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INFLUENCE OF CONCENTRIC X-BRACING OR CHEVRON INVERTED V-BRACING ON THE NONLINEAR BEHAVIOR OF DUCTILE MOMENT-RESISTING REINFORCED CONCRETE FRAMES

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ABSTRACT: In this paper the author summarize the results of a study devoted to evaluate, using nonlinear static analyses, the influence of two different bracing configurations on the behavior of low to medium rise ductile moment-resisting reinforced concrete concentric braced frames structures (RC-MRCBFs), as well as its impact in some specific design parameters. RC-MRCBFs height ranges from 4 to 20 stories, using both chevron and X-steel bracing. From the results obtained in this study it is possible to conclude that overstrength reductions factors (*R*) are dependent on the bracing configuration, therefore equations for the estimation of *R* factors for each studied bracing configuration are proposed. A story drift limit for service purposes, independent of bracing configuration, is proposed. It was also found that the peak story drift limit proposed in NTCS-04 is suitable for both bracing configurations. For the two bracing configurations, an equation to estimate the minimum shear strength provided by the columns of the RCMRCBFs as a function of the slenderness ratio of the building is proposed. Finally, it was observed that if proposed capacity design methodology is used, it is possible to design low and medium rise ductile RC-MRCBFs when the columns of the moment frames resist at least 50% of the total seismic shear force, obtaining a collapse mechanism that correlate reasonably well with the expected failure mechanism of strong column–weak beam–weaker brace.

1. Introduction

Steel bracing has been both studied and used as a retrofitting technique to limit earthquake damage in buildings, as well as stiffening and shear resisting system in the seismic design of new RC framed buildings. Some researchers have recently focused their attention on the study of seismic behavior, design parameters and guidelines for the design of new RC-MRCBFs using different steel bracing configurations (Maheri y Akbari 2003, Godínez-Domínguez and Tena-Colunga 2010, Godínez-Domínguez and Tena-Colunga 2012). In those studies the authors shows that the structural behavior of such systems is influenced by many factors. One of the most important factors on the seismic behavior of braced frame structures is precisely the bracing scheme used; presenting differences both locally and globally (Maheri and Akbari 2003). In a previous research (Godínez-Domínguez and Tena-Colunga 2010) the behavior of low to medium rise ductile moment-resisting reinforced concrete concentric braced frames structures (RC-MRCBFs) using chevron steel bracing only was studied. However, RC-MRCBFs using X-steel bracing have been widely used in Mexico (Del Valle *et al.* 1988) and are still used nationwide. Therefore, the study of low to medium rise ductile moment-resisting reinforced concrete concentric braced frames structures (RC-MRCBFs) using X-steel bracing allows to study the two more common bracing arrangements used in Mexico.

The main objective of this paper is to determine the influence of the bracing scheme used on the determination of key design parameters such as: a) overstrength reduction factors, b) story drift limits for the serviceability and collapse prevention limit states, and c) minimum shear strength provided by the columns of RC-MRCBFs in order to try to warrant a ductile behavior, and to achieve the expected collapse mechanism of strong column-weak beam-weaker brace.

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 both 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 (Fig. 1).

2. Subject buildings

Sixty regular reinforced concrete moment-resisting concentric braced frames (RC-MRCBFs) using both chevron and X-steel bracing were analyzed and designed using a proposed capacity design methodology adapted to the seismic, reinforced concrete and steel guidelines of current Mexico's Federal District Code (MFDC-04, Godínez-Domínguez and Tena-Colunga 2010 and 2012) for lake-bed region (zone IIIb) 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.

