BEHAVIOR OF MOMENT RESISTING REINFORCED CONCRETE CONCENTRIC BRACED FRAMES (RC-MRCBFs) IN SEISMIC ZONES

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ABSTRACT

This paper presents the results of a study devoted to evaluate, using nonlinear analyses, the behavior of ductile moment-resisting reinforced concrete concentric braced frames structures (RC-MRCBFs) using steel bracing. RC-MRCBFs were designed for lakebed region of Mexico City. It is possible to conclude from the results obtained in this study the need to improve current guidelines in the Mexican building code in order to warrant a ductile behavior for this structural system and to achieve the expected collapse mechanism.

KEYWORDS: Reinforced concrete structures, concentric braced frames, dual systems, seismic design, capacity design

1. INTRODUCTION

Due to the good behavior observed in the retrofitted RC structures by the addition of steel bracing in Mexico City, it is attractive to consider using this system for original design as well. In fact, the seismic guidelines of Mexico's Federal District Code (MFDC-04) allow for more than 30 years the design of a moment-resisting reinforced concrete concentric braced frames structures (RC-MRCBFs), considering even the possibility of a ductile behavior. However, in the design practice, this system has almost never been used for new construction.

Currently, there are shortcomings in MFDC-04 and other international codes to design ductile RC-MRCBFs. Most building codes extrapolate recommendations developed for ductile moment-resisting frames to design ductile MRCBFs, which seems to be an incomplete strategy. The absence of enough specific guidelines in building codes to insure a ductile behavior of MRCBFs using conceptual capacity design principles is notorious. Therefore, the expected failure mechanism of strong column–weak beam–weaker brace is not warrant following general guidelines available in international building codes.

According to the guidelines of Mexico's Federal District Code (MFDC-04), moment-resisting reinforced concrete concentric braced frames structures (RC-MRCBFs) should be analyzed considering the shear contribution of two structural systems as shown in Figure 1: the RC frame and the steel bracing system. Moment frames at all the stories must resist, without the bracing system contribution, at least 50% of the seismic force.

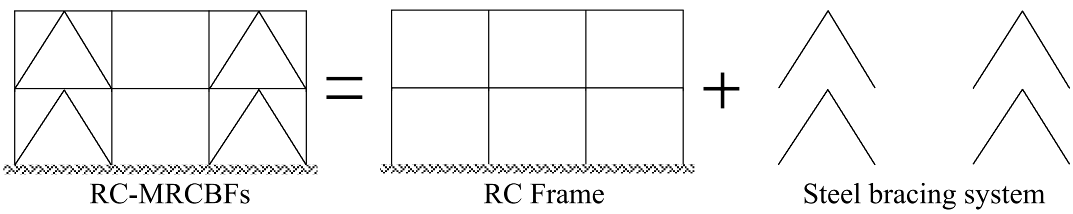


Figure 1. Assumption on how seismic shear forces must be resisted in RC-MRCBFs according with MFDC-04

2. SUBJECT BUILDINGS

Twenty-seven regular reinforced concrete moment-resisting concentric braced frames (RC-MRCBFs) using steel bracing were designed according to the seismic, concrete and steel guidelines of Mexico's Federal District Code (MFDC-04) for lake-bed region (zone III_b) and a seismic response modification factor Q=4, the maximum allowed for these structures. The corresponding elastic and inelastic design spectra are shown in Figure 2.

