



## SEISMIC BEHAVIOR OF CODE-DESIGNED STEEL MOMENT RESISTING CONCENTRICALLY BRACED FRAMES (MRCBFS) IN SOFT SOILS

Edgar Tapia<sup>1</sup> and Arturo Tena-Colunga<sup>2</sup>

### ABSTRACT

Results related to the assessment of force modification factors obtained with pushover analyses of 13 regular steel buildings structured with ductile moment – resisting concentrically braced frames are summarized in this paper. Four-story to 16-story models were designed for the soft soil condition according with Mexico’s Federal District Code (MFDC-04). Different balances between the story shear resisted by the columns with respect to the one resisted by the bracing system were considered. Improved equations are proposed for a more realistic assessment of ductility- and overstrength- force modification factors.

### Introduction

According to recent researches, CBFs do not adjust acceptably in all cases with the initial assumptions related to code’s design philosophy. Soft-stories mechanism with excessive storey drift is the typical response of multi-storey structures, as it is shown in Lacerte and Tremblay (2006); Izvernari (2007) and Tapia and Tena-Colunga (2009). Thus, CBFs designed under a ductile behavior philosophy could have not well assessed capability to dissipate energy through inelastic behavior with important overstrength reserves under the current building codes. The attention of this paper is focused on the performance of inverted-V Braced Steel Framed buildings, in order to improve the knowledge acquired and propose a more realistic assessment of force modification factors for these structural systems in soft soils.

### Buildings studied

Twenty-six subject buildings from four-story to 16-story have been studied. They were designed for soft soil site condition according to MFDC-04 with a seismic coefficient  $c=0.45$  and a Seismic Response Modification Factor  $Q=3$  (the maximum allowed for these structures). The reference building configuration is illustrated in Figure 1. Two different bracing configurations were studied for each five bay model proposed with one and two braced bays.

MRCBFS were designed for different lateral shear strength balances between the bracing

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<sup>1</sup>PhD Student at Universidad Autónoma Metropolitana-Azc. Departamento de Materiales. San Pablo 180. Col. Reynosa Tamaulipas, 02200, Mexico City. MEXICO. [etapiah@hotmail.com](mailto:etapiah@hotmail.com).

<sup>2</sup>Professor at Universidad Autónoma Metropolitana-Azc. Departamento de Materiales. San Pablo 180. Col. Reynosa Tamaulipas, 02200, Mexico City. MEXICO. [atc@correo.ac.uam.mx](mailto:atc@correo.ac.uam.mx).

system itself and the corresponding columns of the moment frame. This column contribution was varied in at least three times at each different buildings height as shown in Figure 2. The notation is identified in the left hand side of the graph. The one hundred percent of the column contribution represents the resisting frame without the bracing system; whereas the zero percent of column contribution would represent the theoretical case of a truss system. Further information about other structural characteristics is reported in Tapia and Tena-Colunga (2008).

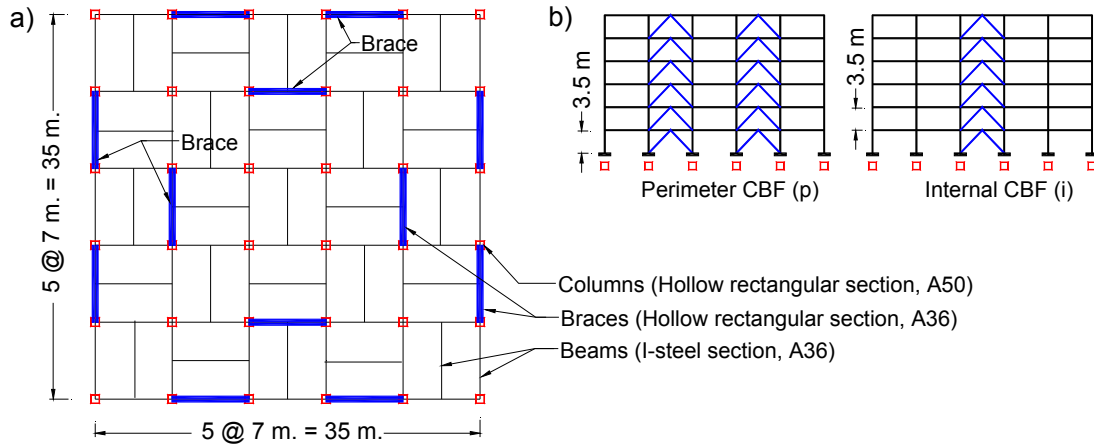


Figure 1. Buildings studied: a) Typical floor plan view; b) CBF elevations.