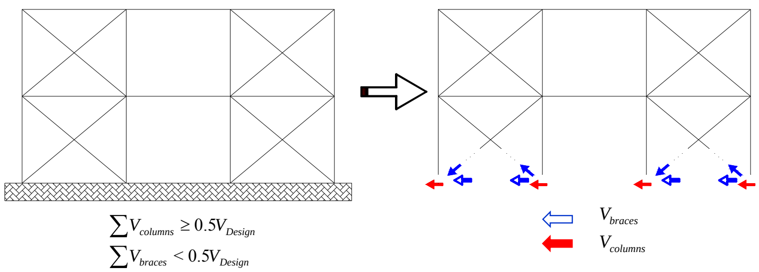


Fig. 1 – Shear resistance mechanisms in RC-MRCBFs according to MFDC-04

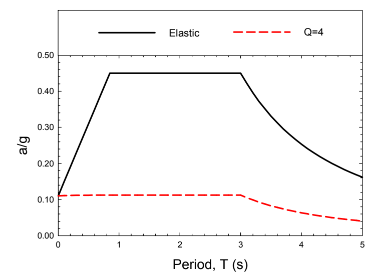
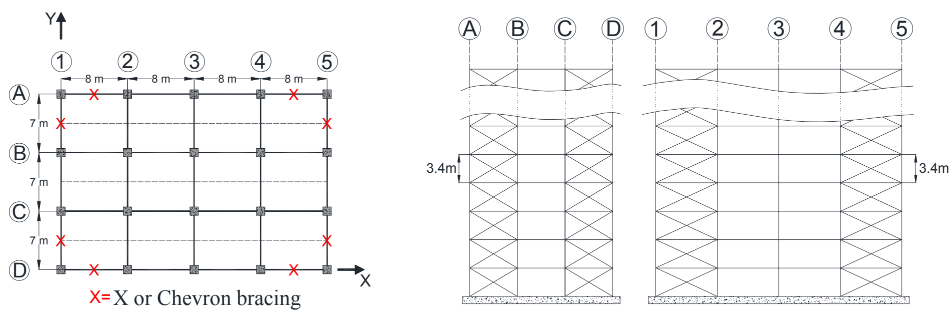


Fig. 2 – Design spectra for zone IIIb according to the MFDC-04

Building models ranged from four to 20 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). The configuration shown in Fig. 3 for the study of X- braced frames was previously employed for the study of chevron braced frames (Godínez-Domínguez and Tena-Colunga 2010).



a) Floor Plan b) Frame elevation in Y and X direction

Fig. 3 – Typical floor plan and frame elevation for the building models under study

3. Lateral strength balance

For the two bracing configuration selected (X and chevron) three different values for the lateral strength balance between the bracing system and the moment frame were studied for each building height, indicating the lateral shear strength percentage resisted by the columns of the RC-MRCBFs:

> ***Case I.*** The percentage of the lateral shear strength provided by the steel bracing system is greater than that provided for the columns of the RC moment-resisting frame. Up to 25% of the lateral shear strength is provided by the columns of the RC moment-resisting frame. This strength balance is not allowed in NTCS-04 for the ductile design of RC-MRCBFs.

Case II. Up to 50% of the lateral shear strength is provided by the columns of the RC moment-resisting frame. This is the minimum shear strength percentage that the columns of RC-MRCBFs must resist in order to do a ductile design according to NTCS-04.

Case III. Nearly 75% of the lateral shear strength is provided by the columns of the RC moment-resisting frame, an intermediate strength balance with respect to the limiting value recommended in NTCS-04 for the ductile design of RC-MRCBFs.

The values mentioned above were selected as they would allow a better understanding of the behavior of this dual structural system based on the shear strength distribution between their two different components (Fig. 1) and the building height. Besides, it is important to review the minimum shear strength balance 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 than in specific studies devoted to define an adequate limiting value. The corresponding cases of study, design parameters and dynamic characteristics of the investigated buildings are reported in detail elsewhere (Godínez-Domínguez and Tena-Colunga 2010, Godínez-Domínguez 2014).

