Latin American Journal of Solids and Structures
www.lajss.org

Redundancy Factors for the Seismic Design of Ductile Reinforced Concrete Chevron Braced Frames

Abstract

In this paper the authors summarize the results of a study devoted to assess, using nonlinear static analyses, the impact of increasing the structural redundancy in ductile moment-resisting reinforced concrete concentric braced frames structures (RC-MRCBFs). Among the studied variables were the number of stories and the number of bays. Results obtained were compared with the currently proposed values in the Manual of Civil Structures (MOC-08), a model code of Mexico. The studied frames have 4, 8, 12 and 16-story with a story height h=3.5 m. and a fixed length L=12 m., where 1, 2, 3 or 4 bays have to be located. RC-MRCBFs were assumed to be located in soft soil conditions in Mexico City and were designed using a capacity design methodology adapted to general requirements of the seismic, reinforced concrete and steel guidelines of Mexican Codes. From the results obtained in this study it is possible to conclude that a different effect is observed in overstrength redundancy factors respect to ductility redundancy factors due to an increase of the bay number considered. Also, the structural redundancy factors obtained for this particular structural system varies respect to the currently proposed in MOC-08.

Keywords

Structural redundancy; Seismic design; RC braced frames; Steel braces; ductility; overstrength

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http://dx.doi.org/10.1590/1679-78252827

Received 29.01.2016
Accepted 30.05.2016
Available online 07.06.2016

1 INTRODUCTION

Although the importance and the positive effects of structural redundancy have been long recognized, structural redundancy became the focus of research in earthquake-resistant design in the mid 80s and particularly after the 1994 Northridge and 1995 Kobe earthquakes. Some researchers have investigated the benefit of redundancy for the structural system. However, the definition and interpretation of structural redundancy vary significantly and it remains a controversial subject (Liao and Wen 2004). Three different focuses have been mainly used for the study of redundancy (Tena-

Colunga and Cortés-Benítez 2015): a) probabilistic, b) a mixed-one, using both deterministic and probabilistic and, c) deterministic.

The experiences from past strong earthquakes (such as 1967 Caracas Earthquake, 1985 Michoacán Earthquake, 1994 Northridge Earthquake and 1995 Kobe Earthquake, etc.), indicated that the ductility and structural redundancy have proven to be the most effective means to provide safety against structural collapse and excessive damage, especially when earthquake demands are greater than those anticipated in the design stage. This happens because when a strong earthquake has to be resisted by a building, the overall response of the structure is no longer linear and it has entered in its inelastic range, which inevitably produces some damage in the structural members (to dissipate the earthquake input energy). The aforementioned it is considered as acceptable in the ultimate strength design philosophy, established in most modern international building codes, where it is taken into account that the structural members has the possibility to enter into its inelastic range of response. Obviously, this philosophy would lead buildings to experience damage during the occurrence of a strong earthquake, and that the earthquake-resistant structural system must be able to accommodate without experiencing collapse (collapse prevention limit state). As it is well known, redundancy contributes to an adequate structural performance, since as the number of resisting elements located in a given direction increases, better plastic stresses redistribution is allowed, contributing to develop a stable collapse mechanism (Tena-Colunga and Cortés-Benítez 2015).

As commented by Bertero and Bertero (1999) and Liao and Wen (2004), in structural engineering textbooks, redundancy is generally defined as the number of equations that are required for solution, in addition to the equilibrium equations. This definition may be inadequate in view of the complicated nonlinear structural behaviors under random earthquake excitations and the effects of uncertainty in demand and capacity. Therefore, according to Liao and Wen (2004), redundancy has been defined in different ways over the years. For example, Ang and Tang proposed in 1984 a definition of a non-redundant system when the failure probability of a component is equivalent to that of the entire system. Cornell in 1987 suggested a redundancy factor for the redundancy study of offshore structures be defined as the conditional probability of the system failure given the failure of any first member. Based on the study of parallel-member systems subject to random static loads, Hendawi and Frangopol proposed in 1994 a probabilistic redundancy factor defined as the ratio of the probability of any first-member yielding minus the probability of the collapse to the probability of collapse. In the "Blue Book", published by the Structural Engineers Association of California in 1999 (SEAOC), Recommended Lateral Force Requirements and Commentary, redundancy is defined as a characteristic of structures in which multiple paths of resistance to loads are provided (Liao and Wen 2004).

Bertero and Bertero (1999) indicated that the degree of redundancy for a structural system under lateral loads due to earthquake excitations could be described as a function of the number of critical regions (plastic hinges) of the structural system that must yield or fail when the structure collapses. In this case, the earthquake redundancy degree depends not only on the geometrical structural properties, reinforcement, and detailing, but also on the dynamic behavior of the structure and the earthquake ground motions time history. They concluded that a reduction factor due to redundancy cannot be established independently of the overstrength and ductility of the system. They also indicated that to take advantage of the redundancy based on its probabilistic effect, it is necessary: (1) to decrease the coefficient of variation of the demand relatively to the coefficient of variation of the supplied capacity; (2) to increase the overstrength; (3) to increase the plastic rotation capacity; and (4) to warrant a minimal rotation capacity in all members of the structural system, so that they can follow the displacement of the structure without failure and allow other elements to dissipate the earthquake input energy. However, it is important to note that they did not suggest a way to incorporate redundancy effect into procedures of structural design.

Whittaker *et al.* (1999) used a reliability index to investigate the redundancy of structures under earthquake excitations assumed to be deterministic. They recommended four lines of strength and deformation compatible vertical seismic framing in each principle direction of a building as the minimum for adequate redundancy. Therefore, it should be possible to penalize less redundant designs by requiring that higher design forces be used for such framing systems. Also, they proposed a draft redundancy factor, which varies as a function of the number of the vertical lines.

Wang and Wen (2000) studied a 3-D building model under seismic loadings. They proposed a uniform-risk redundancy factor to calculate the required design base-shear force for structures of different degrees of redundancy to satisfy a uniform reliability requirement in order to study the behavior of brittle connections of pre-Northridge steel buildings.