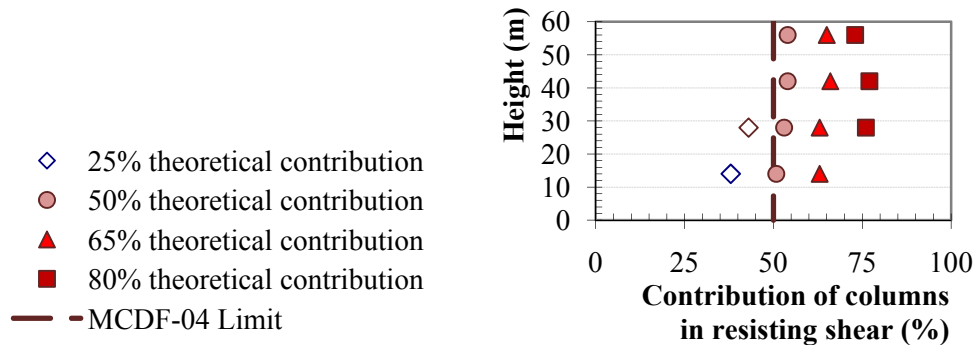


Figure 2. Models under study.

### Ductility reduction factor $\mu$

Bilinear elasto-plastic curves were developed from the pushover curves obtained of the non linear analyses. Pushover curves of model *Ch8p50* is shown in Figure 3, in order to illustrate the developed reasoning. Theoretical yielding drifts of bilinear curves, which are usually considered by the codes, are identified as  $\delta_i$ . All interstory drifts  $\delta_i$  were obtained from models studied to compare it with the global one. The global drift  $\delta_g$  was obtained from the curve that relates the base shear and the drift between the roof and the base.

Thus, it is possible to define two different ductility magnitudes: one related with actual yielding drifts  $\delta_j$  obtained directly from the computed pushover curves, and another theoretical ductility considering yielding drifts  $\delta_i$  of the equivalent elasto-plastic curve.