4. Design methodology

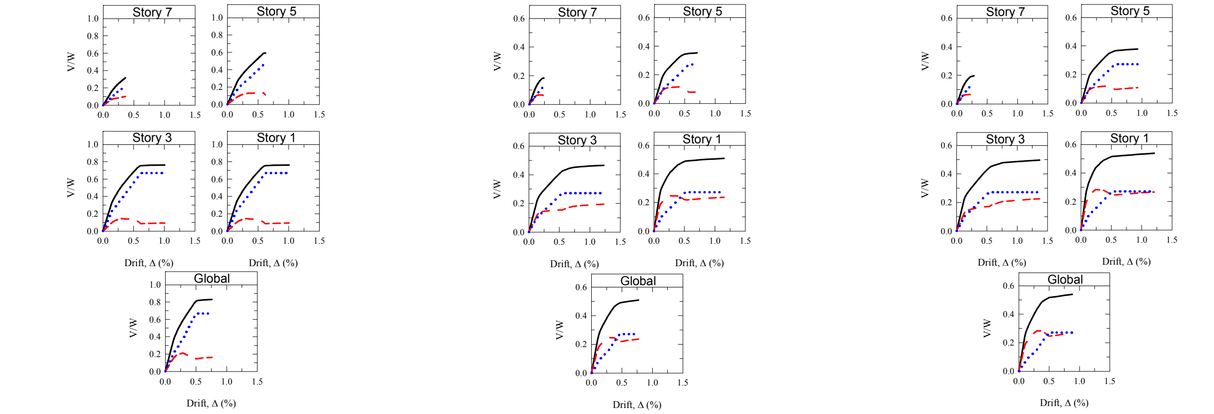
Currently, there are still some shortcomings in the guidelines of many international codes to design ductile RC-MRCBFs. The expected failure mechanism of strong column–weak beam–weaker brace is not necessarily warranted following general guidelines available in many building codes. Therefore, a conceptual 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 2010) 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 this design procedure, something that it is not currently addressed properly in RC building codes.

5. Nonlinear Static Analysis

In order to assess the global and local seismic behavior of the RC-MRCBF's, pushover analyses of representative perimeter frame models (Figure 3), for each considered bracing configuration, for the designed 4 to 20 story buildings were performed using Drain-2DX (Prakash *et al.* 1992). P-Δ effects were considered in all analyses. For simplicity, lateral load distributions selected to perform the pushover analyses were based upon the fundamental mode of vibration for all models. This was done to have a general framework of comparison, taken into account that: (a) building height ranges from 4 to 20 stories, (b) the modal mass associated to the fundamental mode is higher than 70% for most of the buildings with ductile behavior and, (c) RC-MRCBF's have a relatively large lateral stiffness and, therefore, higher mode effects have a reduced impact, as demonstrated previously (Godínez-Domínguez 2012) when comparing the results obtained with pushover analyses based upon the fundamental mode with those obtained with modal pushover analyses as presented in the literature (Chopra and Goel 2002, Goel and Chopra 2004).

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

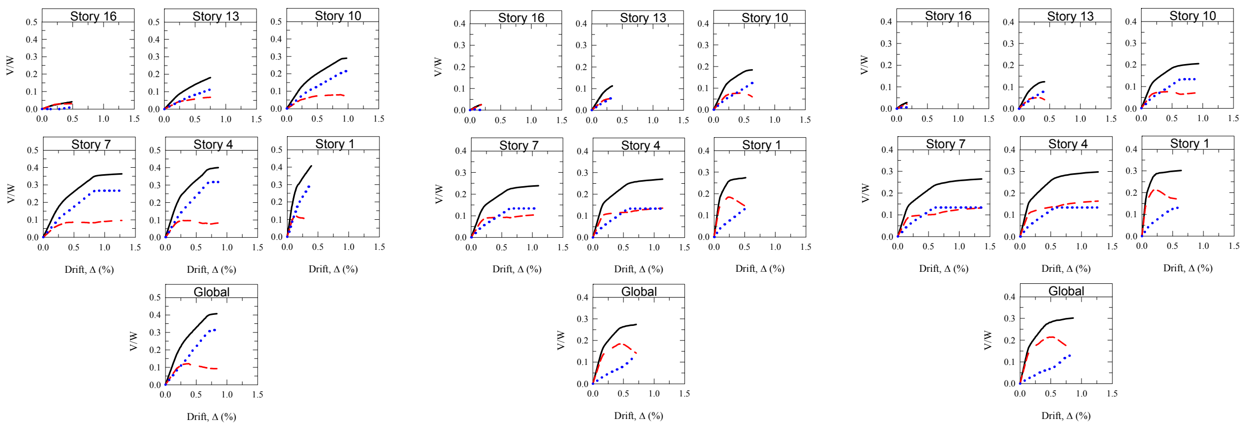
Story and global lateral shear *vs* drift curves were obtained for all the described models. The curves obtained for the X-braced frames of eight-story and sixteen-story models are shown in Figure 4 and Figure 5 respectively. These curves give insight on the medium-rise height building category that 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. The story and global lateral shear *vs* drift curves as well as all the results obtained from the nonlinear static and dynamic analyses for chevron braced frames are reported and discussed in detail elsewhere (Godínez-Domínguez and Tena-Colunga 2010 and Godínez Domínguez *et al.* 2012).



