

Lateral Demands of Steel Moment Resisting Concentrically Braced Frames In Soft Soils



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SUMMARY: (10 pt)

Results of six regular building structured with concentrically braced steel frames are reported in this paper. Models were designed following a capacity design procedure for this structural system adapted for the Mexico's Federal District Code (MFDC-04) in order to obtain a collapse mechanism advocated by the code: strong column – weak beam – weaker brace, and a more realistic assessment of ductility and overstrength seismic modification factors. Buildings 10, 14 and 18 stories in height located in soft soils according to MFDC-04 were evaluated. Pushover analyses and nonlinear time-history analyses under ten selected earthquake ground artificial motions related to the corresponding design spectrum of MFDC-04 were performed using OpenSees software.

Keywords: Steel braced frames, codes-designed buildings, ductility, overstrength.

1. INTRODUCTION

It was observed from previous studies (Tapia-Hernández 2011, Tapia-Hernández and Tena-Colunga 2011) that for low-rise models, the inelastic response started with incipient buckling for braces under compression which reasonably agreed with the strong column –weak beam –weaker brace mechanism assumed in a design according to MFDC-04. However, the collapse mechanism changed as models become taller. Thus, the obtained collapse mechanism resulted in somewhat different from the one assumed in the design process.

Results obtained in the framework of this study (Tapia-Hernández and Tena-Colunga 2008, 2011a) suggest the existence of a relationship among the height of the structure, the lateral shear strength ratio (between the bracing and frame systems) and the developed collapse mechanism. Therefore, a design strategy was proposed to define the minimum shear strength ratio for the resisting columns of the moment frames $V_{R\ Col}$ to have a consistent collapse mechanism (to favor a ductile behavior). It can be roughly assessed according to the Eq. 1.1. H is the building height, B is the building length in the earthquake direction, $F_{y\ Brace}$ is the brace yielding strength and $F_{y\ Col}$ is the column yielding strength.

$$\frac{V_{R\ Col}}{V_{R\ Total}} \geq 0.50 + 0.05 \left(\frac{H}{B} \right)^2 \sqrt{\frac{F_{y\ Col}}{F_{y\ Diag}}} \quad (1.1)$$

From global base shear to roof displacement pushover curves, it was observed that the assessed ductility (deformation capacity) decreases as the number of stories increases. Considering the obtained results (Tapia-Hernández 2011, Tapia-Hernández and Tena-Colunga 2011), an improved equation to assess the ductility modification factor Q (C_d according to the US codes notation) is proposed as a function of the frame slenderness H/B (Eq. 1.2).

A strong relationship between the overstrength and the lateral shear contribution of resisting columns was noticed. It was observed that, as a consequence of the capacity design process and the member typification usually done in engineering practice, braces are designed more tightly whereas columns

are over-designed, particularly as the lateral shear contribution for the columns increases. Therefore, suitable overstrength load reduction factor R (Ω_0 according to the US codes notation) for CBF were proposed (Eq. 1.3) considering average values obtained in the analysis (Tapia-Hernández and Tena-Colunga 2011). The following values must be used in Eq. 3 for MRCBFs: $R_0= 4.5$ and $\alpha=1.0$. T_a is the initial period that defines the plateau of the design spectrum in relation of the ground period T_g .

$$\left. \begin{array}{l} \text{If } H/B \leq 1.0 \\ \text{If } 1.0 < H/B \leq 1.6 \\ \text{If } 1.6 < H/B \end{array} \right\} \begin{array}{l} Q = 3 \\ Q = 3 - \frac{5}{3} \left(\frac{H}{B} - 1 \right) \\ Q = 2 \end{array} \quad (1.2)$$

$$\left. \begin{array}{l} \text{If } T \leq T_a \\ \text{If } T > T_a \end{array} \right\} \begin{array}{l} R = R_0 + \alpha(1 - \sqrt{T/T_a}) \\ R = R_0 \end{array} \quad (1.3)$$

2. PROPOSED DESIGN STRATEGY

The proposed design methodology for MRCBFs is schematically depicted in Figure 2.1. As it may be noticed, the minimum lateral shear strength ratio between the columns of the frame and the bracing system should be defined first (Eq. 1.1). According to the results of the analyzed buildings (Tapia-Hernández and Tena-Colunga 2011a), the fundamental period for pre-design purposes could be estimated as $T= 0.08n$, where n is the number of the building's stories. A representative 3D model of the MRCBF building should also be used in the design process.