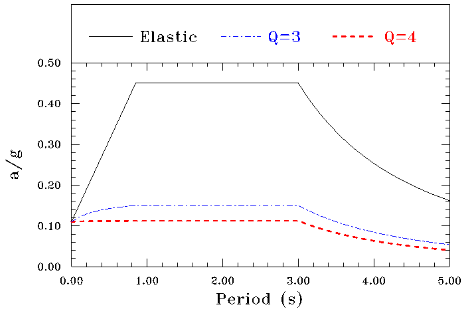
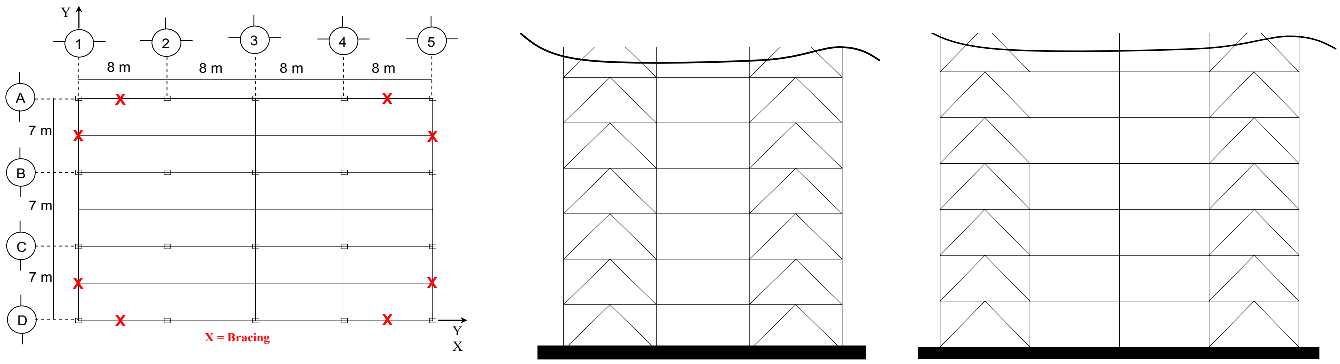


Figure 2. Design spectra for zone III_b according to the MFDC-04

Building models ranged from 4 to 16 stories, using two different bracing layout configurations (Fig. 3). The typical floor plan considered in the study is depicted in Figure 3. RC-MRCBFs were designed using different shear strength ratios between the bracing system and the moment frame system (Figure 1).



a) Floor plan for the building b) Frame layout in Y direction c) Frame layout in X direction

Figure 3. Floor plan of the building and elevation view of the studied frames

3. CASES OF STUDY

Three different cases were studied for each considered building height, based on the lateral shear strength percentage resisted by the columns of the RC-MRCBFs. The cases studied were:

(a) The percentage of the lateral shear strength provided for the steel bracing system is greater than the percentage provided for the columns of the RC moment-resisting frame. Near to twenty-five percent of the lateral shear strength is provided by the columns of the RC moment-resisting frame (condition no allowed in MFDC-04).

(b) Fifty percent of the lateral shear strength is provided by the columns of the RC moment-resisting frame (minimum shear strength percentage allowed in MDFC-04).

(c) Near to seventy-five percent of the lateral shear strength is provided for the columns of the RC moment-resisting frame (allowed condition in MDFC-04).

The considered strength balances just mentioned above were selected as they would allow a better understanding on the behavior of this structural system based on the shear strength distribution between the two different components of this dual system (Fig. 1) and the building height. Besides, it is important to review the minimum shear strength required for the columns of RC-MRCBFs, as the proposed value in MDFC-04 is based more in the experience and common sense of code committee members rather than in specific studies devoted to define an adequate target value.

4. DESIGN METHODOLOGY

Currently, there are shortcomings in most international codes to design RC-MRCBFs. Most RC building codes extrapolate recommendations developed for moment-resisting frames to design MRCBFs, which is an incomplete strategy. The absence of specific guidelines in building codes to warrant a ductile behavior in MRCBFs using conceptual capacity design principles is notorious. The expected failure mechanism of strong column–weak beam–weaker brace is not warrant following general guidelines available in many building codes. Therefore, a conceptual capacity design methodology has been explored in this research study for the design of MRCBFs. The methodology, which is described in detail elsewhere (Godínez-Dominguez and Tena-Colunga 2007) explicitly takes into account the sequence for designing resisting elements in order to warrant the expected collapse mechanism: (1) bracing elements, (2) beams, (3) columns, (4) connections between the frame and the bracing system and, (5) panel zone (joint area). The axial force transmitted from the bracing system to connections, columns, as well as to the beams subjected to such forces because of the bracing configuration is addressed in the design procedure, something that it is not currently addressed properly in RC building codes.