a) Model 8x25X b) Model 8x50X c) Model 8x75X

Total Columns Bracing

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



a) Model 16x25X b) Model 16x50X c) Model 16x75X

Total Columns Bracing

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

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). The same effect was observed in chevron braced frames, as commented in detail elsewhere (Godínez-Domínguez and Tena-Colunga 2011). For both bracing configurations, 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 (8x25X and 16x25X) that a reduced inelastic behavior is demanded in the upper stories, remaining basically elastic. On the other hand, for the models where columns resist near 75% of the total seismic shear force (8x75X and 16x75X), 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. The same effect was observed for chevron braced frames (Godínez 2010, Godínez-Domínguez and Tena-Colunga 2010).

6. Yielding mapping

In order to distinguish the principal elements responsible for the nonlinear response and to discern if the mapping is consistent with the expected failure mechanism of strong column-weak beam-weaker brace, yielding mappings were carried out at different load-steps. For illustrative purposes, yielding mappings for models where columns resist near 75% of the total seismic shear force are presented in this section for both chevron and X-steel braced frames.

From this section, in order to ease the cross results comparison and to evaluate the influence of the bracing configuration on the nonlinear behavior of ductile moment-resisting reinforced concrete frames, some results previously obtained for chevron braced frames are shown. As previously commented, the results of the nonlinear static and dynamic analyses for chevron braced frames are reported and commented in detail elsewhere (Godínez-Domínguez and Tena-Colunga 2010 and 2012).

The magnitude of inelastic deformations in beams and columns are shown by a color scale using full 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 full oval marks.

The yielding mapping for the final collapse mechanism for the four, eight, sixteen and twenty story models where columns resist nearly 75% of the total seismic shear force are depicted in Figure 6. The maximum inelastic deformations were controlled taking into account 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, which defines the failure of the element, was computed according with the methodology proposed by Kemp (1996), which it is based on a comprehensive compilation of experimental research.

The study of models where columns resist nearly 25% of the total seismic shear force (not shown), which are not allowed in MFDC-04 for ductile RC-MRCBFs, was made to explore the effects of smaller values of the minimum shear strength required in the columns of RC-MRCBFs in MDFC-04. This was done to assess the validity of this requirement as it is based more in experience and common sense than in specific studies focused to determine a minimum reasonable balance of the shear strength provided by each component of the RC-MRCBFs that would lead to a ductile behavior. It was found that this strength balance lead to the use of stocky braces. Consequently, the expected failure mechanism of strong column–weak beam–weaker brace for the two studied bracing configurations is not warranted. In fact, the first plastic hinge rotation usually develops in a column for the chevron braced frames, and in a beam for the X-braced frames. Also, due to the axial force transmitted by the braces to the columns, plastic hinges can be formed at both columns ends in the same story (Godínez-Domínguez and Tena-Colunga 2010, Godínez-Domínguez 2014). This is not desirable, since it can lead to the formation of soft-story mechanisms.

For models where columns resist nearly 50% and 75% of the total seismic shear force, for the two-studied bracing configurations, the distribution and magnitude of plastic hinge rotations are similar. It is worth nothing that for chevron frames, the first plastic deformation always occurs in a brace element; but for X-frames the first plastic deformation sometimes occurs in a brace or in a beam. Nevertheless, the collapse mechanisms for low-rise and medium-rise models (four to sixteen stories) correlate reasonably well with the expected failure mechanism of strong column–weak beam–weaker brace. It is worth noting that plastic hinge rotations in columns at their base are unavoidable, because of the fixed-base modeling assumption, but they are usually small. Nevertheless, as the structure becomes taller (twenty-story models) some incipient plastic rotations are formed at the column ends in the lower levels. These plastic rotations are developed because axial forces in the exterior columns of the twenty-story models are higher than those developed in the low-rise and medium-rise models. In this case, the magnitude of the plastic rotations are greater in X-frames than in chevron frames; this is mainly due to for X-frames the inelastic global displacements are greater than in chevron frames.