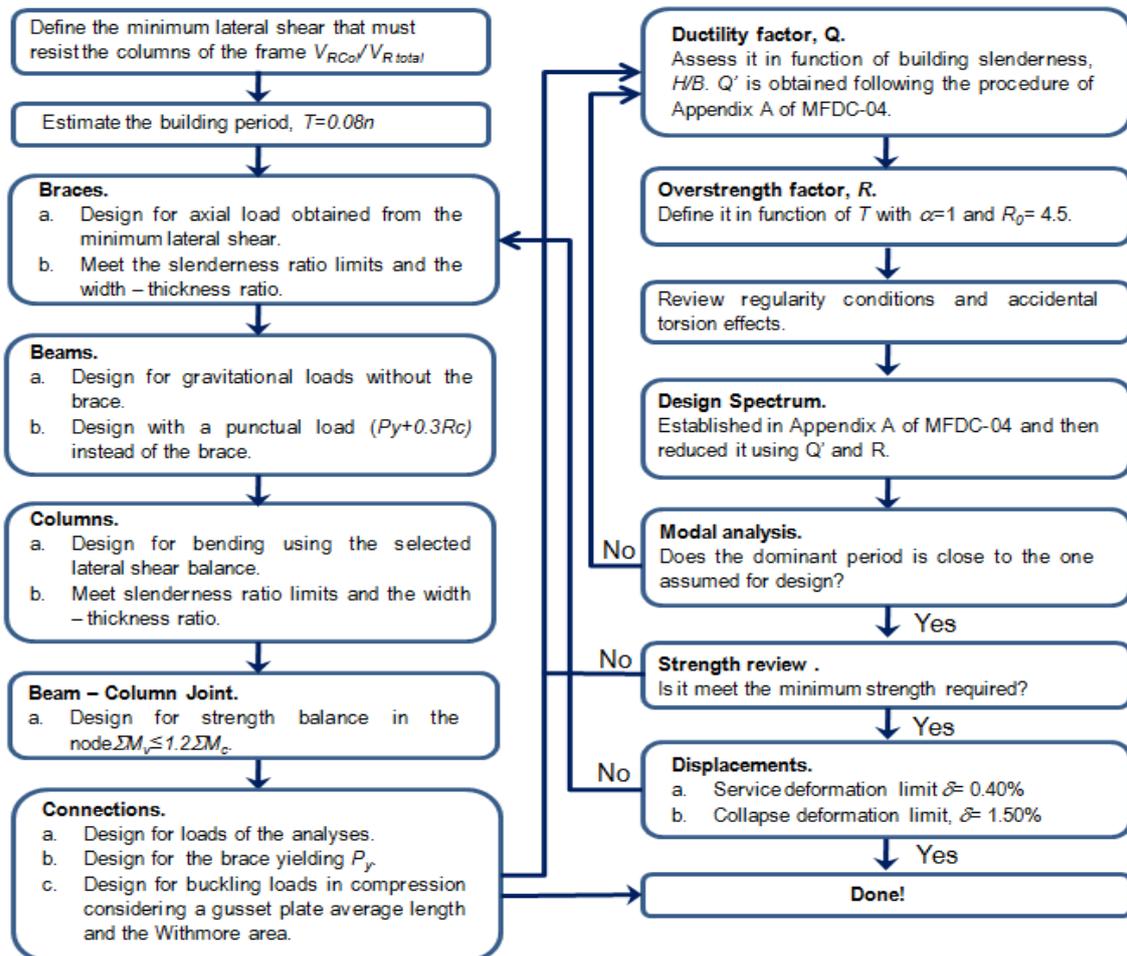


Figure 2.1. Proposed design methodology of ductile moment resisting braced frames

On the pre-design stage, each brace shall be designed to resist the axial load obtained from the projection of the minimum lateral shear that must resist the bracing system (Eq. 1.1). Also, braces must meet the maximum slenderness ratio (equal to $kL/r < 167$ for steel A-36, according to MFDC-04) and meet the limits of width to thickness ratio to be considered as compact section. Beams shall resist bending moments due to: a) those obtained from load combinations (seismic and gravitational) of a representative 3D models and, b) gravity loads assuming no vertical support is provided by braces. Beams directly connected to braces or found to take axial loads due to the bracing configuration shall be designed as beam-columns resisting a punctual load equal to $P_y + 0.3P_c$ (representing the estimated loads of the bracing members). Columns shall be designed to resist the bending due to the determined lateral shear force balance and their slenderness ratio should be less or equal to 60. Also, columns in braced bays should be able to resist a bending moment $M_p \geq 0.2ZF_y$ for the critical axial load combination obtained in the analyses. In addition, it shall be designed to resist an increment of axial load associated to the yielding or buckling of the braces.

Finally, the resisting bending moments for the columns should be checked to insure a strong-column weak-beam mechanism, consistent with a ductile behavior. This was done by checking that at the joint $\Sigma M_c \geq \alpha \Sigma M_g$, where ΣM_c and ΣM_g are, respectively, the sum of the unfactored nominal design moments for columns and beams at the joint. In this study $\alpha = 1.2$ was used, a different proposal to what it is currently available in most steel guidelines ($\alpha = 1.0$), but in agreement to what it is proposed in most reinforced concrete codes for ductile frames. In the humble opinion of the authors, using $\alpha = 1.0$ might be very optimistic, particularly for unexperienced designers that may rely heavily in design modules of commercial software, and this practice may lead to designs where the premature buckling of some columns may occur.

Connections should be designed to remain elastic for the peak loads of analyses and for the peak forces in tension and compression that the bracing members would transmit them. The compression design shall be calculated considering an average length for the gusset plate and the Whitmore area.

3. SUBJECT BUILDING

Three medium rise buildings were designed according to the proposed methodology including the assumed global design parameters in order to evaluate its effectiveness. Considered buildings (Fig. 3.1) were 10-, 14- and 18-stories in height and were located in soft soil having a site period equal to $T_g = 1.05$ sec and $T_a = 0.558$ sec, according to MFDC-04. Following the proposed methodology, the design characteristics of the analyzed frames were defined, as summarized in Table 3.1. Further information of designed buildings can be found elsewhere in Tapia-Hernández (2011).