Song and Wen (2000) investigated the redundancy of special moment resisting frames (SMRF) in terms of the system reliability under SAC project ground motions. The variables that they considered included structural configuration (number of moment resisting frames), uncertainty in demand (in terms of column drift ratio) and uncertainty in material strength. The study included 3-D ductile SMRF of three and nine stories and equal floor area and strength, with a different numbers of bays and different beam and column sizes. Brittle and ductile connections were considered. They proposed a uniform-risk redundancy factor and compared with the redundancy factor (ϱ) proposed in UBC-97 and IBC-2000 codes. They found that the ϱ factor was inconsistent, as it overestimates the effect of system configuration and underestimates the effects of ductility capacity.

Liao and Wen (2004) developed a probabilistic study of redundancy of steel moment frame systems and developed a uniform-risk redundancy factor for assessment of redundancy. The inelastic behavior of connections (pre and post Northridge) was considered in structural response and redundancy evaluation. In order to account for the biaxial interaction of buildings with non-uniform mass distribution or with asymmetric plan configuration, they developed a 3-D finite element model based on ABAQUS in which the lateral resistance of the gravity frames was also included. They also include other important factors such as: uncertainties in material, uncertainties in connection capacity, P-Δ effects, panel zone effects, inelastic column behaviors and accidental torsion. The evaluation of structural redundancy against incipient collapse was carried out through a framework that considers: (1) the maximum column drift ratio (MCDR) and biaxial spectral acceleration (BSA) as a measure of both the demand and the capacity of a given building; (2) the demand and capacity using Incremental Dynamic Analysis (IDA) of a building which yields the probabilistic demand curve and the distribution of capacity; (3) both random and epistemic uncertainties in demand and capacity; (4) the uniform-risk redundancy factor, R_R. R_R factor was used in design to achieve a uniform reliability level for buildings of different redundancies as well as to evaluate the redundancy of a given structural system. They proposed regression models to provide a good estimate of l/R_R as function of number of moment frames and number of story and, according to the authors; it could be used as a guide in code provision. They proposed that at least three moment frames in each principal direction are needed to ensure a structure with adequate redundancy. Finally, they found that the ϱ factor proposed in NEHRP 97 generally overestimates the effects of floor area and the ϱ factor proposed in NEHRP 2003 fails to capture the variations and potential for serious damage for non-redundant and poorly designed structures, and also underestimates the seismic response of buildings with asymmetric plan configurations.

Husain and Tsopelas (2004) and Tsopelas and Husain (2004) presented a method based on pushover analysis (based on the assumption that a pushover analysis can be used to predict the dynamic behavior of structural frames) to quantify the deterministic and probabilistic effects of redundancy on the strength of structural systems. They introduced two indices to measure these effects, the redundancy strength index and the redundancy variation index. The redundancy indices were evaluated for plane reinforced concrete frames with different stories (3, 5, 7 and 9), a different number of vertical lines of resistance (1, 2, 4 and 6), and various beam ductility capacity ratios. They concluded that the redundancy modification factor, R_R, which accounts for the effects of plan and vertical irregularity on the strength of framing systems, depends on: a) the number of bays, b) bay widths, c) the number of stories, d) uniformly distributed gravitational beam loads and, e) beam ductility capacity ratios. They also concluded that for RC frames with a member ductility ratio of 10 or more, increasing member ductility does not add significantly to the frames redundancy. Nevertheless, increasing the member ductility capacity of ordinary RC frames (u_ϑ=1.5–3) significantly improves the frames redundancy. Finally, they showed that the redundancy of one, and two-bay special ductile frames improves significantly by adding extra bays. However, the effect is not as pronounced for frames with four bays or more.

Tena-Colunga and Cortés-Benítez (2015) developed a parametric study, using nonlinear static analyses, devoted to assess the impact of increasing the structural redundancy in ductile reinforced concrete moment framed buildings (RC-SMRFs). One of the main objectives of this research project was to evaluate the redundancy factors currently proposed in Mexican building codes (MOC-2008). Among the studied variables were the number of stories (4, 8, 12 and 16) and the number of bays (1, 2, 3 and 4). The authors showed that increasing the number of bays (higher redundancy) of RC-SMRFs starts to become more important for medium-rise frames (eight stories or above) than for low-rise frames (four stories). They showed that the proposed redundancy factors for overstrength and ductility decreases as the number of stories increases and that these factors tend to reach an upper limit as the number of stories increases. They also showed that for RC-SMRFs, the impact of redundancy is higher for their ductility capacity rather than for their strength capacity, which it is different to the currently proposed in MOC-2008, where the same impact for ductility and strength is considered in the redundancy factor. Regarding this point, they observed that, in general, in MOC-2008 the impact of redundancy in strength is overestimated and the impact of redundancy in ductility is underestimated. Finally, they concluded that, for the sake of transparency in the seismic design of RC-SMRFs and other structural systems, it is justified to account directly the structural redundancy in the design by using a redundancy factor, as currently proposed and done in some international building codes.

From the lessons learned of past earthquakes and from the cited analytical and experimental studies, it has been learned that redundancy is one of the most important characteristics in helping structures to avoid collapses during strong earthquakes, particularly when earthquake demands considerably surpass those assumed in their design. As shown by Tena-Colunga and Cortés-Benítez (2015), lack of redundancy could lead to a premature structural collapse due to fail of one or some specific elements. Despite of the poor seismic performance of weakly-redundant buildings observed during past earthquakes, this practice is still used in regions of Mexico with high seismic risk (Fig. 1). This solution for architectural needs is mainly related to land space constraints and the cost associated it. Therefore, it is clear the need to develop clear and easy to apply global code design parameters in the design process, in order to promote a good structural behavior when buildings are subjected to strong earthquakes and prevent the collapses observed in past earthquakes.

Figure 1: Nine-story building in construction process with only one-bay frame in the slender direction located in Tuxtla Gutiérrez, Chiapas, Mexico.

As commented by Tena-Colunga and Cortés-Benítez (2015), despite the acquired knowledge over years, in the last two decades the impact of redundancy has been oversight in building codes and in the seismic design of structures. As previously commented, there are just few research studies available where the impact of redundancy has been evaluated. Few international seismic building codes (or design guidelines) account redundancy for design directly, primarily in the United States (i.e., UBC-97 1997, ASCE-7 2010) and recently in Mexico (MOC-2008 2009). Therefore, there is a need to further evaluate the impact of redundancy in the seismic design and behavior of different structural systems, as well as recommendations currently available in some design guidelines and building codes.