5. NONLINEAR STATIC ANALYSES

Nonlinear static analysis (pushover analyses) of representative perimeter frames models (Figure 3) for the designed 4 to 16 story buildings were carried out using Drain-2DX program (Prakash *et al.* 1992). P-Δ effects were considered in all analyses.

In order to identify the different models, a cryptogram was defined: *FNdpp*, where *F* indicates a chevron bracing frame, *N* indicates the number of stories of the frame, *d* indicates the analysis direction (x or y) according to the floor plan (Figure 3) and finally, *pp* indicates the shear force percentage provided by the RC columns.

The designed member sections for each RC-MRCBFs vary along the height of the frame, following a common design practice done by structural engineers in Mexico. Beams and columns change their cross section every four stories. Also, the box sections for the steel bracing change every three stories, in this case, the bracing system changes only its thickness remaining constant the width of the section.

Story and global lateral shear vs drift curves were obtained for all the described models. The curves obtained for the eight-story and sixteen-story models are shown in Figure 4 and Figure 5 respectively. These curves give insight on the medium-rise height building category that it is common in Mexico City. In these curves the lateral strength provided by the concrete columns and the steel braces elements is shown by separate, as well as the sum of the strength of these two components, which yields the total lateral strength for the RC-MRCBFs.

It is observed from Figures 4 and 5 that columns and braces behave differently as they enter into the inelastic range of response. After first yielding, steel braces keep on increasing their lateral strength (positive slope), whereas the columns, in the majority of the models, often decrease their lateral strength after yielding (negative slope). This effect can also be observed in Table 1, which the strength variation in braces and columns across the analysis are also reported. The mentioned phenomenon becomes less evident as the height of the models increase, as well as the lateral shear strength provided by the reinforced concrete columns of the frame increases. This phenomenon is observed both in the story curves and in the global response curves.

It can be observed from Figures 4 and 5 that for models where columns resist near 25% of the total seismic shear force (F8x25 and F16x25) that a reduced inelastic behavior is demanded in the upper stories, remaining elastic. On the other hand, for the models where columns resist near 75% of the total seismic shear force (F8x75 and F16x75), a better distribution of the inelastic behavior along the height is observed, which is desirable in order to obtain a more uniform distribution of the energy dissipation and, in fact, dissipate more energy.