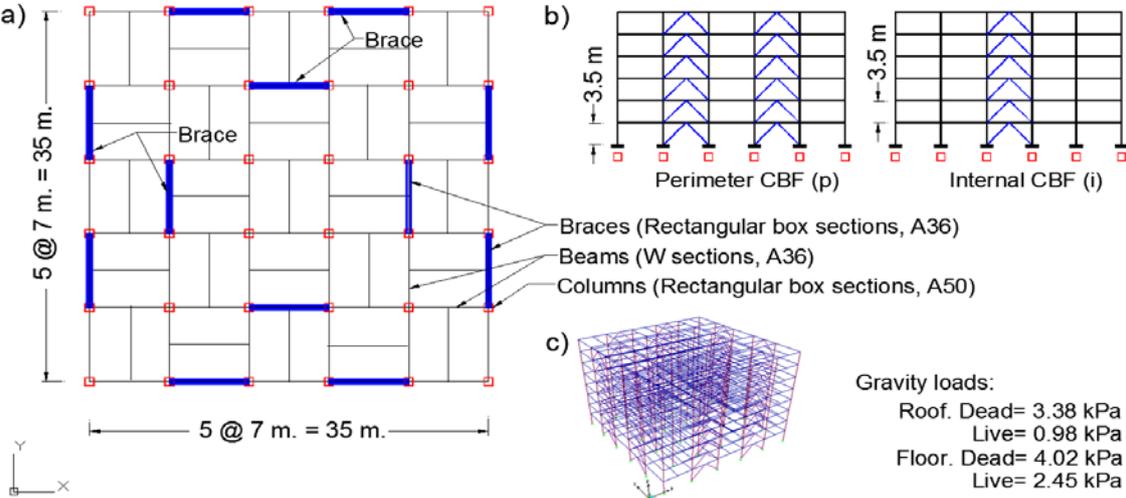


Figure 3.1. Building studied: a) Typical floor plan view; b) CBF elevations, c) 3D ETABS model.

In order to avoid abrupt changes in the lateral stiffness, for the studied models, the thickness of the box cross sections of columns was changed every M -stories, whereas the thickness of the box cross sections of the bracing was changed every N -stories. Different thickness of the box cross section was proposed for the interior and perimeter columns in function of demands. More information about the final designed member sections are reported in further detail in Tapia-Hernández (2011).

Table 3.1. Design characteristics for the studied models

Model	Stories	Building height (m)	Period (sec)	Minimum shear resisted by columns (Eq. 1)	Ductility factor, Q (Eq. 2)	Seismic force reduction factor, Q' (MFDC-01)	Overstrength factor, R (Eq. 3)
<i>Ch10_56</i>	10	35.0	0.72	55.9%	3.00	3.05	4.50
<i>Ch14_61</i>	14	49.0	1.21	61.6%	2.33	2.37	4.50
<i>Ch18_69</i>	18	63.0	1.56	69.1%	2.00	2.02	4.50

4. NONLINEAR ANALYSES

Nonlinear analyses were performed using the *OpenSees* computer program (Mazzoni *et al* 2006). Each brace was modeled using nonlinear beam-column elements with a fibre discretization of the cross-section with eight elements having four integration points, as shown in Figure 4.1. Rigid beam elements were used at the brace ends to simulate the size of the designed gusset plates with the zero-length rotational elements to reproduce the out-of plane bending stiffness and hysteretic behaviour of the gusset plates upon brace buckling. Further details and validation of this modeling technique can be found elsewhere (Izvernari 2007).

The cross – section of beams were divided in three rectangular patches: two for the flanges and one for the web. Patches were discretized into fibres with quadrilateral shapes both for the web and the flanges following the results of a parametric study (Izvernari 2007). Nonlinear beam – column elements were used in beams of the braced bays, whereas beam with hinges elements were used for the rest of the modeling beams. Finally, columns were modeled using nonlinear beam-column elements with a fibre discretization with eight elements at the brace, as shown in Figure 4.1b.

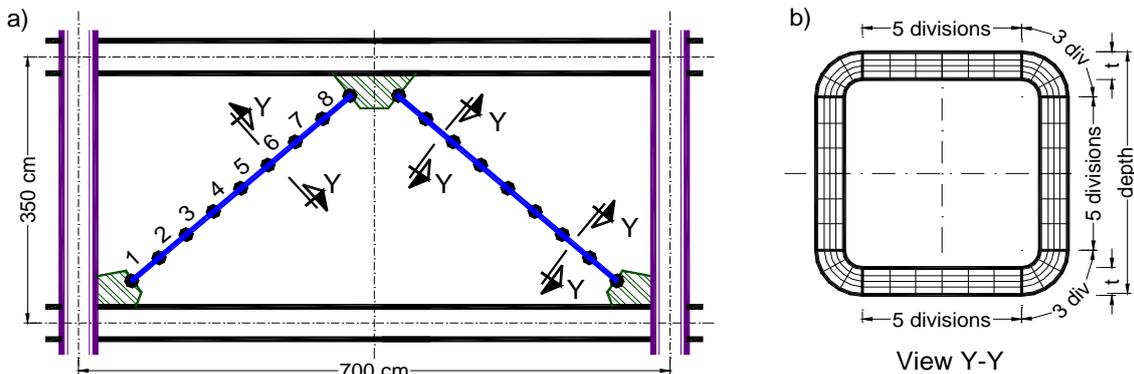


Figure 4.1. a) Discretization along the brace and, b) Brace fiber discretization of the cross-section

Through 292 certified laboratory coupon tests of steel samples available in the Mexican market, a better reference of the real yielding steel stress was obtained. The available samples were angles, HSS and plates with thickness between $t = 4.06$ (3/16") to $t = 62.5$ mm (2.5"), where 92.3% were from the Mexican production and the other 7.7% came from China and the USA (Tapia-Hernández y Tena-Colunga 2011a). Obtained results showed average values of material overstrength around 1.29 and 1.23 for A36 and A50 steel respectively, these corresponding to real values of yielding stress $f_{y \text{ real}}$ equal to 320 MPa and 424 MPa. Based on these results, two different numerical models were developed for each building height: one with the theoretical nominal strength (*Ch10_56*, *Ch14_61* and *Ch18_69*) and another considering the material overstrength (*Ch10_56a*, *Ch14_61a* and *Ch18_69a*).