For the X-braced frames, and as previously reported for chevron braced frames (Godínez-Domínguez and Tena-Colunga 2010), it was found that for twenty story models or taller, it is likely that the design methodology commonly used must be adjusted to prevent the formation of plastic hinges at column ends at the lower stories, or to further increase the shear strength contribution for the columns to resist lateral seismic loads.

Based on the results discussed above, it is considered that the recommendation given in MFDC-04 that does not allow the design of ductile RC-MRCBFs where columns resist less than 50% of the total seismic shear force is adequate for RC framed buildings when X-bracing or chevron bracing is used.

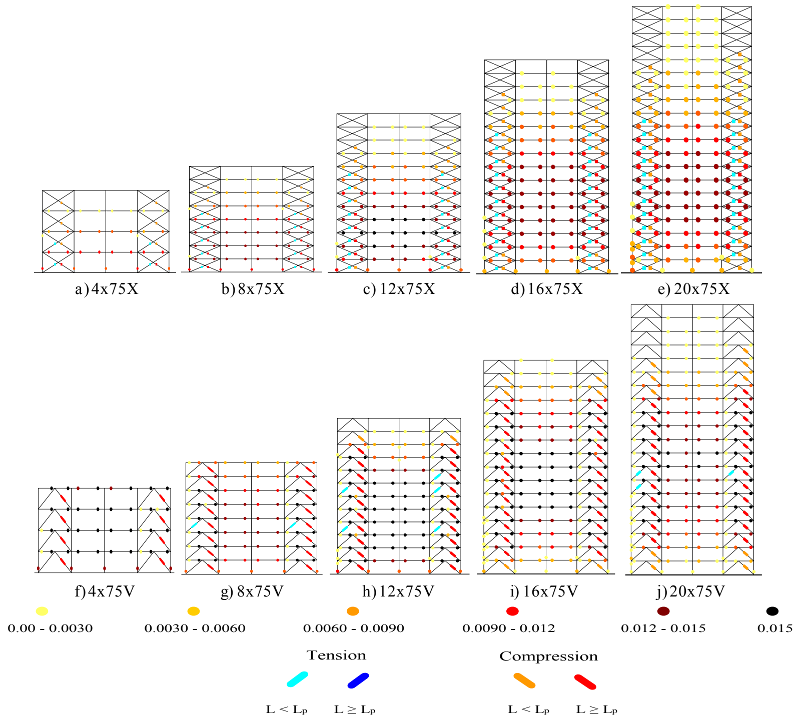


Fig. 6 - Collapse mechanisms for models where columns resist near 75% of the total seismic shear force (numbers under colored circles are rotations in radians)

7. Influence of the bracing configuration on some key design parameters

7.1. Overstrength factors (R or Ω_0)

Overstrength factors ($R=V_u/V_{des}$) for both chevron braced frames and X-braced frames are shown in Figure 7, which were assessed from the global pushover curves. Also, the overstrength factor value proposed in the seismic provisions (MFDC-04) is depicted. As can be seen from Figure 7, overstrength reductions factors (R) are dependent on the bracing configuration, as for the range of studied periods, the computed *R* factors for X-braced frames are always greater than those obtained for chevron braced frames. Therefore equations for the estimation of *R* factors for each studied bracing configuration are proposed (ec. 1 for chevron frames and ec. 2 for X-frames).

$$R = \begin{cases} 1.7 + 5.8\left(1 - \sqrt{T_e / T_a}\right) & si \;\; T_e \le T_a \\ 1.7; & si \;\; T_e > T_a \end{cases} \tag{1}$$

$$R = \begin{cases} 2.5 + 5.0\left(1 - \sqrt{T_e / T_a}\right) & si \;\; T_e \le T_a \\ 2.5; & si \;\; T_e > T_a \end{cases} \tag{2}$$

For both cheveron braced frames and X-braced frames it is clear that structures with low natural periods (four-story buildings) have greater overstrength levels than those proposed in MDFC-04; also, for X-braced frames with long natural periods, overstrength levels are greater than those proposed in MDFC-

04, whereas for chevron braced frames, overstrength levels are smaller than those proposed in MDFC-04.