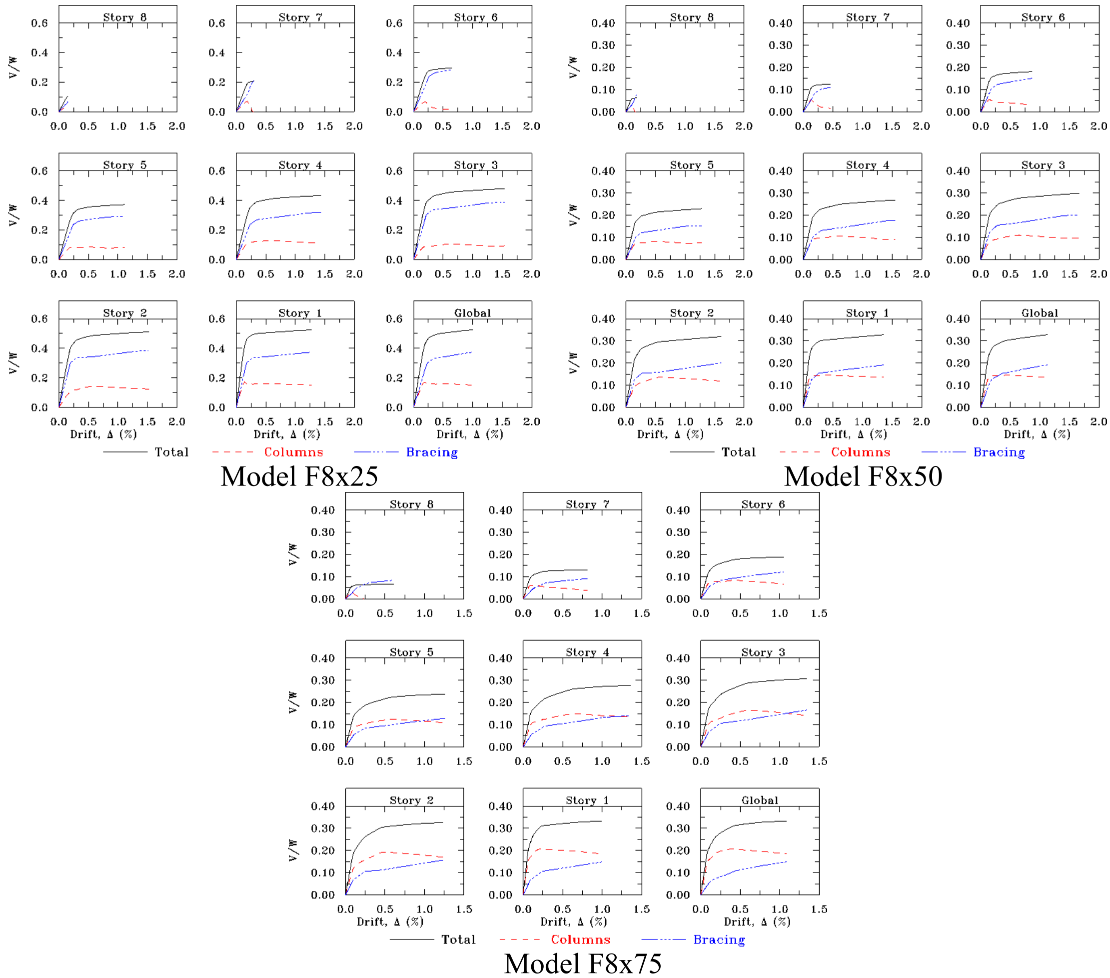


Figure 4. Story and global lateral shear-drift curves for the eight-story models

Table 1. Shear strength contribution for columns and braces to resist lateral seismic loads at different time-steps

	Elastic response		First yielding		First plastic hinge		Collapse mechanism	
Model	%Columns	%Braces	%Columns	%Braces	%Columns	%Braces	%Columns	%Braces
F4x25	34.13	65.87	31.71	68.29	33.32	66.68	32.26	67.74
F4x50	64.05	35.95	65.78	34.22	64.15	35.85	58.32	41.68
F4x75	79.19	20.81	80.43	19.57	75.42	24.58	64.12	35.88
F8x25	44.45	55.55	39.02	60.98	32.41	67.59	28.72	71.28
F8x50	59.84	40.16	55.34	44.66	47.98	52.02	41.64	58.36
F8x75	77.63	22.37	73.42	26.58	67.57	32.43	55.50	44.5
F12x25	44.66	55.34	40.85	59.15	27.62	72.38	23.54	76.46
F12x50	66.55	33.45	65.04	34.96	53.02	46.98	44.70	55.3
F12x75	71.98	28.02	69.28	30.72	63.19	36.81	45.63	54.37
F16x25	50.14	49.86	45.34	54.66	40.38	59.62	33.03	66.97
F16x50	63.94	36.06	63.47	36.53	56.76	43.24	44.29	55.71
F16x75	76.39	23.61	75.44	24.56	63.61	36.69	50.35	49.65

The largest story drifts observed in the lateral shear-drift curves correspond to intermediate levels, exactly at the stories where a change of section exist. As presented in following sections, the resisting elements at those levels experienced the highest plastic hinge rotations.