5. PUSHOVER ANALYSES

Collapse mechanisms for the six studied models agreed reasonably well with the initial design assumptions of strong column – weak beam -weaker brace, with a good distribution of the inelastic demand along the height. Top stories presented practically an elastic behavior, whereas peak story drifts were developed at the middle stories. Brace buckling in compression was developed while no damage in columns was noticed, with exception of the first floor. The behavior of the *Ch10_56* model along the analysis is shown in Figure 5.1. The magnitude of inelastic deformations is plotted in color scale normalized with respect to maximum yielding rotation (for beam and column elements) or to axial extension/shortening (for brace elements).

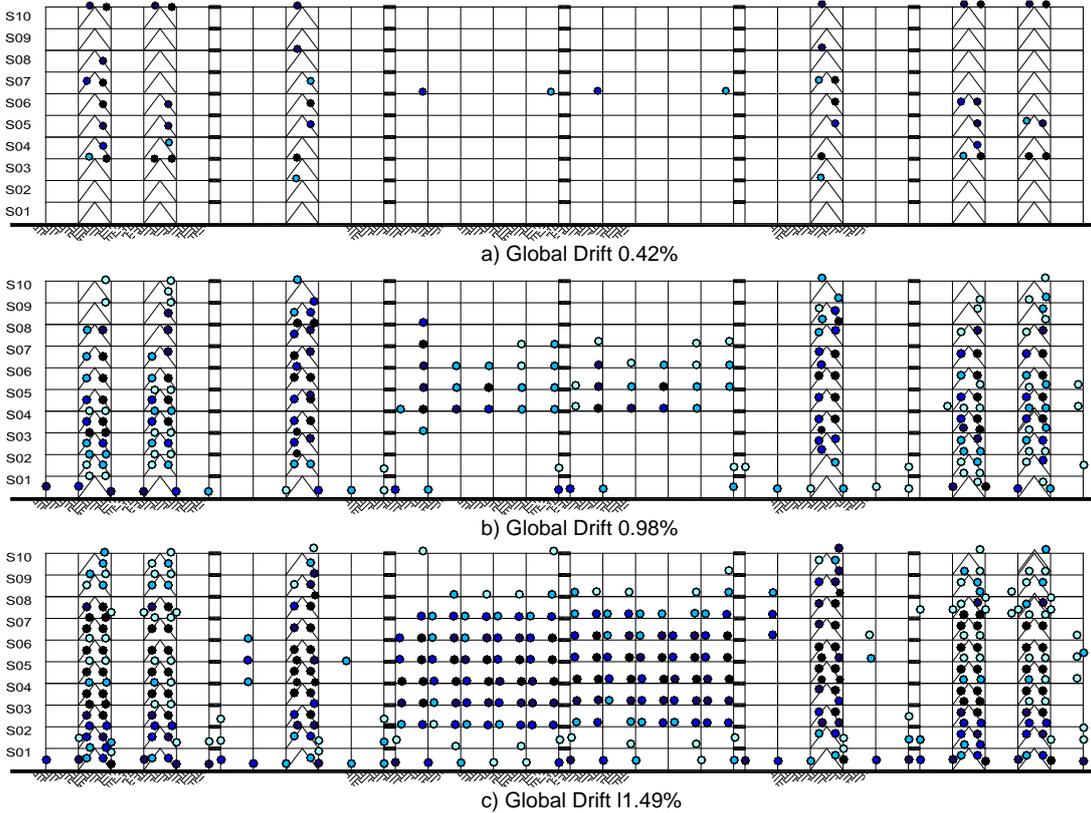


Figure 5.1. Behavior of the *Ch10_56* model along the pushover analysis

The obtained average of the drift at first yielding shows a slight tendency with the building’s height (Figure 5.2a). Nevertheless, in all models, this tendency is close to the Mexican Code’s limit equal to 0.4% (MFDC-04); therefore, this deformation limit seems adequate for practical purposes. In all cases, the average value of ultimate drifts is quite smaller than the Code limit equal to 1.5% (Figure 5.2b). Considering these results, the developed ductility for each building height is reported in Figure 5.3, including the assumed ductility in the elastic design stage.

The drift concentration factor (DCF) corresponds to the ratio between the maximum peak story drift angle along the building height and the peak overall roof deformation angle. A value of DCF close to 1.0 indicates that all floors are equally deformed and the story deformations are comparable to the average deformation of the building, which means there is no soft story. Expressions to predict the DCF values for multi-storey buildings with continuous columns are proposed in the literature (MacRae *et al* 2004, Izvernari 2007).

Drift concentration factor limits computed according to each prediction procedure (MacRae *et al* 2004, Izvernari 2007) are depicted in Figure 5.4, where they are compared with the DCFs obtained for each story of each considered buildings. For all models, peak demands are related to the interstory around

the mid-height of the building. The values obtained for the studied building are smaller than the limiting values, meaning that building models have fewer tendencies to develop soft stories.