The results of a study devoted to assess, using nonlinear static analyses, the impact of increasing the structural redundancy in ductile moment-resisting reinforced concrete concentric braced framed structures (RC-MRCBFs) is presented in following sections. Chevron steel bracing is considered. The studied variables are the number of stories and the number of bays. Results obtained were compared with the currently proposed values in the Manual of Civil Structures (MOC-08), a model code of Mexico. In this study, the adopted definition of redundancy is based on the proposal by Tena-Colunga and Cortés-Benítez (2015), in which redundancy refers to a characteristic of structures in which multiple continuous paths of resistance to loads exists, which causes a high degree of static indeterminacy, and has impact on both deformation capacities and strength.

2 CODE REQUIREMENTS

According to Liao and Wen (2004) and Tena-Colunga and Cortés-Benítez (2015), a reliability/redundancy factor for the seismic design of buildings, ϱ, was first introduced in NEHRP 97, UBC 1997, and IBC 2000. This factor was used as a multiplier of the lateral design earthquake load and took into account only the floor area and maximum element-story shear ratio in its first version. This ϱ factor lacked an adequate rationale and it could lead to poor structural designs, because it neglected important aspects such as (Liao and Wen 2004): a) it did not consider the differences between ductile and fragile connections, b) uncertainties in demands and capacities, c) irregularities in the structural configuration, d) bi-axial and torsional effects, e) relative stiffness and strength of vertical seismic framing.

Based on the results from analytical and experimental studies, the original proposal of UBC-97 has changed in the most recent recommendation of US Codes (ASCE-7 2010). According to ASCE-7 (2010), a redundancy factor, ϱ, shall be assigned to the seismic force-resisting system in each of two orthogonal directions for all structures. In this case, the redundancy factor is taken into account when the horizontal seismic load effect is determined and when the load combinations are computed for the structure analysis. A summary of the recommendations of US and Mexican building codes is available elsewhere (Tena-Colunga and Cortés-Benítez 2015). In the next section the recommendation of Mexican codes are summarized, as they are the subject of evaluation of this study.

2.1 MOC-2008

In MOC-2008 (MOC-2008 2009, Tena-Colunga *et al.* 2009) the redundancy factor (ϱ) is taken into consideration at the time of defining spectral design forces (Fig. 2). The purpose of this factor is to recognize directly that structural systems are able to develop more strength and increase their deformation capacity as they become more redundant. In MOC-2008, ϱ is a factor that basically corrects the previous assessment of the overstrength factor (R in Mexican codes) and the ductility factor (Q in Mexican codes), as most of the studies consulted in MOC-2008 to define the R values were done in 2-D models with different degrees of redundancy (MOC-2008 2009, Tena-Colunga *et al.* 2009). In addition, this factor takes into account unfavorable performances of weakly-redundant structures in strong earthquakes occurred worldwide in the last 40 years (Tena-Colunga and Cortés-Benítez 2015).

The proposed values for ϱ in MOC-2008 are illustrated in Fig. 3, and are the following (MOC-2008 2009, Tena-Colunga *et al.* 2009):

a) $\varrho = 0.8$ for structures with at least two earthquake-resistant parallel frames or lines of defense in the direction of analysis, if such frames are one-bay frames (or equivalent structural systems).

b) $\varrho = 1$ for structures with at least two earthquake-resistant parallel frames or lines of defense in the direction of analysis, if such frames have at least two bays (or equivalent structural systems).

c) $\varrho = 1.25$ for structures with at least three earthquake-resistant parallel frames or lines of defense in the direction of analysis, if such frames have at least three bays (or equivalent structural systems).

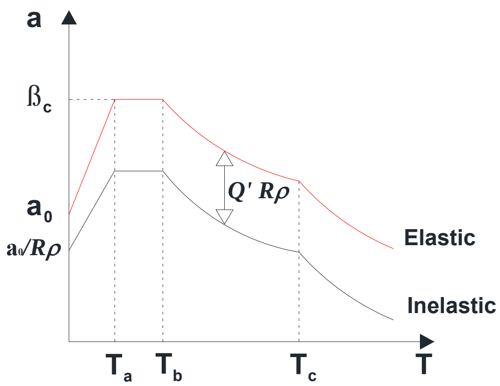


Figure 2: Schematic representation of inelastic acceleration design spectrum for MOC-2008.

As one can observe from Fig. 3, in MOC-2008, one-bay framed buildings are penalized in the design because they are weakly redundant, and their observed performances during strong earthquakes have been poor. Some collapses or partial collapses have been documented in reconnaissance reports (Tena-Colunga and Cortés-Benítez 2015). In addition, numerical collapses of such structures designed according to modern building codes have also been reported (i.e., Tena-Colunga 2004). Finally, smaller R factors have been reported in the literature for such frames (R=1.5, Tena-Colunga *et al.* 2009). The structural systems where $\varrho = 1$ is proposed correspond to those considered in most of the consulted studies to define target values for the overstrength factor R. The proposal for $\varrho = 1.25$ is based in some recent studies where parallel frames of these characteristics have been studied and where higher R factors were obtained (i.e., Tena-Colunga *et al.* 2009). It is also worth noting that the value of ϱ may vary in each main orthogonal direction.

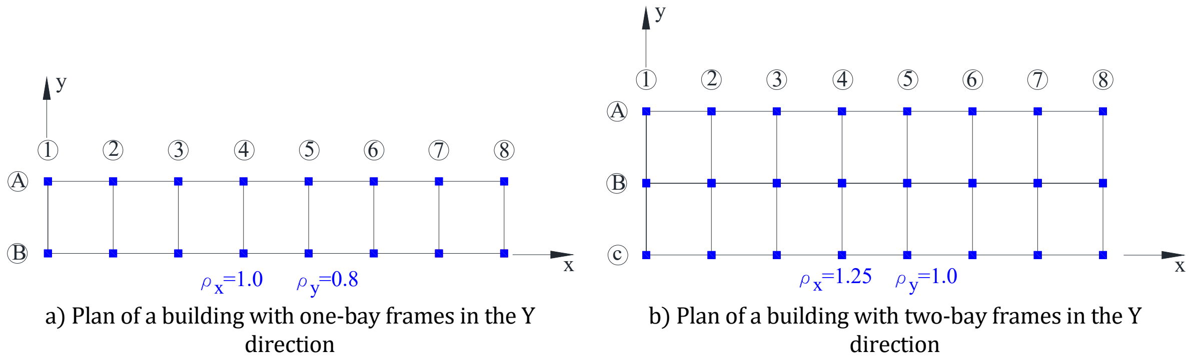


Figure 3: Sample buildings to illustrate the assessment of the ϱ factor of MOC-2008 (adapted from Tena-Colunga *et al.* 2009).