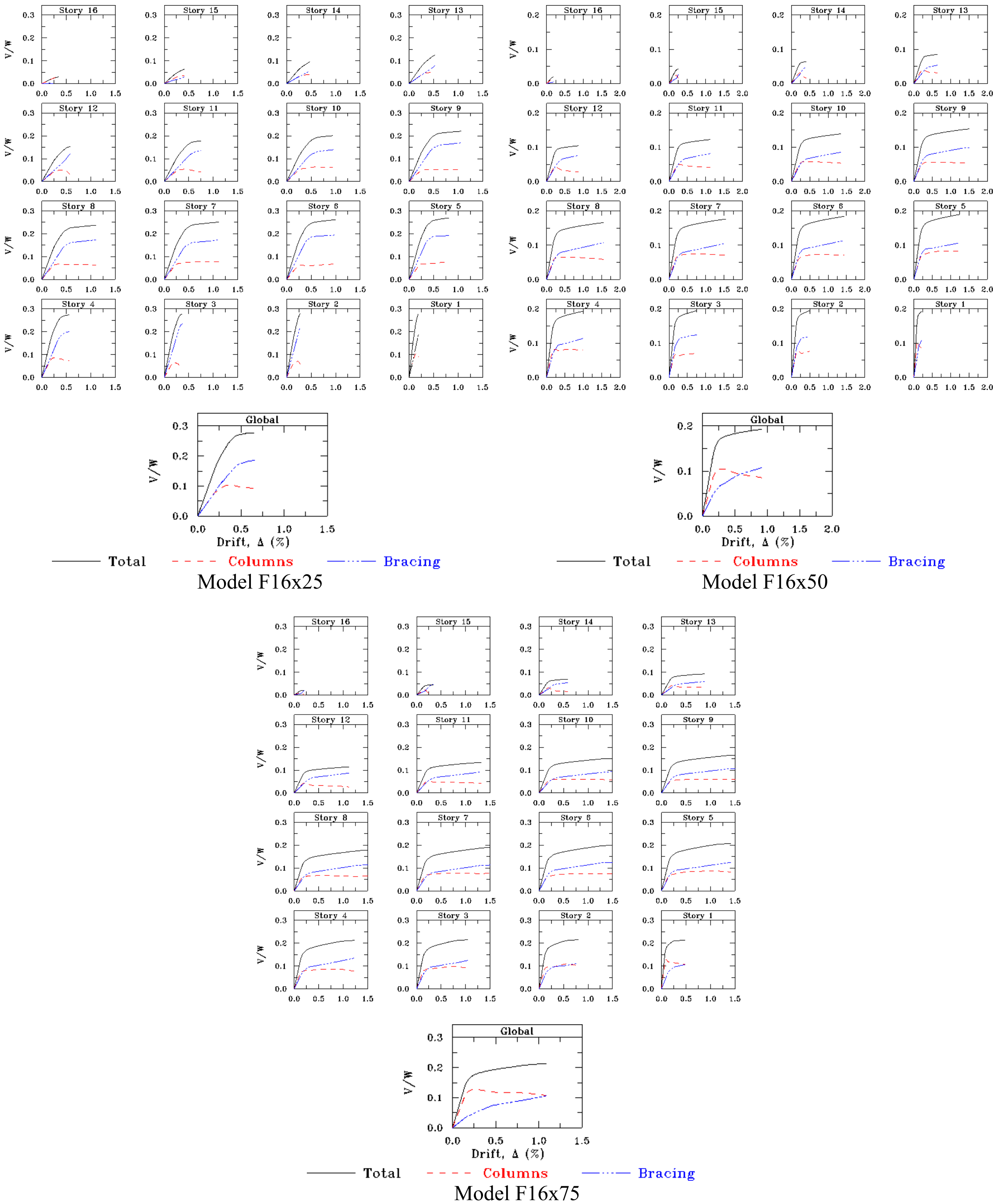


Figure 5. Story and global lateral shear-drift curves for the sixteen-story models

5.1. Yielding mapping

The final collapse mechanisms are presented for the models in the *x* direction only. The magnitude of inelastic deformations in beams and columns are shown by a color scale using circles, whereas the axial extension in braces (braces in tension at the left side of the braced bays) and the axial shortening in braces (braces in compression at the right side of the braced bays) are shown by a second color scale using oval marks.