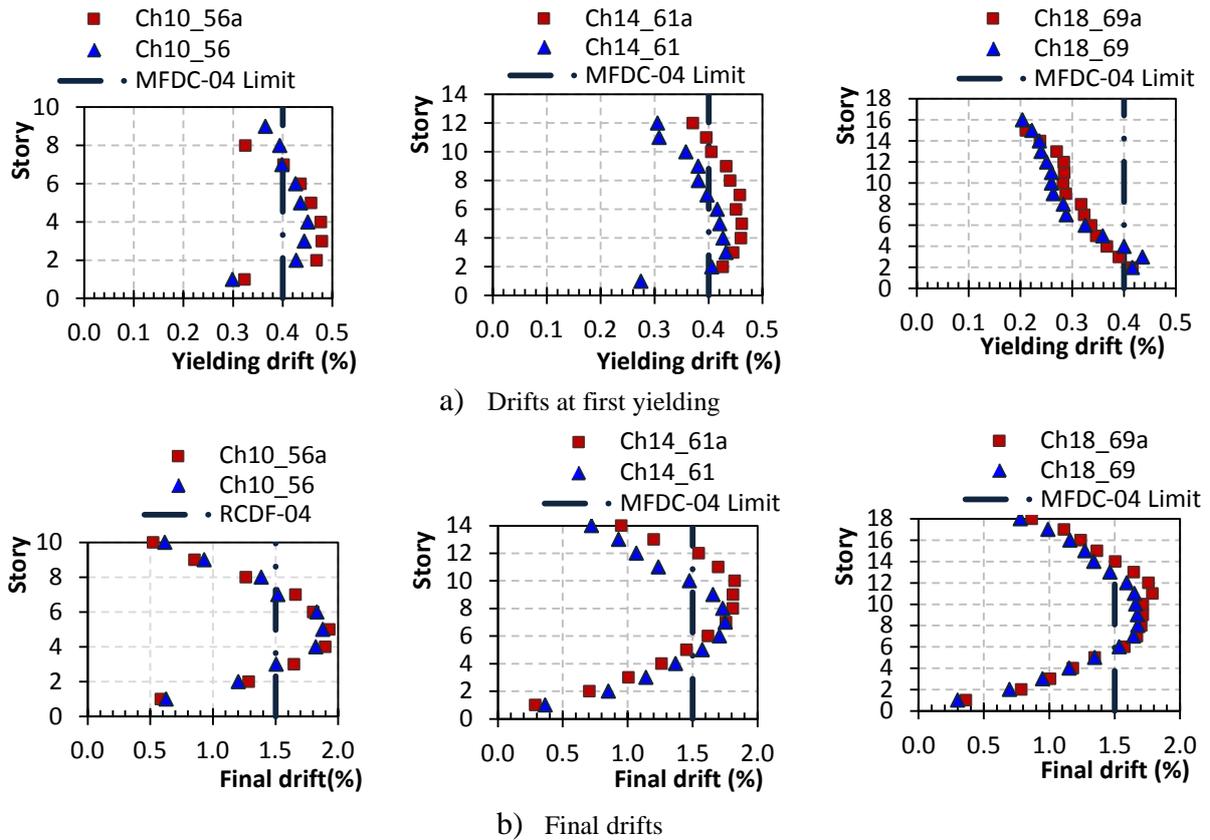


Figure 5.2. Deformation demands for the analyzed models

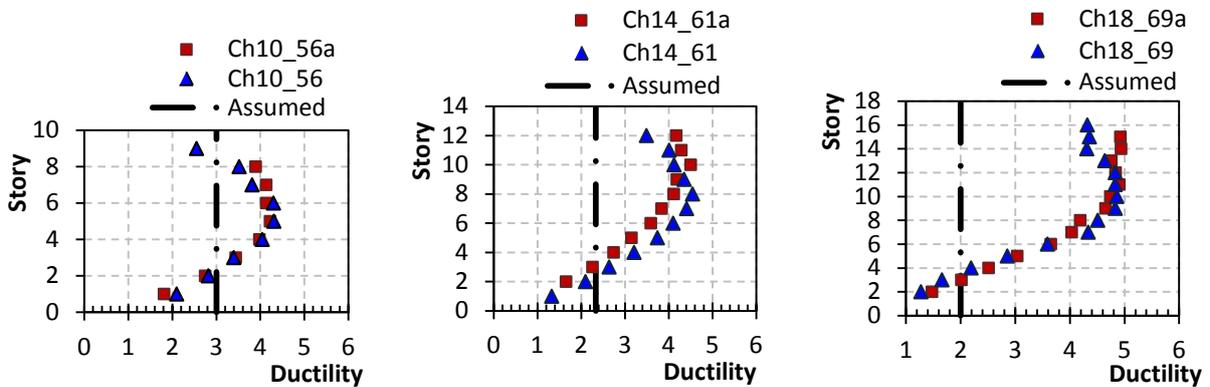


Figure 5.3. Developed ductility by the studied models

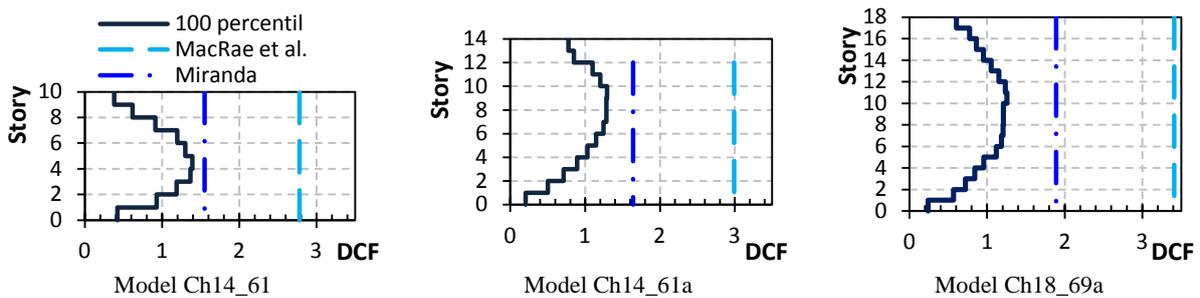


Figure 5.4. Drift Concentration Factor (DCF) in 10-, 14- and 18-story buildings

6. NONLINEAR DYNAMIC ANALYSES

Given that existing acceleration records corresponding to strong earthquakes are below the spectral accelerations considered for MFDC-04 design earthquake scenario, artificial records for a postulated $M_s=8.4$ subduction earthquake for a selected number of stations located in the studied zone (subzone IIIa) were used. These records were obtained by Godínez-Domínguez (2010). Elastic response spectra for the 10 artificial records that were finally selected for the detailed nonlinear dynamic analyses using OpenSees are compared with MFDC-04 design spectrum (Figure 6.1). As can be noticed, synthetic records compare reasonably well with the seismic hazard established in the Mexican Code.

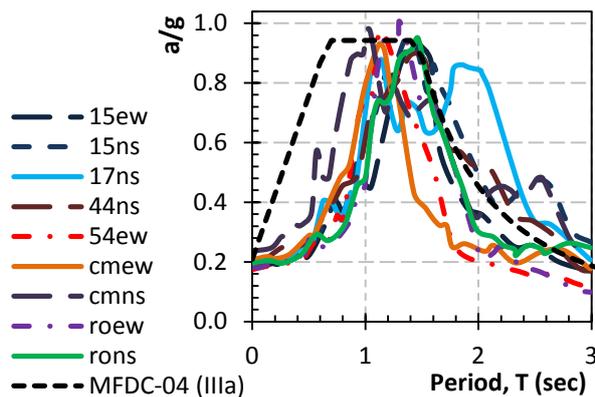


Figure 6.1. Response spectra for the artificial records and the elastic design spectrum

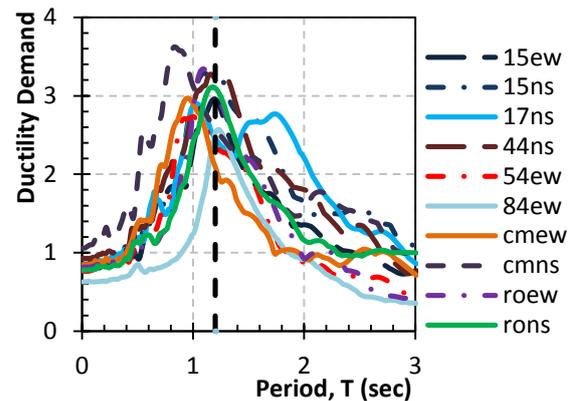


Figure 6.2. Ductility demand spectra for model Ch14_61a

Thus, once the lateral strength and the fundamental structural period for the analyzed buildings are determined, one can identify the ground motions records which will lead the subject structures to the highest ductility demands for a predetermined strength capacity (Tena-Colunga 2001). As an example of the computed ductility demand spectra, the one determined for the 14-stories building under the 10 selected artificial records is shown in Figure 6.2. Further information of records can be found in Tapia-Hernández and Tena-Colunga (2011a). In order to assess the global and local seismic behavior of the capacity-designed buildings, nonlinear dynamic analyses of 2D models accounting for the interaction among frames were performed. The story and global hysteresis curves (V/W_T vs Δ) and the normalized yielding mapping envelopes for all time-steps were processed from the dynamic analyses.