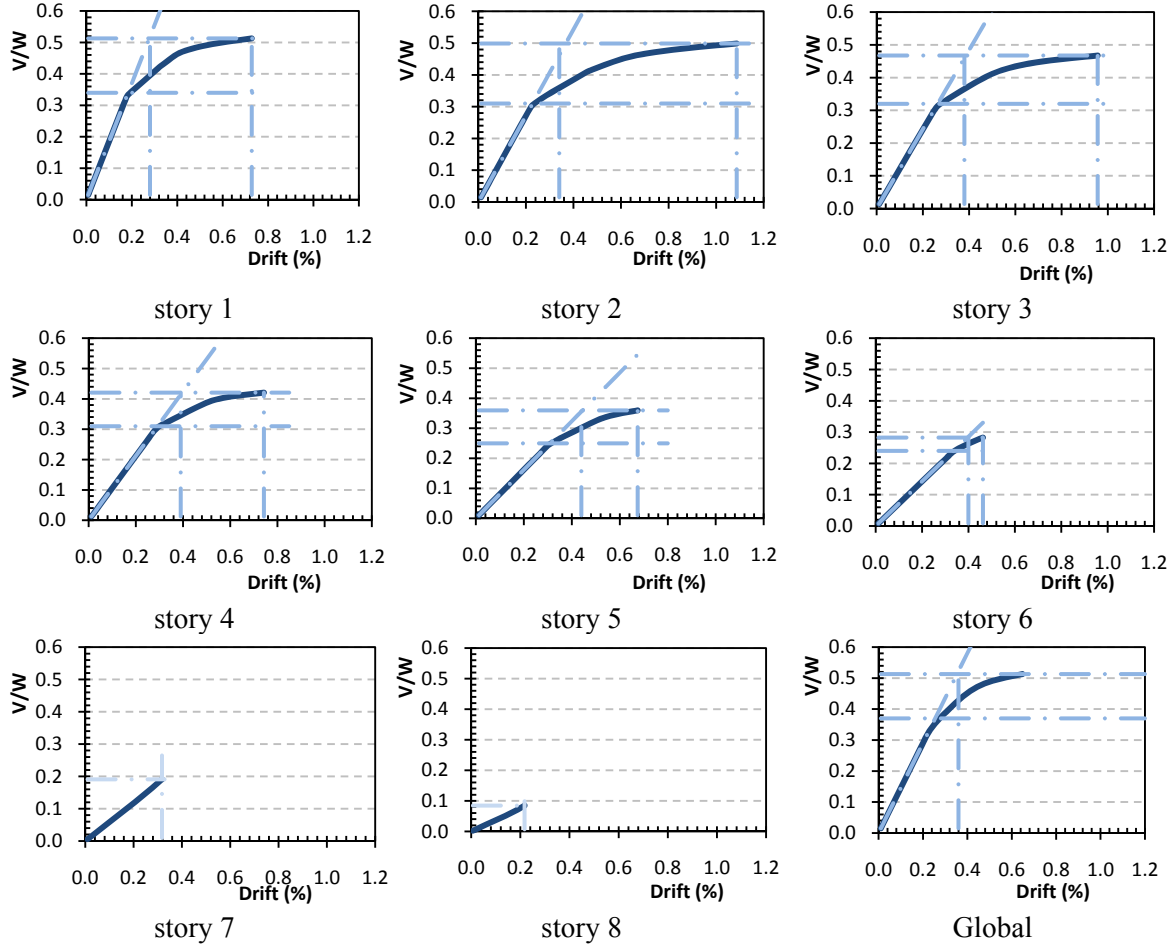


Figure 3. Pushover curves of the *Ch8p50* model.

### Drifts

The global drift  $\delta_g$  and the interstory drift average  $\delta_i$  are shown in Table 1, including the average of both,  $\delta_{av}$ . The interstory drift average  $\delta_i$  excludes the ground story results due the fixed boundary condition and the results of stories with an elastic behavior. In order to define the final drifts  $\delta_u$ , the theoretical limits for the plastic hinge rotation capacities in columns and beams and the theoretical limits for the buckling shortening of brace sections were taken into account considering the equations derived from experimental research that are reported in Kemp (1996), according to the procedure outlined in Tapia (2005).

A maximum service deformation limit equal to  $\delta_{y perm} = 0.004h$  is proposed in MFDC-04 when the main structural system is properly separated from non-structural components. The collapse deformation limit defined in MFDC-04 is  $\delta_{u perm} = 0.015h$  where  $h$  is the interstory height. Average drifts at yielding  $\delta_{y av}$  obtained from the pushover curves are compared with MFDC-04 limit  $\delta_{y perm}$  in Fig. 4a. The average yielding drifts  $\delta_{i av}$  obtained from the elasto-plastic curves are depicted in the Fig. 4b. Finally, the average final drifts  $\delta_{u av}$  are shown in Fig. 4c.

Table 1. Peak drifts obtained for the studied models.

Model	Drifts at first yielding $\delta_y$ from the pushover curve (%)			Drifts at yielding $\delta_t$ from the equivalent elasto-plastic curve (%)			Final drift $\delta_u$ (%)		
	Global	Interstory	Average	Global	Interstory	Average	Global	Interstory	Average
	$\delta_{y,g}$	$\delta_{y,i}$	$\delta_{y,av}$	$\delta_{t,g}$	$\delta_{t,i}$	$\delta_{t,av}$	$\delta_{u,g}$	$\delta_{u,i}$	$\delta_{u,av}$
Ch4p50	0.20	0.22	0.21	0.30	0.35	0.33	0.86	0.99	0.93
Ch4p65	0.29	0.29	0.29	0.40	0.40	0.40	1.27	1.30	1.29
Ch8p50	0.25	0.28	0.26	0.33	0.39	0.36	0.65	0.78	0.72
Ch8p65	0.27	0.31	0.29	0.37	0.41	0.39	1.16	1.40	1.28
Ch8p75	0.25	0.26	0.26	0.39	0.41	0.40	1.49	1.61	1.55
Ch12p50	0.43	0.46	0.44	0.59	0.61	0.60	0.95	1.00	0.98
Ch12p65	0.34	0.37	0.36	0.44	0.47	0.45	1.00	1.21	1.11
Ch12p80	0.24	0.25	0.25	0.38	0.40	0.39	1.22	1.29	1.25
Ch16p50	0.42	0.45	0.43	0.62	0.66	0.64	1.54	1.62	1.58
Ch16p65	0.36	0.40	0.38	0.46	0.49	0.47	0.63	0.67	0.65
Ch16p80	0.29	0.29	0.29	0.42	0.44	0.43	1.32	1.38	1.35

### Ductility capacities

From the drifts obtained, global ductility  $\mu_g$ , average interstory ductility  $\mu_i$  and the average of both  $\mu_{av}$  were calculated from pushover curves and bilinear curves (Table 2). Like manner, ground story results and interstory results with elastic behavior were excluded.

