{y\,Diag}}}\ (\%) \qquad 5.1$$

where h is the height of the building, $F_{y\,Diag}$ is the yielding strength for the braces and $F_{y\,Col}$ is the yielding strength for the columns. The proposed curve given in Eq. 1 is compared versus the studied models in Figure 11, taken into account that A50 steel was used in columns and A36 steel in braces. It is worth noting that if both elements would be made with the same steel, the proposed limit will be smaller, considering the possibility that the columns could start yielding before. Thus, the proposed expression allows to crudely estimate the minimum percentage of the seismic shear force that columns of the moment frames must take in order to keep them with little or null damage, concentrating the inelastic deformations in the bracing system and the beams.

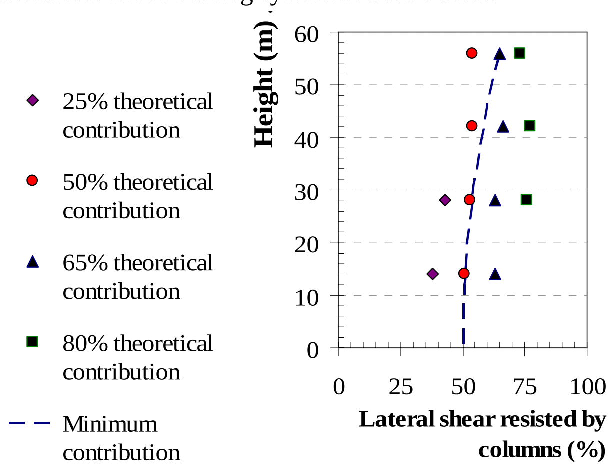


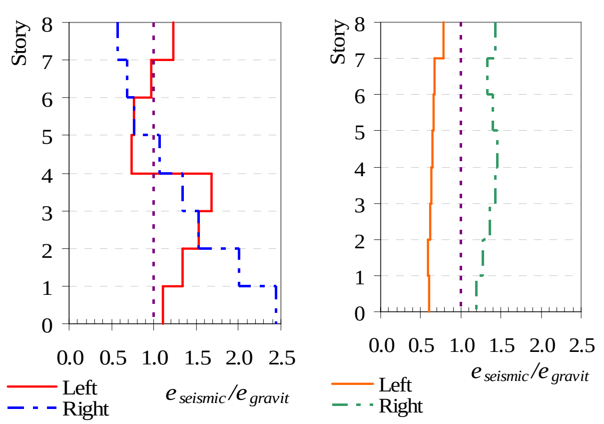
Figure 11. Minimum percentage of seismic shear force resisted by the columns.

Nevertheless, it is worth noting that the previous equation is not enough by itself to help develop consistent collapse mechanisms for CBFs, because their complex inelastic behavior is also influenced by other parameters like the deformation capacity provided by the beams and the panel zone. The deformation capacity of the beams depends on their stiffness and strength. The design of the panel zone strongly depends on the strength/stiffness balance of beams and columns, the dimensions of those elements, as well as the selected connection.

The seismic design of structural systems according to building codes require to include combinations with gravitational loads. Although this issue is well-known, unfortunately many parametric studies devoted to assess the "expected" seismic behavior of different structural systems using nonlinear analyses that have been already presented in the literature simply avoid including gravitational load combinations within the design process "in sake of simplicity". This approach is wrong, particularly if design strategies are evaluated (proposed) and their impact in deformation/strength capacities and collapse mechanisms are assessed.

5.1 Beam and brace design philosophy

In Figure 12, the normalized elastic stress ratios for both beam ends of two central bays of model *Ch8p6* are presented. The elastic seismic stress $e_{seismic}$ is normalized with respect to the elastic gravitational stress ratio e_{gravit}. Therefore, if the stress ratio is smaller than one, then the design of the beam is governed by gravitational loads, whereas if this ratio is higher than one, the design of the beam is controlled by seismic forces. It can be observed that gravitational loads are particularly important in the design of all beams in the unbraced bay and in the upper stories of the braced bays. Similar results are obtained for models with more number of stories and it is even more important for the low-rise four-story models.



a) right braced bay b) central unbraced bay
Figure 12. Beams stress ratio of the central bay, Ch8p6 model

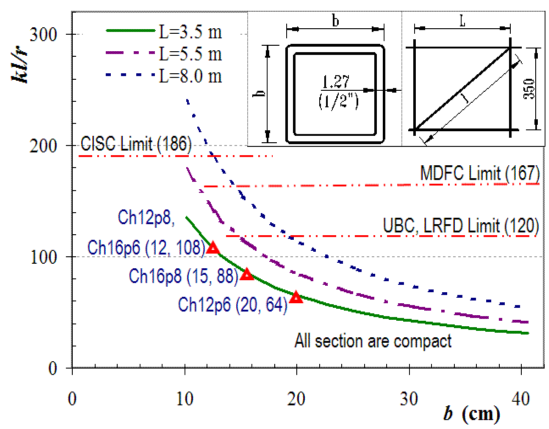


Figure 16. Review of slenderness ratio and compact section criteria for the braces of box section used.

In fact, to develop the mechanism weak brace–strong beam, the inelastic behavior of the inverted-V-braced CBFs is strongly dependent on the stiffness and strength characteristics of the beams (Khatib *et al*. 1988, Remennikov and Walpole 1998). In order to minimize the impact of gravitational loads in the seismic design of beams of CBFs it would be advisable to develop a minimum stiffness and strength index for beams with respect to the columns that might be relatively independent of load combinations that prevails in their design, to warrant an efficient and consistent ductile behavior for the CBFs. Nevertheless, such a strategy will imply to develop higher lateral overstrength, which impact must also be carefully assessed.

In Figure 16, curves that relate the slenderness ratio with the brace width (box sections) are provided for a thickness for diagonal bracing systems in bays between 3.5 m to 8.0 m in width and 3.5 m in height. The slenderness ratio limits proposed for SMRCBFs established by different international code (UBC-97, LRFD-96, MDFC-04 and CISC-93) and the compact section's limit to identify local buckling failure are also included. It can be observed that in all cases, the slenderness ratios are smaller to the codes restrictions, inclusively increasing up the wide bay up to 8 meters. Additionally, all the brace's sections used in this study fulfill the bracing slenderness ratio restrictions established in the codes, even for braces with the smallest thickness (t= 0.64 cm. = 1/4").

CONCLUDING REMARKS

The results obtained from the pushover analyses of 26 regular steel buildings structured with moment–resisting concentrically braced frames (MRCBFs) are presented and discussed. Subject buildings were designed for a soft soil site condition according to the seismic and steel guidelines of Mexico's Federal District Code (MFDC-04), which are similar to other international codes, such as CISC, AISC-LRFD, UBC and IBC codes. Building models range from 4 to 16 stories, with two different bracing configurations. All bracing sections used in this study fulfill the restriction of maximum brace slenderness ratios. The MRCBFs were designed with different shear strength ratios between the bracing system itself and the corresponding columns of the moment frame.

The analysis of the obtained results suggest that the assumed collapse mechanism of strong column–weak beam–weaker brace cannot always be obtained following the general guidelines recommended in the design philosophy for MRCBFs in MFDC-04 for soft soil sites. This research study also found a relationship between the developed collapse mechanisms with the height of the building that it is not currently considered in building codes. From the results obtained, an expression is proposed to define the minimum strength ratio between the resisting columns of the moment frames and the bracing system to have consistent strong column – weak beam – weaker brace collapse mechanisms. This expression takes into account the slenderness aspect ratio of the building and balances of yield stresses for the different structural elements.

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