Yielding mappings shown in Figures 6 to 8 correspond to the time-step where the collapse mechanism was formed. The maximum inelastic deformations were controlled taking into account the assessment of the theoretical plastic rotation capacities for beams and columns and axial extensions and buckling shortenings for the steel braces. For the braces, the magnitude of the buckling length (L_p), which defines the failure of the element, was computed according with the methodology proposed by Kemp (1996), which is based in a comprehensive compilation of experimental research.

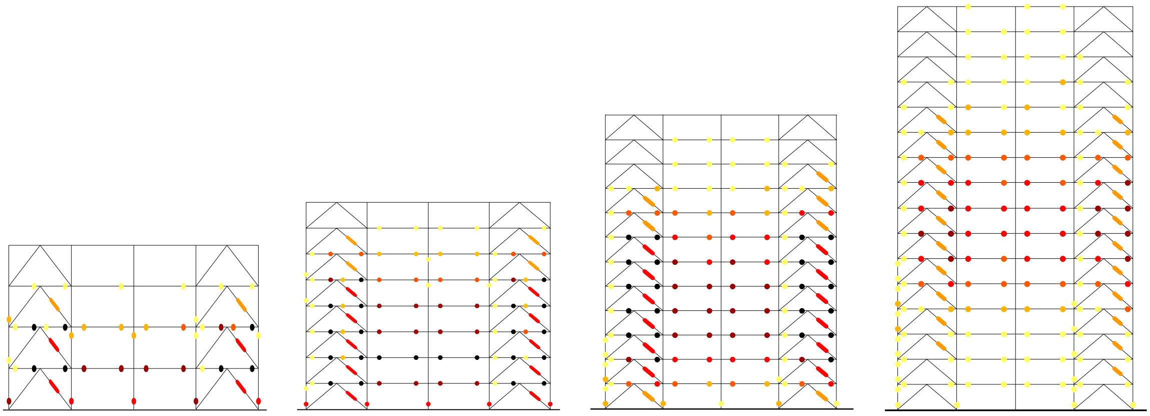
The yielding mapping for the final collapse mechanism for the four, eight, twelve and sixteen-story models where columns resist near 25% of the total seismic shear force are depicted in Figure 6; models where columns resist near 50% of the total seismic shear force are depicted in Figure 7; and finally, models where columns resist near 75% of the total seismic shear force are depicted in Figure 8.

The study of models where columns resist near of 25% of the total seismic shear force (not allowed in MFDC-04 for a ductile RC-MRCBFs, Figure 6) was done to explore smaller values to the minimum shear strength required in the columns of RC-MRCBFs in MDFC-04, as this recommendation is based more in experience and common sense than in specific studies focused to assess a minimum reasonable balance of the shear strength provided by each component of the RC-MRCBFs (Figure 1) that would lead to a ductile behavior. It was found that this strength balance lead to the use of stocky braces. As a consequence, the expected failure mechanism of strong column–weak beam–weaker brace is not warrant. In fact, the first plastic hinge rotation usually develops in a column. Also, due to the axial force transmitted by the braces to the columns, plastic hinge rotations can be formed at both columns ends in the same story (Figure. 6), which it is not desirable as it can lead to the formation of soft-story mechanisms.

For the models where columns resist near 50% (minimum shear strength contribution currently required for columns of ductile RC-MRCBFs by MFDC-04) and 75% of the total seismic shear force (Figures 7 and 8, respectively), the distribution and magnitudes of plastic hinge rotations are similar. The first plastic deformation always occurs in a brace element. Collapse mechanisms for low-rise and medium-rise models (four to twelve stories) adjust reasonably with the expected failure mechanism of strong column–weak beam–weaker brace. It is important to consider that plastic hinge rotations in columns at their base are due to the fixed-base modeling assumption. Nevertheless, as the models start to become taller (sixteen-story models, Figures. 7d and 8d), some incipient plastic rotations are formed at the column ends in the lower levels, which according with the direction of the applied loads, they are located in the tension side. These plastic rotations are developed because the magnitude of the axial force in the exterior columns of the 16-story models is higher than those developed in the other low-rise and medium-rise models.