Models that do not meet the minimum shear contribution columns (50%) for ductile SMRCFs as defined in MFDC-04 presented an elastic behavior  $\mu \approx 1$  (Fig. 5). Given that yielding drifts  $\delta_t$  for elasto-plastic curves are larger than yielding drifts  $\delta_y$  from actual pushover curves, the theoretical ductility  $\mu_t$  reaches higher magnitudes (up to  $\mu=6$ ). In both cases, a strong relationship between ductility, building height and the shear contribution of columns is found.

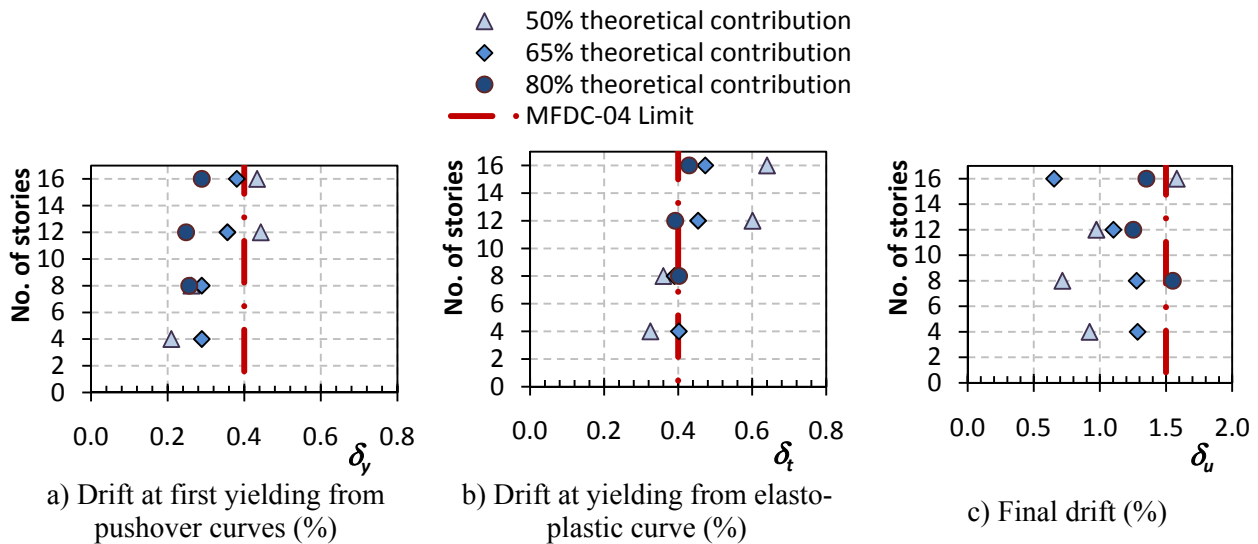


Figure 4. Drifts obtained in function of the number of stories on the models studied.

From pushover curves (Table 2, Fig. 5a) the ductility capacity  $\mu_y$  flow-rise models that meet the minimum requirement of MFDC-04 for SMRCBFs (shear contribution of columns of 50%) correlates well with the design ductility  $\mu=3$  proposed in MFDC-04. Nevertheless, different results were obtained when the ductility capacity  $\mu_t$  from design-oriented elastic-plastic curves are considered, where the minimum shear contribution of columns should be 65% in order to achieve the design ductility  $\mu=3$  (Table 2, Fig. 5b). It can also be observed from Table 2 and Fig. 5 that the obtained ductility (deformation capacity) decreases with the height increment.

Table 2. Ductility capacities of the studied models.