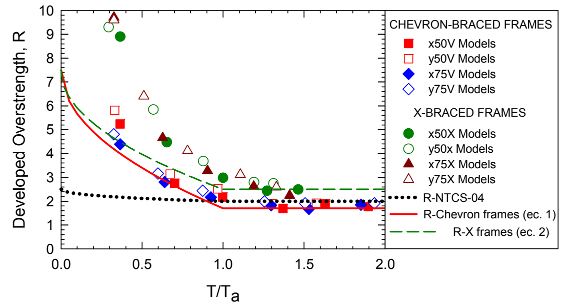
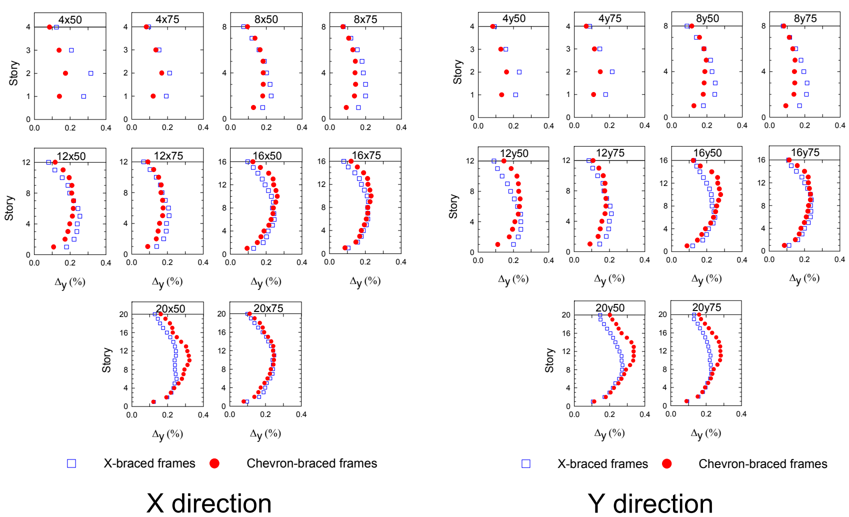


Fig. 7 – Influence of the bracing configuration on the overstrength reduction factors R

7.2. Equivalent story drift at yielding (Δ_y)

Envelopes for the equivalent story drift at yielding (Δ_y) computed from a bilinear idealized curve of the actual force-displacement response curve are shown in Figure 8 for the two bracing configurations. These curves are important as most seismic codes use this simple analogy to define global design parameters. Results for models where columns resist nearly 50% or 75% of the total seismic shear force are presented only. It is worth noting that some stories do not yield (μ=1). From the results shown in Figure 8, it can be observed that the equivalent peak story drift at yielding (Δ_y), usually occurs at the middle height of the building and it becomes larger as the height of the building increases. Similar results are obtained for both chevron and X-braced frames for models in the X and Y directions.

As observed previously for chevron braced frames, for all the studied X-braced frames, the computed story drifts at yielding are less than the corresponding story drift limit for service purposes (Δ_{ser}=0.004) specified in NTCS-04. This value was defined based on studies developed for moment-resisting frames, a fact that poses again the need to assess and define specific design parameters for RC-MRCBFs.



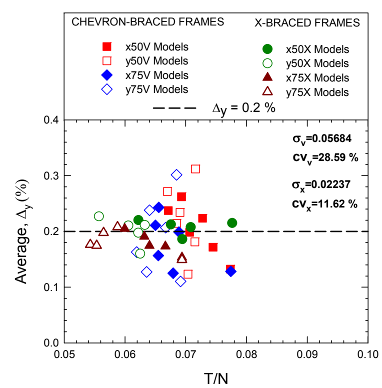


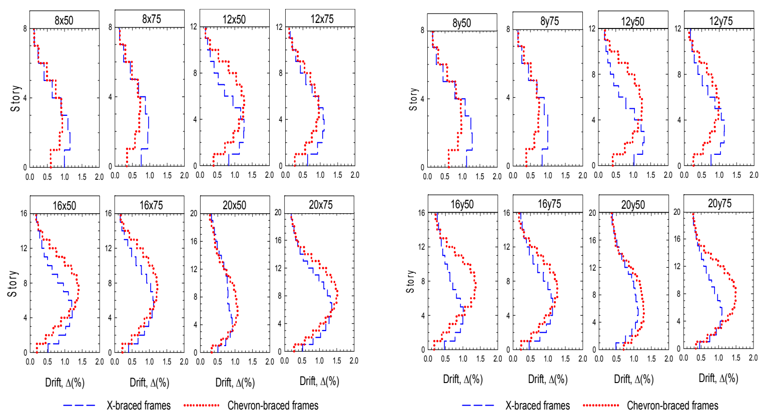
Fig. 8 – Equivalent yield drift envelopes for the RC-MRCBFs studied