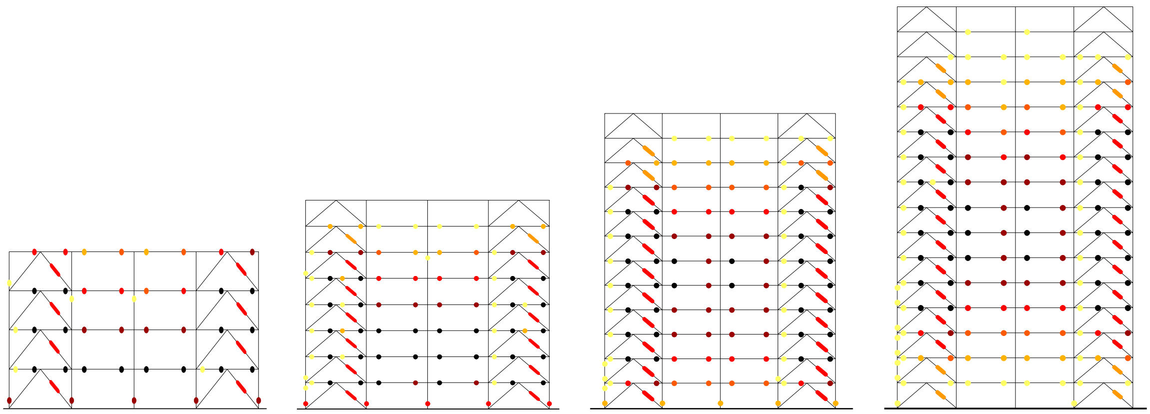
For building sixteen stories in height or taller, it is likely that the design methodology used must be adjusted to prevent the formation of plastic hinges at column ends at the lower stories, or to further limit the shear strength contribution for the columns to resist lateral seismic loads. The effect of the height in the structural behavior of MRCBFs has been explored. Most of the referred studies have focused in steel structures (i.e. Bruneau et al. 1998, Tapia-Hernández and Tena-Colunga 2008), and they have shown that rather different collapse mechanism could be developed for CBFs for medium-rise buildings.

Based upon the results discussed above, it is found that the guideline available in MFDC-04 that does not allow the design of RC-MRCBFs where columns resist less than 50% of the total seismic shear force seems reasonable enough.



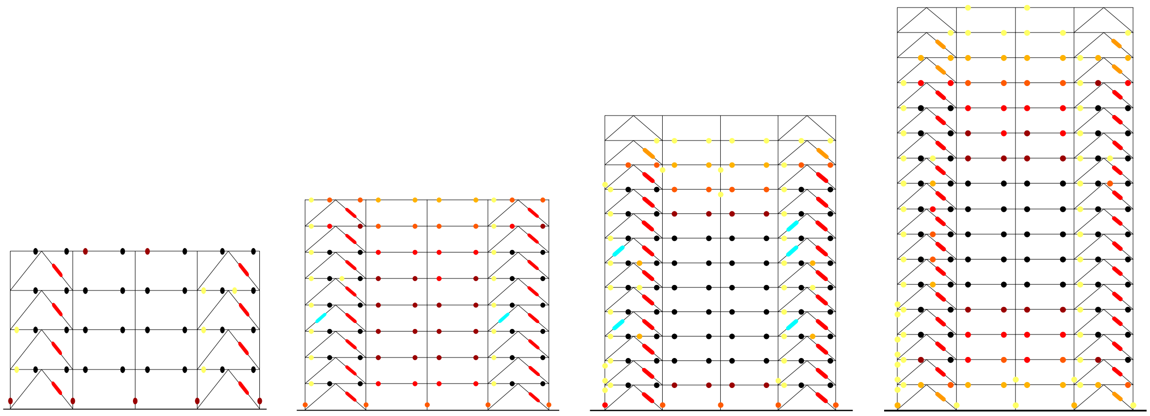
a) F4x25 b) F8x25 c) F12x25 d) F16x25

Figure 6. Collapse mechanisms for models where columns resist near 25% of the total seismic shear force