Model	Model slenderness $H/B$	Ductility $\mu_y$ from pushover curves			Theoretical ductility $\mu_t$ from elasto-plastic curves		
		Global	Interstory	Average	Global	Interstory	Average
		$\mu_{y,g}$	$\mu_{y,i}$	$\mu_{y,av}$	$\mu_{t,g}$	$\mu_{t,i}$	$\mu_{t,av}$
Ch4p25	0.40	1.17	1.00	1.08	1.08	1.00	1.04
Ch4p50		4.20	3.19	3.69	2.87	2.17	2.52
Ch4p65		4.35	4.45	4.40	3.18	3.22	3.20
Ch8p25	0.80	1.05	1.00	1.03	1.00	1.00	1.00
Ch8p50		2.64	2.93	2.79	1.97	2.06	2.32
Ch8p65		4.34	4.57	4.46	3.14	2.72	2.93
Ch8p75		5.96	5.93	5.95	3.82	4.09	3.95
Ch12p50	1.20	2.00	1.94	1.97	1.36	1.38	1.37
Ch12p65		2.95	3.36	3.15	2.27	2.31	2.29
Ch12p80		5.00	4.79	4.90	3.21	3.12	3.16
Ch16p50	1.60	1.99	1.90	1.95	1.86	1.77	1.81
Ch16p65		1.76	1.63	1.70	1.37	1.36	1.37
Ch16p80		4.59	4.51	4.55	3.14	2.97	3.06

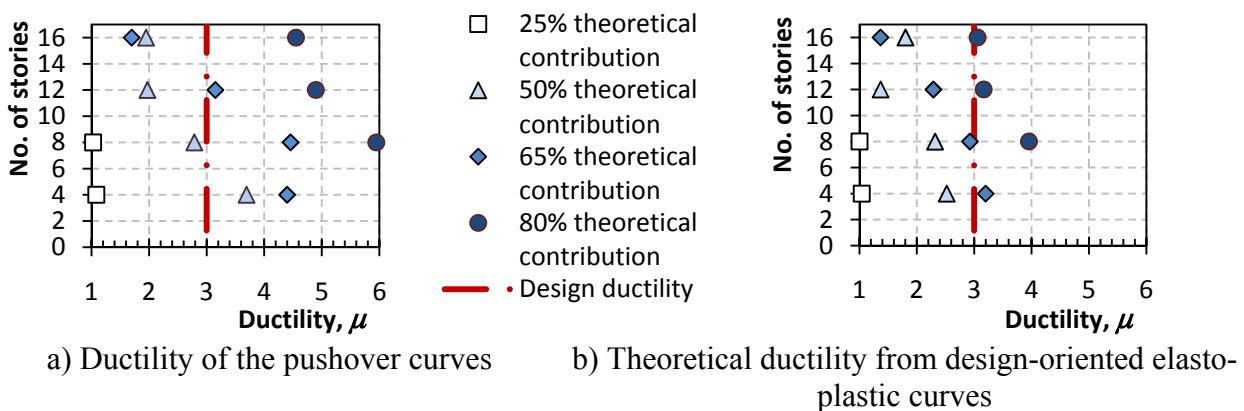


Figure 5. Assessed ductility in relationship to the number of stories of the studied models.

## Current design philosophies of building codes

Actually, some international codes define the ductility reduction factor in function of the building height in agreement with the results presented and the soft storey mechanism tendencies observed for CBF (Izvernari, 2007; Tapia and Tena-Colunga, 2008).

In the National Building Code of Canada (CNBC-2005), a ductility modification factor of  $\mu=3$  is proposed for moderately ductile CBF (Type MD) and a value of  $\mu=2$  for limited-ductility CBF (Type LD). It is required by this code that the seismic design forces be increased by 3% per meter of height for Type MB frames taller than 32 m and 2% per meter of height above 48 m for Type LD frames. The relative minimum CBF design loads ( $1/\mu$ ) of the CNBC-05 and MFDC-04 (which propose constant values equal to  $\mu=3$  and  $\mu=2$ , respectively) are shown in Figure 6.

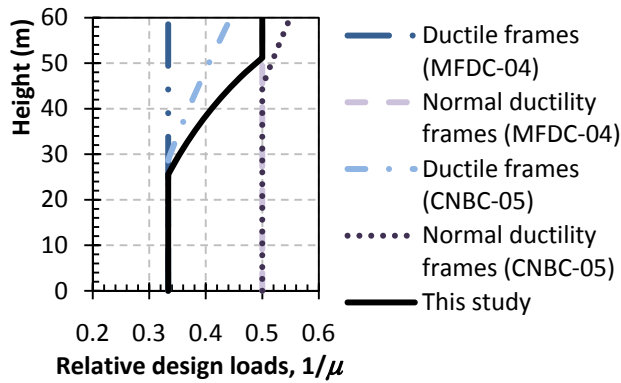


Figure 6. Relative CBF design loads.