In Figure 6.3, accumulated (plastic) rotations in column and beam elements and accumulated shortening (inelastic buckling) or extensions (yielding) in brace elements are depicted for model *Ch14_61a*. Inelastic rotations and shortening/extensions are presented in a color scale normalized in relationship to the greatest demand along the record. From the yielding mappings, it was evident that bottom stories are associated to the highest demands. Despite the fact that upper stories show practically an elastic behavior, the building yielded reasonably along their height.

The first yielding in braces, beams or columns for each story was monitored along the dynamic analyses for the ten different records, in order to assess drifts at first yielding for each story. Then, peak interstory drifts at yielding were assessed for each model and for each considered acceleration record. Finally, an average interstory drift value was assessed from obtained results from all records. Despite that in some stories, the developed drifts at yielding are larger than the service drift limit equal to 0.4% (MFDC-04), the average value for the stories with inelastic response was 0.37% (0.4%). Therefore, for practical purposes, the code drift limit seems adequate.

As depicted in Figure 6.4, it was found that the design drift curve for each building under the design earthquake scenario (full triangles) envelops the average of peak dynamic drifts for the ten selected acceleration records (full rhomboids). Despite peak dynamic drifts at the lower stories are larger than drifts at the design stage, all drifts meet the code drift limit equal to 1.5%, as depicted in Figure 6.4.