As commented by Tena-Colunga *et al.* (2009), the assessment of the ϱ factor for a given structure is straight-forward and it is illustrated with the buildings which plans are depicted in Figure 3. For the building plan depicted in Figure 3a, $\varrho = 0.8$ should be taken in the Y direction as it has eight parallel one-bay frames, whereas in the X direction, $\varrho = 1$ because it has two parallel seven-bay frames. In contrast, for the building plan depicted in Figure 3b, $\varrho = 1$ should be taken in the Y direction as it has eight parallel two-bay frames, whereas in the X direction, $\varrho = 1.25$ because it has three parallel seven-bay frames. This simple example illustrates the philosophy behind the ϱ factor. A-priori, most engineers would agree that the building plan depicted in Figure 3b is more redundant than the building plan depicted in Figure 3a. Former Mexican codes did not recognize directly this fact for their seismic design until MOC-2008 was released.

As commented in MOC-2008 (2009), although the values proposed for the ϱ factor are based on some studies, they are also based on past experiences and intuition. Therefore, there is room for improvement in assessing these values with specific-oriented research studies for future revision of this manual, which represents the main goal of this analytical research study.

3 SUBJECT FRAME MODELS

In order to assess the redundancy factor ϱ as proposed in the MOC-2008 code, regular RC-MRCBFs using chevron steel bracing were designed for a base shear V=0.10 W, where W is the total weight for the structure for seismic design. RC-MRCBFs buildings were designed for a specific shear strength ratio between the bracing system and the moment frame system depending on the slenderness ratio, as it is commented in following sections. A typical story height of 3.5 m (11.48 ft) and a fixed total width L=12 m (39.4 ft) were considered for all models. The studied frames have 4, 8, 12 and 16 stories, were 1, 2, 3 or 4 bays have to be located (Fig. 4). As commented previously by Tena-Colunga and Cortés-Benítez (2015), in order to assess the redundancy effect, the use of models with a fixed total width (as it is do it in this study) seems to be more adequate than those models with a variable floor plan.

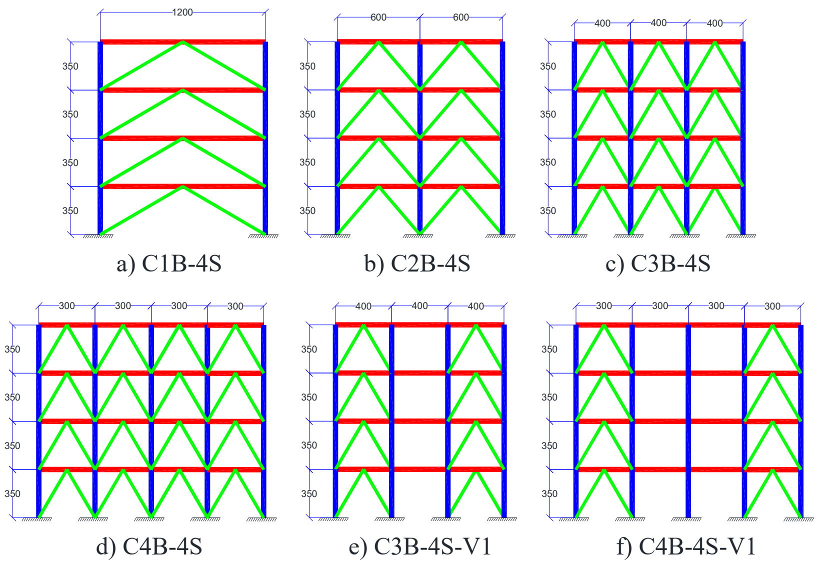


Figure 4: Frame layout and identification for the subject buildings of interest (dimensions in cm.).

Studied models were developed using ETABS software (ETABS 2013) and lateral load distributions were obtained using the static method of analysis, and were based upon the fundamental mode of vibration for all models due to the fundamental period T_e is smaller than T_b (Fig. 2). However, a correcting procedure for the lateral load distribution to account for higher mode effects is established for structures where the fundamental period T_e is greater than T_b (Fig. 2), as described elsewhere (MOC-2008 2009). An effective rigid-end zone of 50% was considered at beam-column joints. A fixed-base support condition was assumed. As a general strategy, all building were attempted to be designed as closely as possible to the limiting drift ratio Δ=0.015 (Δ =1.5%, Eq. 3) allowed by MOC-2008 for RC-MRCBFs (MOC-2008 2009, Tena-Colunga *et al.* 2009), in agreement with the results of previous studies (Godínez-Domínguez et al. 2012). This strategy was taken to crudely evaluate cases where MOC-2008 is less conservative and, therefore, in theory, buildings with such designs would be at higher risk of experiencing important inelastic deformations and damage during a severe earthquake. P-Δ effects were considered in all analyses. Soil-structure interaction was not included to avoid the introduction of other variables that may interfere in the interpretation of results.

For all building models, chevron bracing was designed using A-36 steel. Reinforced concrete beams and columns were designed assuming a compressive strength for the concrete=24.53 MPa (250 kgf/cm^2 or 3,550 psi). The elastic modulus for the concrete was estimated as E=$14000\sqrt{f_c'}$ (in kgf/cm^2) or E=$4400\sqrt{f_c'}$ (in MPa). A yielding stress f_y =412 MPa (4,200 kgf/cm^2 or 60 ksi) was considered for longitudinal and transverse steel reinforcement.