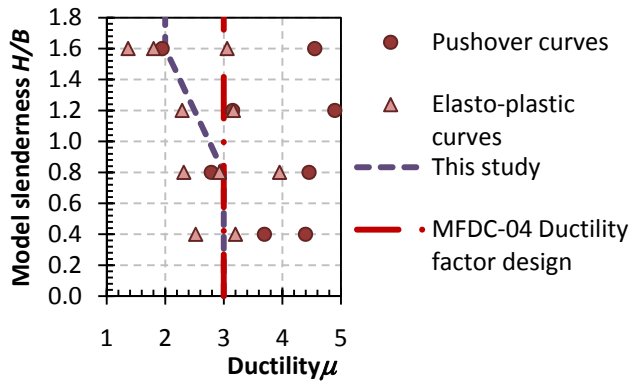


Figure 7. Ductility obtained by the models.

Considering the results obtained in the ongoing research, an improved equation to assess the ductility modification factor is proposed as a function of the model slenderness  $H/B$  (Eq. 1). Here,  $H$  is the building height and  $B$  the dimension of the building at its base. The models studied have a 3.5 m of interstorey height and  $B=35$  m (Fig. 6).

$$\left. \begin{aligned}
 \text{If } H/B \leq 0.80 & \quad \mu = 3.0 \\
 \text{If } 0.80 < H/B \leq 1.60 & \quad \mu = 3 - 1.25 \left( \frac{H}{B} - 0.80 \right) \\
 \text{If } 1.60 < H/B & \quad \mu = 2.0
 \end{aligned} \right\} \quad (1)$$

The behavior of the ductility load modification factor proposition is shown as a relative design load in Figure 6, whereas the proposed equation is compared in Figure 7 with the ductility obtained for the studied models in relationship of models slenderness  $H/B$ .

### Overstrength reduction factor ( $\Omega$ , $R$ )

The overstrength effect is considered in current international codes as a reduction of the design loads by an overstrength factor ( $R$  in the Mexican codes and  $\Omega$  in the USA codes). This overstrength factor take into account the restricted choices for sizes of members and elements; the difference between nominal and factored resistances; the ratio of actual yield strength to minimum specified yield strength; the development of strain hardening and the collapse mechanism (Mitchell *et al.*, 2003), among other parameters.

Stress ratios of columns, beams, and braces of the Ch16p80 model at the design stage are shown in Figure 8. A strong relationship between the overstrength and the lateral shear contribution of resisting columns is noticed. It can be observed that, as a consequence of the capacity design process and the member typification previously described, braces are designed tightly whereas columns are over-designed, particularly as the lateral shear contribution for the columns increase (model *Ch16p80* vs model *Ch16p50*). Thus, beams and columns could develop larger overstrength while the damage is concentrated in the braces.

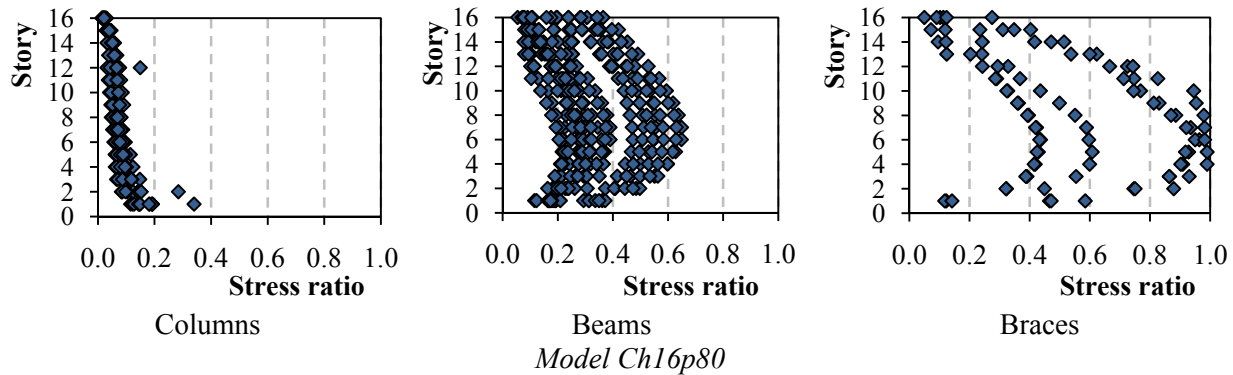


Figure 8. Stress ratio in elements of sixteen-story models.