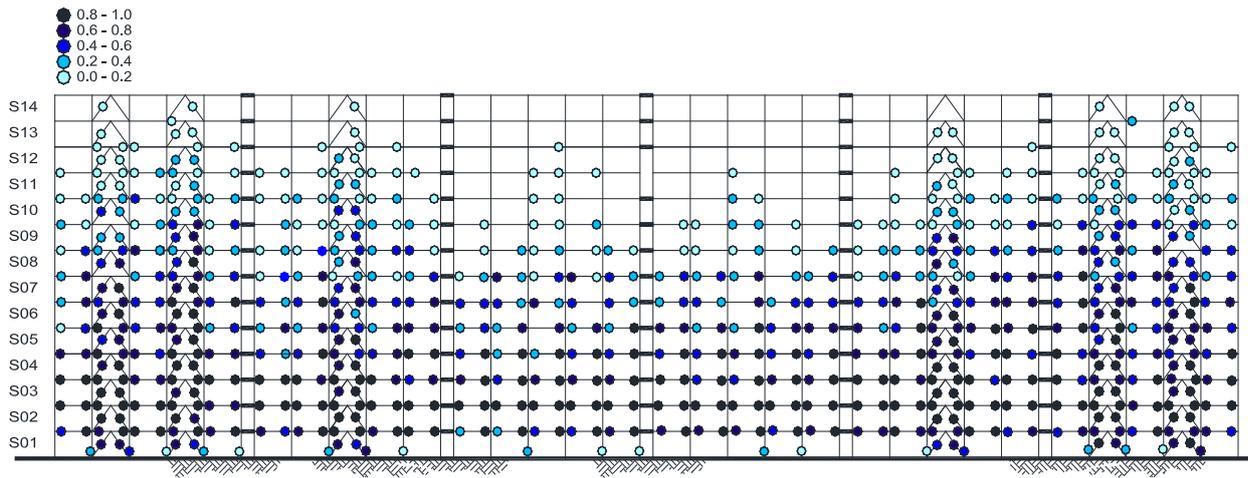


Figure 6.3. Accumulated hinge and extension/shortening in Ch14_61a along record 44ns

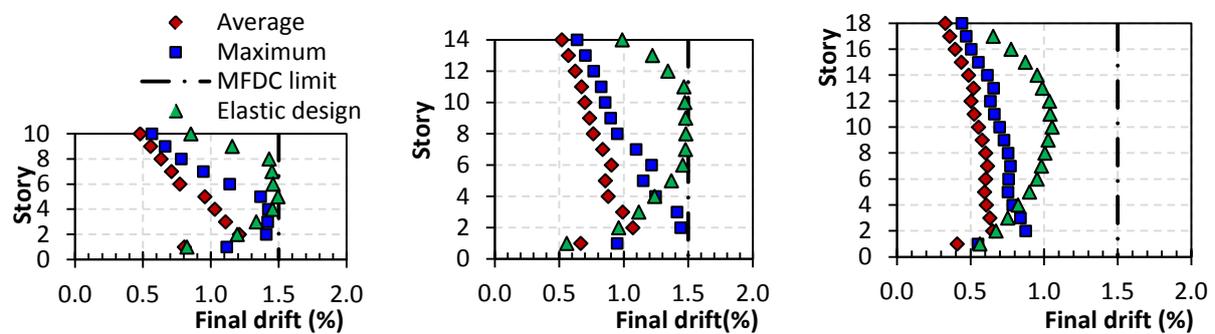


Figure 6.4. Peak dynamic drifts obtained by the studied models

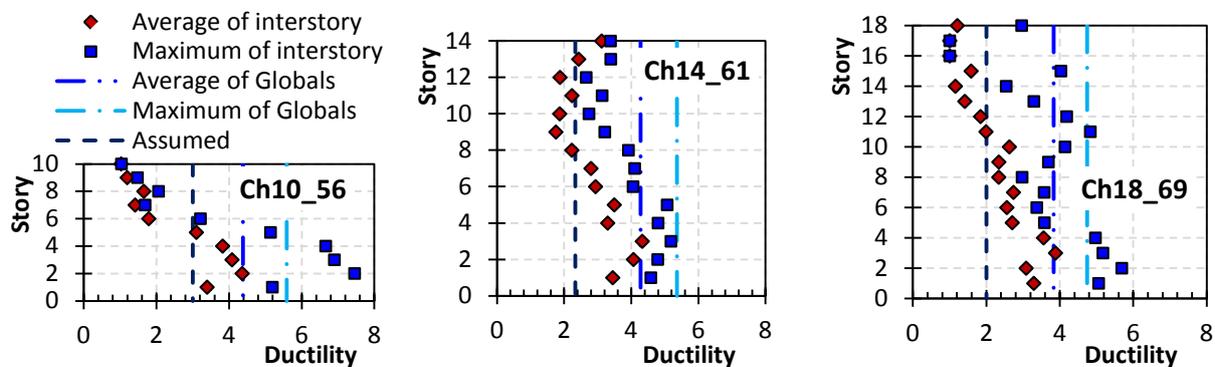


Figure 6.5. Peak ductility demands

Response envelopes for the peak ductility developed by the models are depicted in Figure 6.5 from two perspectives: the developed story ductility and the global ductility (considering the roof deformation and the building height). The global ductility assumed in the design using the proposed methodology is also included (Table 3.1). In these graphs, the decreasing tendency of the developed (demanded) ductility in relation to the building's height is noticed as in previous researches (Tapia-Hernández and Tena-Colunga 2008, 2011). Averages for the global ductility demanded to the models under study are reported in Table 6.1. According to these results, the proposed methodology predicts much better the ductility that the studied buildings can develop compared to current code's criterion.