Following a common design practice by structural engineers in Mexico, the designed member sections for each RC-MRCBFs varied along the height of the frame. In order to prevent, as much as possible, important stiffness and strength irregularities along the height of the buildings, the cross sections of beams and columns change at stories different from those where the cross sections of the chevron braces change. Therefore, beams and columns change their cross section and/or steel reinforcement every four stories. The box cross section of the steel bracing typically changes every three stories, particularly in building models ranging from four to twelve stories, but for the sixteen-story building models, the cross section changes at different stories in order to achieve a design as "optimum" as possible. For all buildings, the bracing system changes only in thickness, remaining constant the width of the section.

Dynamic characteristics of the investigated buildings are summarized in Table 1. It is worth noting that the following notation is used to identify the models in Table 1: *CiB-jS*, where *C* indicates a chevron braced frame, i identify the number of bays and j the number of stories (Fig. 4).

In order to evaluate the effect of varying the number of braced bays over the redundancy factors, for models with three and four bays two different cases were analyzed: (a) models where all bays are braced and, (b) models where only the exterior bays are braced (identified using the suffix V1, Fig. 4). It is worth noting that some models considered in case (a) could tend to behave like a truss structure, while models considered in case (b) behave as dual systems.

4 DESIGN METHODOLOGY

As commented in previous studies (Godínez-Domínguez and Tena-Colunga 2010, Godínez-Domínguez *et al.* 2012) 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 guaranteed following general guidelines available in many building codes. Therefore, a conceptual capacity design methodology was used in this research for the design of RC-MRCBFs, which is described in detail elsewhere (Godínez-Domínguez and Tena-Colunga 2010, Godínez-Domínguez *et al.* 2012, Godínez-Domínguez 2014a), and explicitly takes into account the sequence for designing resisting elements in order to promote 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.

Model	T (s.)	Modal mass (%)	Model	T (s.)	Modal mass (%)
C1B-4S	0.286	81.07	C1B-12S	0.810	76.06
C2B-4S	0.302	83.57	C2B-12S	0.789	72.65
C3B-4S	0.274	84.77	C3B-12S	0.779	73.32
C4B-4S	0.295	85.29	C4B-12S	0.820	75.67
C3B-4S-V1	0.348	84.69	C3B-12S-V1	0.898	71.61
C4B-4N-V1	0.361	84.33	C4B-12S-V1	1.027	76.73
C1B-8S	0.561	79.23	C1B-16S	1.164	73.62
C2B-8S	0.572	78.18	C2B-16S	1.064	68.68
C3B-8S	0.518	77.46	C3B-16S	1.060	68.82
C4B-8S	0.542	79.73	C4B-16S	1.161	72.46
C3B-8S-V1	0.629	78.13	C3B-16S-V1	1.179	68.41
C4B-8S-V1	0.644	77.35	C4B-16S-V1	1.285	72.76

Table 1: Dynamic characteristics of the investigated buildings

4.1 Key Design Parameters Used for Design

As commented above, some design recommendations based on the results of nonlinear static and dynamic analyses of RC-MRCBFs ranging from 4 to 24 stories (Godínez-Domínguez and Tena-Colunga 2010, Godínez-Domínguez *et al.* 2012, Godínez-Domínguez 2014a) were used in this research. For example, overstrength reduction factors (R in Mexican codes or Ω_0 in USA codes), used for the design of all buildings, were obtained using equation 1 (which it is different to the currently proposed in MOC-2008). Similarly, the following design parameters were used in this study: (a) story drift limit for the serviceability limit state (Eq. 2), (b) story drift limit for the collapse prevention limit state (Eq. 3) and, (c) minimum required shear strength percentage provided by the columns of the RC-MRCBFs to resist earthquake loading (Eq. 4).

$$R = \Omega_0 = \begin{cases} 1.7 + 5.8\left(1 - \sqrt{T_e / T_a}\right) & if \quad T_e \le T_a \\ 1.7; & if \quad T_e > T_a \end{cases} \tag{1}$$

$$\Delta_y = 0.002 \tag{2}$$

$$\Delta_{\max} = 0.015 \tag{3}$$

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

In Equations 1 to 4, the following notation is used: T_e is the natural period for the building, T_a is the control period that defines the starting point of the plateau for the design spectrum, R is the overstrength factor (Ω_0 in USA codes), Δ_y is the story drift for serviceability limit state, Δ_{max} is the story drift for collapse prevention limit state, V_{RCol} is the minimum required shear strength percentage provided by the columns, H is the height of the building and, L is the dimension in plan for the subject building in the direction of analysis.

A simple estimate of the minimum percentage of the seismic shear strength that columns of the RC-MRCBFs must provide is assessed in Eq. 4, 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 is aimed to promote the expected strong-column, weak-beam, weaker-bracing collapse mechanism. The formulation of Eq. 4 is congruent to the specified in the seismic guidelines of Mexican codes (NTCS-04), where moment-resisting reinforced concrete concentric braced frames structures (RC-MRCBFs) should be analyzed considering the shear contribution of two structural systems: the RC frame and the steel bracing system. As commented in NTCS-04, for ductile behavior, the columns of the moment frames at all the stories must resist at ultimate, without the bracing system contribution, at least 50% of the seismic force (which is independent of the slenderness ratio of the studied building). For models where $H/L \leqslant 0.5$ (where gravity load combinations ruled the design of most elements) the criteria of NTCS-04 was adopted. It is worth noting that for buildings where H/L is higher than 0.5, a greater minimum shear strength percentage provided by the columns is required, as a function of the slenderness ratio, as proposed in Eq. 4.

It is worth noting that the proposed Eq. 4 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 might be a good starting point to develop consistent collapse mechanisms for code-designed ductile RC-MRCBFs.

5 NONLINEAR STATIC ANALYSES

In order to assess the redundancy factors, pushover analyses of all RC-MRCBF's models were performed using Drain-2DX (Prakash *et al.* 1992). For simplicity, lateral load distributions selected to perform the pushover analyses were based upon the fundamental mode of vibration for all models, which are congruent with those considered in design stage. This was also done to have a general framework of comparison, taken into account that: (a) building height ranges from 4 to 16 stories,

(b) the modal mass associated to the fundamental mode is higher than 70% for most of the buildings with ductile behavior (Table 1) and, (c) RC-MRCBF's have a relatively large lateral stiffness. For such conditions, higher mode effects have a reduced impact to assess global nonlinear demands using pushover analyses, as demonstrated previously (Godínez-Domínguez and Tena-Colunga 2014) 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).