Overstrength (nominal shear  $V_{nom}$  and maximum shear  $V_{max}$ ) obtained from analyses and the lateral static load  $V_{nom}$  in relationship of the yielding shear  $V_y$  are reported in Table 3.

Table 3. Average overstrength associated to the studied models.

Modelo	$V_{base}$	$V_y$	$V_{max}$	$V_{max}/V_{nom}$	$V_{max}/V_y$
<i>Ch4p50</i>	77.37	247.55	404.52	5.228	1.634
<i>Ch4p65</i>		381.64	442.68	5.722	1.160
<i>Ch8p50</i>	161.15	365.27	551.09	3.420	1.509
<i>Ch8p65</i>		397.50	605.52	3.758	1.523
<i>Ch8p75</i>		569.39	919.90	5.708	1.616
<i>Ch12p50</i>	244.95	930.77	1177.35	4.807	1.265
<i>Ch12p65</i>		1028.74	1302.02	5.315	1.266
<i>Ch12p80</i>		947.10	1438.39	5.872	1.519
<i>Ch16p50</i>		854.69	1809.42	5.504	2.117
<i>Ch16p65</i>	328.73	1030.02	1284.05	3.906	1.247
<i>Ch16p80</i>		964.27	1515.68	4.611	1.572



Overstrength magnitudes obtained are always larger than the values proposed in available building codes (Fig. 9). There was not found a relationship between the overstrength and the building height or the percentage of the lateral shear resisted by the columns. The graphics exclude models that do not meet the minimum lateral shear contribution for the columns of 50%, which developed a brittle failure mechanism with practically null overstrength. In Figure 10, the correlation of the maximum shear obtained and the yielding shear is exhibited.

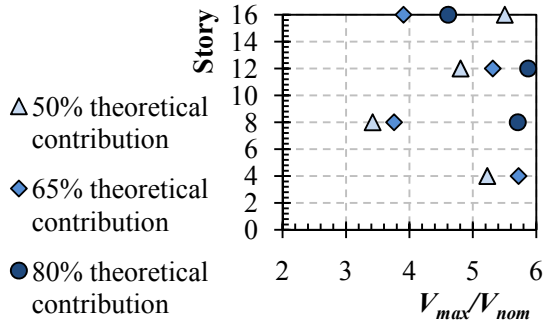


Figure 9. Overstrength obtained in the models studied.

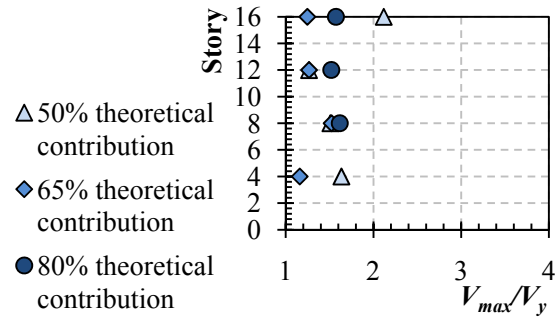


Figure 10. Correlation between the maximum and yielding resisting shear.

The reported results are consistent with those obtained in a previous research (Tapia, 2005; Tapia and Tena-Colunga, 2009), where the assessed overstrength for fifteen-story MRBF models were  $\Omega=4.72$  and  $\Omega=4.55$ . Thus, despite that the magnitude of the reduction factor varies deeply in relationship of the building configuration and the seismic design criteria, the overstrength reduction factor established in the current codes are not completely representative of the overstrength that could be developed by the buildings designed with these codes.

### Current design philosophies of building codes

According with ATC 63-08 (Sect. 11.2.1), the overstrength reduction factor in Table 12.2-1 (ASCE/SEI 7-2005) is not consistent with recent research results and varies between  $\Omega=1.5$  (in the worst case) to  $\Omega=6.0$ . The Canadian Code establish  $\Omega=1.3$  for limited ductile CBF and  $\Omega=1.5$  for ductile one (CNBC-05, 2005). The EC8-05 recommends only one value  $\Omega=1.25$  for steel frames; nevertheless different values for use in a given European Country may be found in its National Annex. In MFDC-04 an equation is proposed to determine the reduction factor as a function of the characteristic period  $T_a$ , which is dependent of the ground period  $T_g$ . This criterion is shown in Figure 11, using  $T_g=2$  sec and  $T_a=1.175$  sec. In the plot, the overstrength obtained for the models under study is included.