Demanded overstrength was assessed as the peak base shear obtained in the nonlinear dynamic analyses divided by the base shear at the design stage, and it is useful to evaluate the proposed methodology. In Figure 6.6, the average of the demanded overstrength for the studied models under

the ten acceleration records, the peak value obtained, the overstrength factor proposed by current MFDC-04 and the overstrength factor obtained from the proposed methodology are compared.

Table 6.1. Comparison of the ductility

Model	Slenderness building ratio, H/B	Q (proposed methodology)	Q (current code criteria)	Average developed ductility, μ
Ch10_56a	1.00	3.00	3.00	2.859
Ch14_61a	1.40	2.33	3.00	2.698
Ch18_69a	1.80	2.00	3.00	2.238

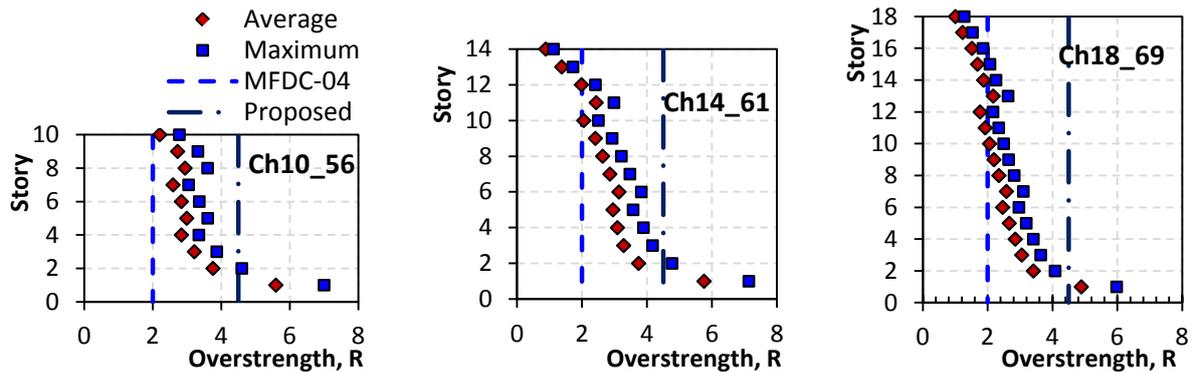


Figure 6.6. Peak overstrength demands

In an important amount of stories, the demanded (developed) overstrength is greater than the current code criteria $R= 2$. Only in some stories this overstrength is greater than the overstrength factor used for design, $R= 4.5$ (Table 3.1). These overstrength magnitudes are in agreement with the results of this and other studies (Tapia-Hernández 2011, Marino and Nakashima 2006) and some other international codes (European, USA and Japan) which propose that overstrength factors shall be at least 3 (EC8-2005, ASCE 7-05, Tada *et al* 2003). As can be noticed, results are close to the factor proposed in this research (Eq. 1.3), which ranges from $R=4.5$ to $R= 5.5$.

7. CONCLUDING REMARKS AND RECOMENDATIONS

In this paper, a study devoted to assess the effectiveness of the improved general design methodology of Mexico's Federal District Code (MFDC-04) using the design parameters proposed in previous studies (Tapia-Hernández and Tena-Colunga 2008, 2011) was presented. The behavior of investigated frames was evaluated performing pushover analyses and nonlinear time-history analyses under ten selected artificial ground motions. Realistic values for the steel's yielding stress considering 289 coupon test certificates of A-36 and A-572 Gr. 50 steel were used in the nonlinear analyses. Obtained results from nonlinear analyses lead to the following observations:

- Models designed with the proposed methodology developed a reasonable close strong column – weak beam – weaker brace mechanism. A reasonable distribution of the inelastic response along the building's height associated to null or reduced damage in columns was noticed.
- The model's tendency of developing a weak-story mechanism was evaluated considering the drift concentration factor using the results of pushover analyses. It was found that design procedure was effective in not driving potential weak stories, even though cross sections of columns, beams and braces were typified for a given number of stories, as normally happens in common practice.
- From the results of dynamic analyses, it was obtained that the average drift at yielding for all models is very close to the serviceability drift limit proposed by MFDC-04. Also, it was also observed that peak dynamic drifts satisfied the ultimate drift limit specified by MFDC-04.
- The drift curve at the design stage envelopes the average peak dynamic drifts computed for the

selected acceleration records in almost all the stories.

- According to the average peak ductility obtained in each analysis, the proposed methodology predicts much better the ductility that can be developed by the studied models than current criterion of MFDC-04.
- The average overstrength developed in each analysis are closer to the overstrength factor proposed, which varies from $R= 4.5$ to $R=5.5$.

According to these results, the proposed design methodology is successful in driving these structures to the desired collapse mechanism and allows designers to predict much better the ductility and overstrength that these systems can develop, thus, an improvement with respect to what it can be obtained following current recommendations of MFDC-04 and design modules of commercial software. However, further research is still needed to evaluate, for example: (a) different concentric bracing configurations and, (b) taller buildings where the slenderness ratio (height over plan dimension, H/B) may play a much more important role in limiting the ductility that this system may develop, as a consequence of an excessive axialization of the columns of the bottom stories.

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