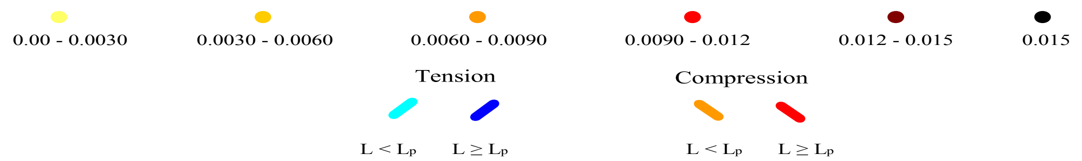
a) F4x50 b) F8x50 c) F12x50 d) F16x50

Figure 7. Collapse mechanisms for models where columns resist near 50% of the total seismic shear force



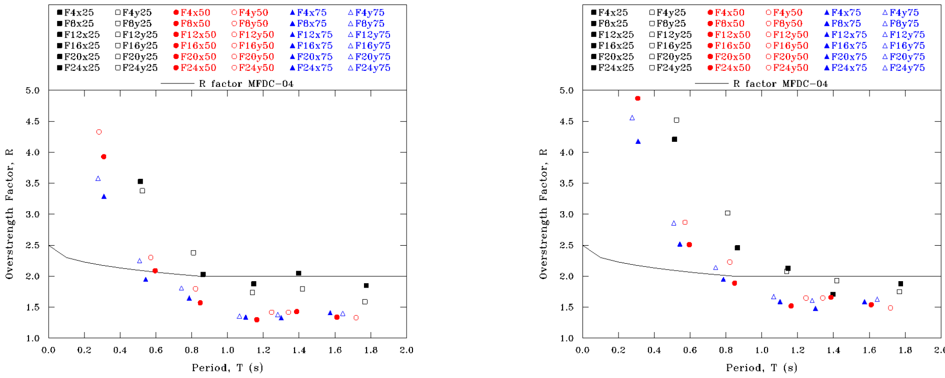
a) F4x75 b) F8x75 c) F12x75 d) F16x75

Figure 8. Collapse mechanisms for models where columns resist near 50% of the total seismic shear force



5.2. Overstrength factors (R)

Overstrength factors (R) are shown in Figure 9, which were assessed from the global pushover curves. Figure 9a and 9b shows the V_{nom}/V_{design} and V_{sr}/V_{design} relations, respectively. Also, the overstrength factor value proposed in the seismic provisions (MFDC-04) is depicted. It is clear that structures with low natural periods (four-story buildings) have greater overstrength levels than those proposed in MDFC-04. Generally, differences are observed between the assessed values and the proposed values. A relationship between the assessed R factors and the shear strength contribution for the columns to resist lateral seismic loads can be observed.



a) R (taking into account nominal strength) b) R (taking into account overstrength sources)

Figure 9. Assessed overstrength factors (R)

6. CONCLUDING REMARKS

Currently, there are shortcomings in many international codes to design ductile RC-MRCBFs. Many building codes extrapolate recommendations developed for RC moment-resisting frames to design MRCBFs, which is an incomplete strategy. From the results obtained in this study, it seems that the capacity design methodology used by the authors is successful to design ductile RC-MRCBFs when the columns of the moment frames resist at least 50% of the total seismic shear force, supporting with numerical evidence this proposed strength balance established in MFDC-04. However, the optimal strength balance between the RC frame and the steel bracing system seems to vary depending on the building height, so the authors are currently studying this possibility designing buildings with 20 and 24 stories. It was found a notorious difference between the assessed overstrength factors (R) and the proposed values in MFDC-04. A relationship seem to exist between the assessed R factors and the shear strength contribution for the columns to resist lateral loads, so a new expression could be developed to define the R factor for RC-MRCBFs in Mexican codes in order to improve their design.

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