The Manual of Civil Structures (MOC-CFE-08) proposes the follow equation where  $R_0$  is an overstrength index value equal to  $R_0=2$  for ordinary MRBF and  $R_0=2.5$  for intermediate MRBF (Tena-Colunga *et al.*, 2009).

$$\left. \begin{array}{l} \text{If } T \leq T_a \\ \text{If } T > T_a \end{array} \right\} \begin{array}{l} R = R_0 + \alpha(1 - \sqrt{T/T_a}) \\ R = R_0 \end{array} \quad (2)$$



In order to establish a conservative proposal to assess a reasonable overstrength loads reduction factor in CBF a variation of the MFDC-04 criteria was adopted (Eq. 3) from the minimum values obtained in the analysis (Table 3). The proposed equation allows one to use an overstrength factor  $R=\Omega= 4.0$  and a higher ductility factor (in agreement with the proposed criterion for the ductility factor) in low to medium-rise structures, and an overstrength factor  $R=\Omega= 3.0$  for taller buildings. The proposed equation is also plotted in Figure 11.

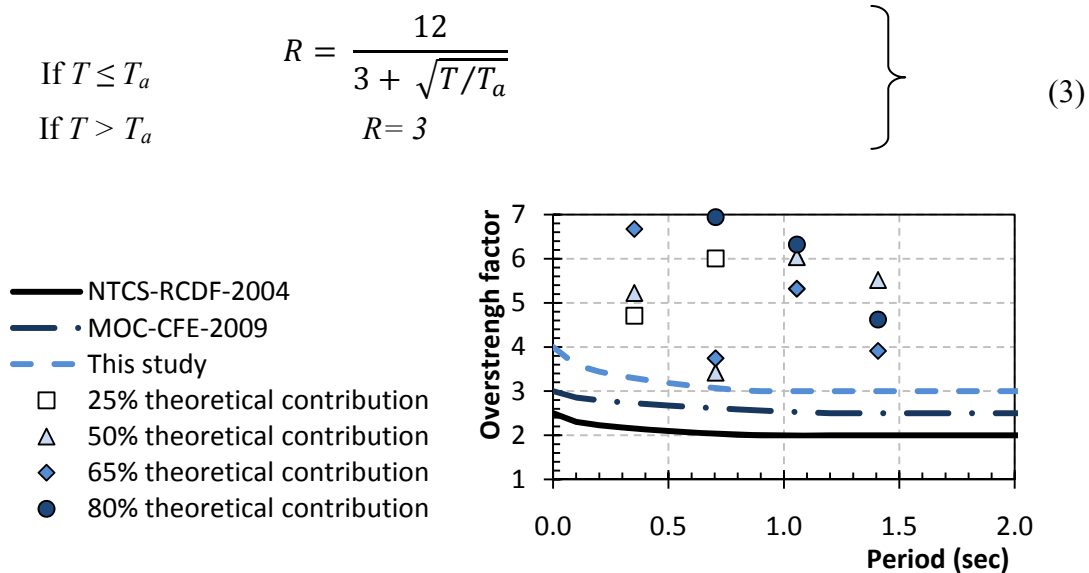


Figure 11. Comparison of the overstrength assessed in the studied models studied and the one obtained from equations proposed in Mexican codes.

### Conclusions

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 soft soil site conditions according with the Mexico’s Federal District Code (MFDC-04) and its structural steel guidelines, which are similar to other international codes. Building models ranged from 4 stories to 16 stories, with two different bracing configurations. The MRCBFs were designed with different shear strength ratios between the bracing system itself and the corresponding columns of the moment frame.

Ductility factors obtained show a strong relationship with the buildings slenderness, which is not currently considered in building codes. Low and medium rise buildings where the minimum lateral shear contribution of the resisting columns varies between 50% to 65% developed deformation capacities that are consistent with the ductility factors  $\mu=3$  considered in the design stage according to MFDC-04. However, the deformation capacity decreases as the height and the slenderness of the building increases.

All overstrength factors obtained in this study are larger than those proposed in the building codes of reference (MFDC-04; MOC-CFE-08; CNBC-05; ASCE-7-05). It was not

found a dependency of overstrength factor with the lateral shear resisting contribution of the resisting columns of the height of the building. Finally, equations were derived for the assessment of ductility and overstrength reduction factors that are consistent with the results from this research and others studies.

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