5.1 Modeling Assumptions

In agreement to what it was considered in the design stage, P-Δ effects were included in all analyses but soil-structure interaction was not. Also, an effective rigid-end zone of 50% was considered at beam-column joints and a fixed-base support condition was assumed.

Structural properties for beams were obtained using moment-curvature relationships using Biax software (Wallace and Moehle 1989), and it correspond to sections located at the beam ends. The slab contribution to the structural properties of beams was considered explicitly (stiffness and strength). Regarding the deformation capacity of structural members of the models, the yield curvature corresponds to the first yield of the tension longitudinal steel, while their ultimate curvature was defined as the least of the curvatures corresponding to the fracture of the tension longitudinal steel and the crushing of the compression concrete block. Regarding columns, their structural properties were obtained using axial-flexure interaction diagrams using Biax software too.

The following assumptions were considered for assessing the member capacities: overstrength due to concrete confinement, using the modified Kent-Park model (Park and Priestley 1982), and the stress-strain curve for the steel reinforcement proposed for rebars produced in Mexico f_y =450 MPa (4,577 kgf/cm^2 or 65 ksi) was considered for RC beams and columns (Rodríguez and Botero 1995). The contribution of the slab reinforcement to the resisting bending moments of beams was also included in the assessment of overstrength capacities. The effective overhanging flange width was determined according to Mexican codes, in which it is established that it shall not exceed: a) one eighth of the clear span minus one-half of the web beam width, b) one-half the clear distance to the next web or, c) eight times the slab thickness. This is similar to what it is established in ACI 318 (ACI 318 2014). For the assessment of overstrength in the steel bracing, f_y =360 MPa (3,670 kgf/cm^2 or 52 ksi) was considered for A-36 steel for the determination of both tension and buckling loads, according to what it is available in the literature (Bruneau *et al.* 1998). These assumptions are consistent with the design procedure for each model and consider the overstrength that may develop if the required detailing by the reinforced concrete provisions (NTCC-04 2004) and the steel provisions (NTCM-04 2004) of Mexican codes are successfully implemented in the construction site.

The ultimate rotational capacity of beams and columns was estimated assuming a plastic hinge length equal to the half of the effective depth of the members. For 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.

For the modeling of the frames, beams and columns were modeled using an elastic-perfectly-plastic hysteretic model. For bracing elements, the considered hysteretic behavior is only capable of elastic buckling. However, this limitation does not impact significantly on the structural response, as has been demonstrated by comparing the results of nonlinear static analyses using DRAIN-2DX software regarding to those obtained using fiber models in OpenSees software, in which the user can reasonably model the out of plane buckling phenomenon (Tapia-Hernandez and Tena-Colunga 2014).

5.2 Yielding Mapping

As a first step yielding mappings were carried out at different load-steps prior to obtaining story and global lateral shear-drift curves, 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.

In this section, 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 diamond marks.

For models where only the exterior bays are braced (suffix V1), and a dual system behavior is promoted, the collapse mechanisms correlate reasonably well with the expected failure mechanism. Nevertheless, for models where all bays are braced, and as consequence it tends to behave like a truss structure, the collapse mechanism does not always correlate with the expected one, since for some of these models, a collapse mechanism of strong column-weak brace- weaker beam were observed. In order to illustrate the mentioned above, the sequence of the formation of the collapse mechanism for a model with these features is shown in Figure 5, where the deformed shape, the hinge plastic location and corresponding magnitude and, drift envelopes are plotted for different stages of behavior. The diagrams shown in Figure 5 correspond to: a) the load step before to the yielding of the first brace element (plastic behavior only in beams), b) the load step before the first column yields (plastic behavior in both beams and braces), c) a load step where plastic hinges at the base of the columns have been formed and, d) the load step where the collapse mechanism was formed. It is worth noting that in this case, the bracing elements start to develop plastic behavior when the beam rotation magnitudes remain still low (moderate nonlinear response), as can be seen from the color scale and magnitudes shown in Fig. 5, so that the collapse mechanism, although it is not entirely consistent with the expected one, it is stable.

For illustration purposes, yielding mappings obtained for the four-story and eight-story models, corresponding to the load-step where the collapse mechanism was formed, are shown 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. It is worth noting that hinge plastic rotation magnitudes shown in Fig. 6, as discussed below, correspond to global ductility levels close to that used as the design value ($Q = 4$).

It can be observed from Fig. 6 that, for most models, a good distribution of plastic hinges in height exist, avoiding plastic concentration demands in an specific story, effect particularly notorious for three-bay and four-bay frames.