Fig. 9 – Proposed Δ_y to assess the deformation capacity associated to the service limit state for the design of RC-MRCBFs

As previously done for chevron braced frames, in order to evaluate the influence of the bracing configuration on the Δy design parameter, average equivalent story drifts at yielding were computed for each X-braced model and then compared with the natural period (T) normalized with respect to the number of stories (N), a simple global stiffness parameter (Fig. 9). It is worth noting that for computing the average, values obtained at the first story were not included, because of the imposed fixed boundary condition. A proposed value of Δ_y=0.002 is also depicted in Figure 10 using a horizontal line, which represent the computed average of all the equivalent story drifts at yielding, considering all chevron-and X-braced models. From the above, it is seen that the proposed value of Δ_y=0.002 could be considered as independent of the bracing configuration, and could be employed for the design of RC frames using both chevron or X-steel bracing systems to assess the deformation capacity associated to the service limit state.

7.3. Peak story drifts (Δ_{max})

Envelopes for peak story drifts (Δ_{max}), for both chevron-braced frames and X-braced frames, are depicted in Figure 10 for models where columns resist nearly 50% and 75% of the total seismic shear force (ductile systems). As commented in a previous study (Godínez-Domínguez and Tena-Colunga 2010) these curves could be useful to define the maximum story drift for design purposes. In that study a peak drift limit (Δ_{max}=0.0013) for the design of RC-MRCBFs was proposed based on the results of nonlinear static analyses of chevron braced frames only. Later, using the results of nonlinear dynamic analyses of chevron braced frames structures (Godínez-Domínguez and Tena-Colunga 2012) it was concluded that the story drift limit for collapse prevention state Δ=0.015 currently proposed in NTCS-04 of MFDC-04, and other international building codes (i.e., ASCE-7), is a better option for reviewing the collapse prevention limit state than the one proposed previously.



a) X direction b) Y direction

Fig. 10 – Peak story drift envelopes for the studied RC-MRCBFs

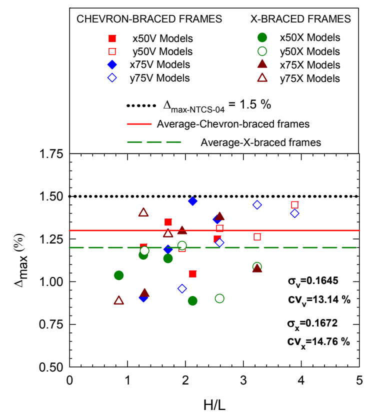


Fig. 11 – Story drift limit for collapse prevention state

In order to verify if the peak drift limit proposed for the design of RC-MRCBFs is suitable to be applied for the design of RC X-braced frames, the peak story drift for each model under consideration was computed, exactly as made for chevron frames. These peak story drifts are depicted in Figure 11 for both bracing configurations, where their potential relationship with the slenderness ratio is also evaluated. It can be observed from Figure 11 that peak story drifts increase as the slenderness ratio for the models increases. As observed from Figure 11, the story drift limit for collapse prevention state Δ=0.015 currently proposed in NTCS-04 of MFDC-04 (depicted in Figure 11 using a horizontal line), and other international building codes (i.e., ASCE-7), seems to be also adequate for reviewing the collapse prevention limit state for both RC chevron-braced frames and RC X-braced frames.

7.4. Minimum shear strength resisted by the columns of the RC-MRCBFs

Currently, 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. For ductile behavior, moment frames at all stories must resist, without the contribution of the bracing system, at least 50% of the seismic force. Based upon the results obtained, this recommendation seems reasonable enough for low-rise and medium-rise buildings designed following capacity design principles for the two bracing systems considered.