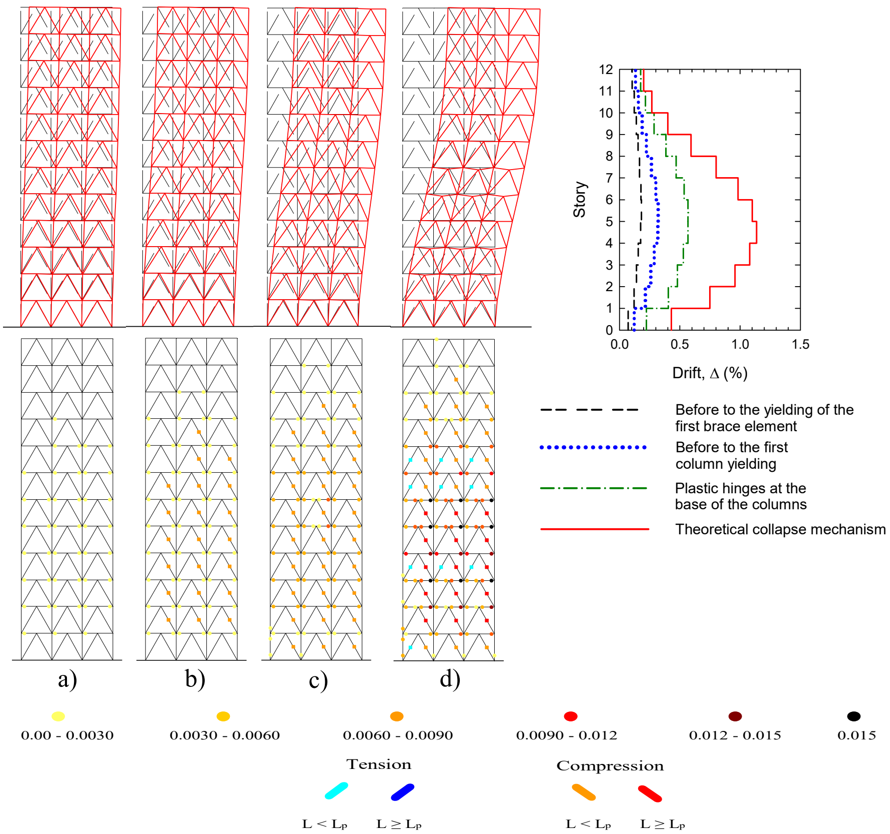


Figure 5: Sequences for the formation of the collapse mechanism for model C3B-12S.

5.3 Peak Story Drift Envelopes and Base Shear-Global Drift Curves

Peak story drift envelopes corresponding to the load-step where the collapse mechanism was formed are shown in Figure 7. It is observed that for all cases, the greater the number of bays, the greater the story inelastic deformation capacity. Also, in general, it can be observed that for models where only the exterior bays are braced and a dual system behavior is promoted, a greater inelastic deformation capacity is observed with respect to those models where all bays are braced (truss structure behavior).

As previously done by Tena-Colunga and Cortés-Benítez (2015), normalized base shear vs average drift curves computed from the roof displacement over the height of the structure (V/W vs Δ) were obtained as a first step to assess redundancy factors according to the proposal of MOC-2008. The results obtained for all models under study are shown in Figure 8. For space constraints, the results obtained for models where all bays are braced were not separated from those where only the exterior bays are braced (dual system models). However, the identification of curves that describes the behavior of dual system models is relatively simple to do, because in all cases, the lower values of the base shear (V/W) were obtained for the collapse state in those curves. It is observed from Fig. 8 that the elastic stiffness for the studied models do not vary significantly as the number of bays increases, except for dual systems where, in general, the stiffness is somewhat smaller than in the other models.

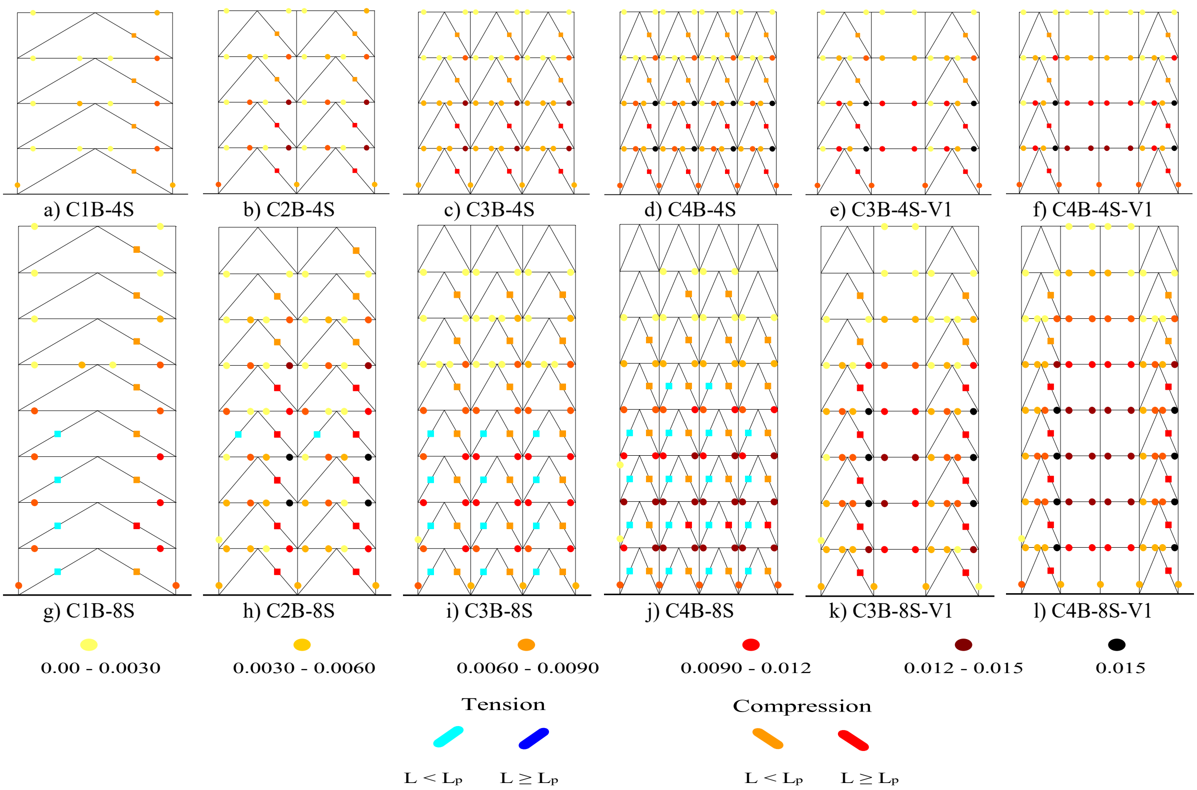


Figure 6: Yielding mappings for the four and eight story models.