Nevertheless, for the taller models of each bracing configuration under study (sixteen stories or taller), it is likely that the design methodology used in this study must be adjusted to prevent the formation of plastic hinges at column ends at the lower stories, or to further increase the shear strength contribution for the columns to resist lateral seismic loads. In order to warrant a ductile behavior, and to achieve the expected collapse mechanism of strong column-weak beam-weaker brace, using the results shown before (and reported in greater detail in Godínez-Domínguez and Tena-Colunga 2010 and Godínez-Domínguez 2014), a modified minimum shear strength provided by the columns of the RC-MRCBFs is proposed as a function of the slenderness ratio of the building:

$$V_{RCol} \geq \begin{cases} 50 & si \quad \dfrac{H}{L} \leq 0.5 \\ 50 + 10\sqrt{\dfrac{H}{L}} & si \quad 0.5 < \dfrac{H}{L} \leq 4 \end{cases} \tag{3}$$

where:
V_{RCOL}= minimum shear strength provided by the columns in a specific story, in percentage.
H= height of the building.
L= base dimension in plan of the building in the direction of analysis.

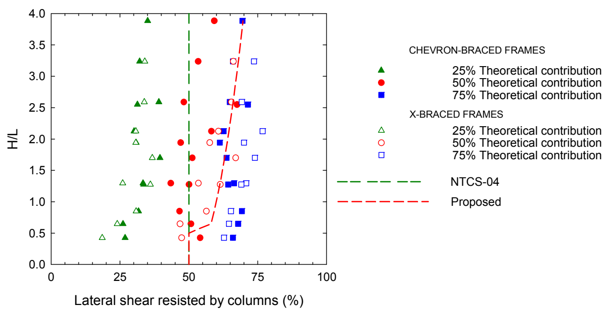


Fig. 12 – Proposed minimum shear strength percentage provided by the columns of the RC-MRCBFs in each story

The proposed expression (Eq. 3) offers a simple estimate of the minimum percentage of the seismic shear strength that columns of the RC-MRCBFs must provide (independently of the bracing configuration used), in order to avoid excessive inelastic behavior in columns at low and intermediate stories, and to concentrate the inelastic behavior on the bracing system and the beams along the height of the building. This insures the expected strong-column, weak-beam, weaker-bracing collapse mechanism.

The variation of the proposed minimum shear strength provided by the columns with the slenderness ratio of the building is shown in Figure 12, as well as the recommendation currently proposed in NTCS-04. Both chevron-braced frames and X-braced frames models studied, which were designed for different shear strength balances between the bracing system and the columns of the RC-MRCBFs, are depicted also. The models where columns resist nearly 25% of the total seismic shear force were not included in the least square approximation. The details of the criteria used to propose equation 3 are discussed elsewhere (Godínez-Domínguez 2014)

It is worth noting that the proposed equation might not be enough to insure consistent collapse mechanisms for ductile RC-MRCBFs, because their complex inelastic behavior is also influenced by other design parameters, like the deformation capacity provided by the beams, the slenderness ratio of the bracing system as well as the selected connection configuration of the panel zone (joint area). Nevertheless, it may be a good starting point to develop consistent collapse mechanisms for code-designed ductile RC-MRCBFs.

8. Concluding remarks

In this paper the author summarized the results of a study devoted to evaluate, using nonlinear static analyses, the influence of two different bracing configurations on the behavior of low to medium rise ductile moment-resisting reinforced concrete concentric braced frames structures (RC-MRCBFs), as well as its impact in some specific design parameters. RC-MRCBFs height ranges from 4 to 20 stories, using both chevron and X-steel bracing. From the results obtained in this study it is possible to conclude that overstrength reductions factors (*R*) are dependent on the bracing configuration, therefore equations for the estimation of *R* factors for each studied bracing configuration were proposed. A story drift limit for service purposes, independent of bracing configuration, was proposed. It was also found that the peak story drift limit proposed in NTCS-04 is suitable for both bracing configurations. For the two bracing configurations, an equation to estimate the minimum shear strength provided by the columns of the RCMRCBFs as a function of the slenderness ratio of the building was proposed. Finally, it was observed that if proposed capacity design methodology is used, it is possible to design low and medium rise ductile RC-MRCBFs when the columns of the moment frames resist at least 50% of the total seismic shear force, obtaining a collapse mechanism that correlate reasonably well with the expected failure mechanism of strong column–weak beam–weaker brace

9. Acknowledgements

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