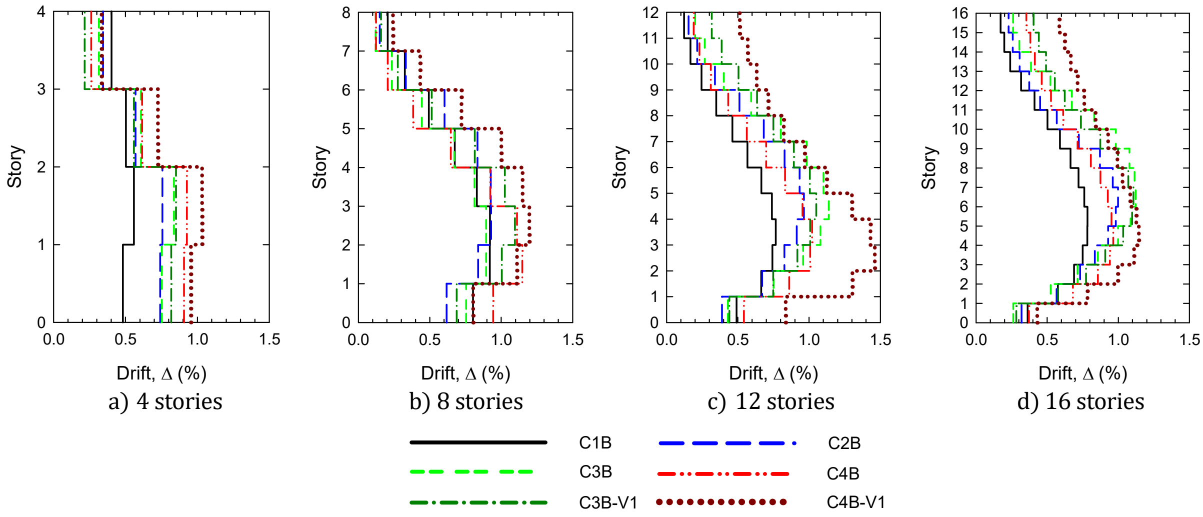


Figure 7: Peak story drift envelopes.

It is also observed from Fig. 8 that for the 4, 12 and 16 story models, the greater the number of bays, the greater the global inelastic deformation capacity. Nevertheless, unexpectedly, for the eight-story models, for frames with one-bay and two-bay, a greater global inelastic deformation capacity was obtained than for three-bay model.

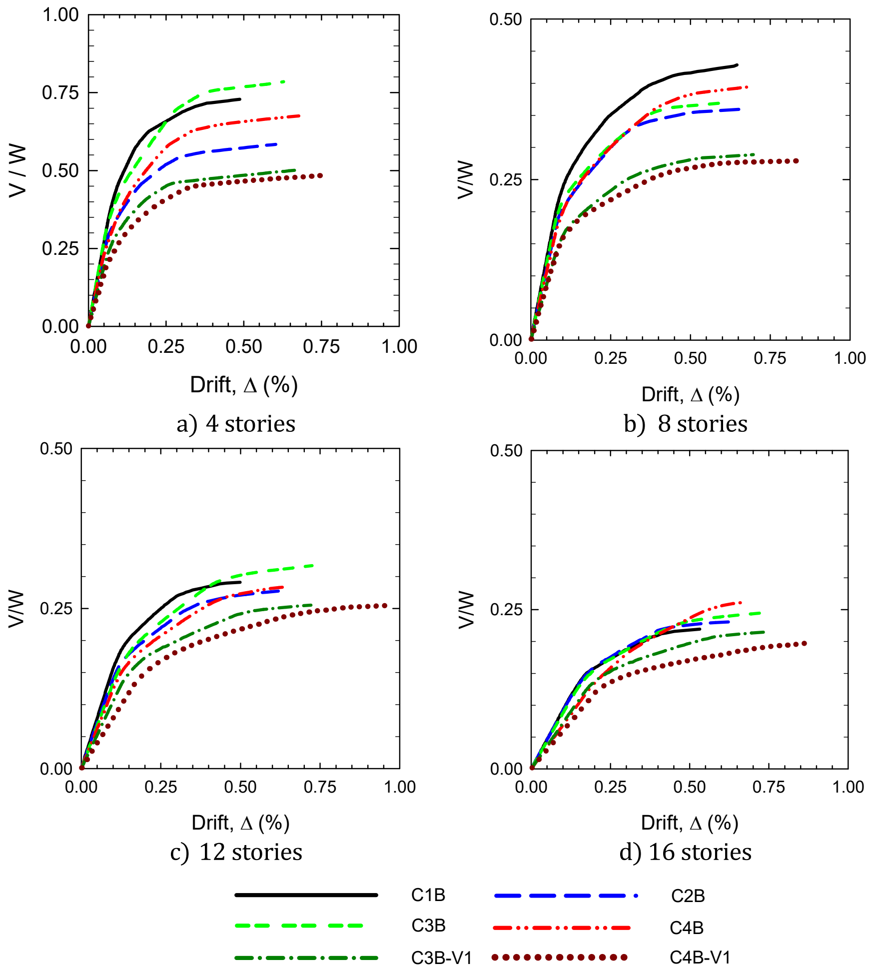


Figure 8: Base shear *vs* global drift curves for the models under study.

In order to try to have a better understanding and to ease qualitatively the impact of having more bays (more redundancy) in the relative deformation capacity for the system (ductility), the obtained global pushover curves were normalized following the criteria proposed by Tena-Colunga and Cortés-Benítez (2015). Therefore, global drifts were normalized with respect to the global drift at the first plastic deformation (or first yielding) for the structure (Δ_{fy}), which usually occurs in braces (or beams in some truss type models), and base shear was normalized with respect to the assumed design base shear V_{DES}=0.10W (Fig. 9). In this case, the corresponding value to the last load step represent directly the overstrength (Ω=V/V_{DES}). According to the cited authors, this double normalization allows one to compare more easily the global behavior of structures for the same or different number of stories, then easing the assessment of redundancy in both deformation capacity (ductility) and overstrength.

The following observations can be done from the normalized curves presented in Fig. 9:

- As expected, for most models where all bays are braced, higher strength (normalized base shear) was obtained respect to those models where only exterior bays are braced. However, in these latter cases, a greater story and global ductility capacities were obtained.
- For the four-story models (Fig. 9a), it is observed that the greater the number of bays, the greater the inelastic deformation capacity. For this frame height, the highest deformation capacity was obtained in models with three and four bays (for the two considered variants). It must be noted that in this case, the results associated with the one-bay model correspond to a load step where a numerical instability was obtained in the DRAIN-2DX software, because, theoretically, all elements still had deformation capacity. However, it was considered that the comparison is valid, since for this model, both the story and global ductility levels are greater than the used as the design value (Q = 4, Figure 11a).

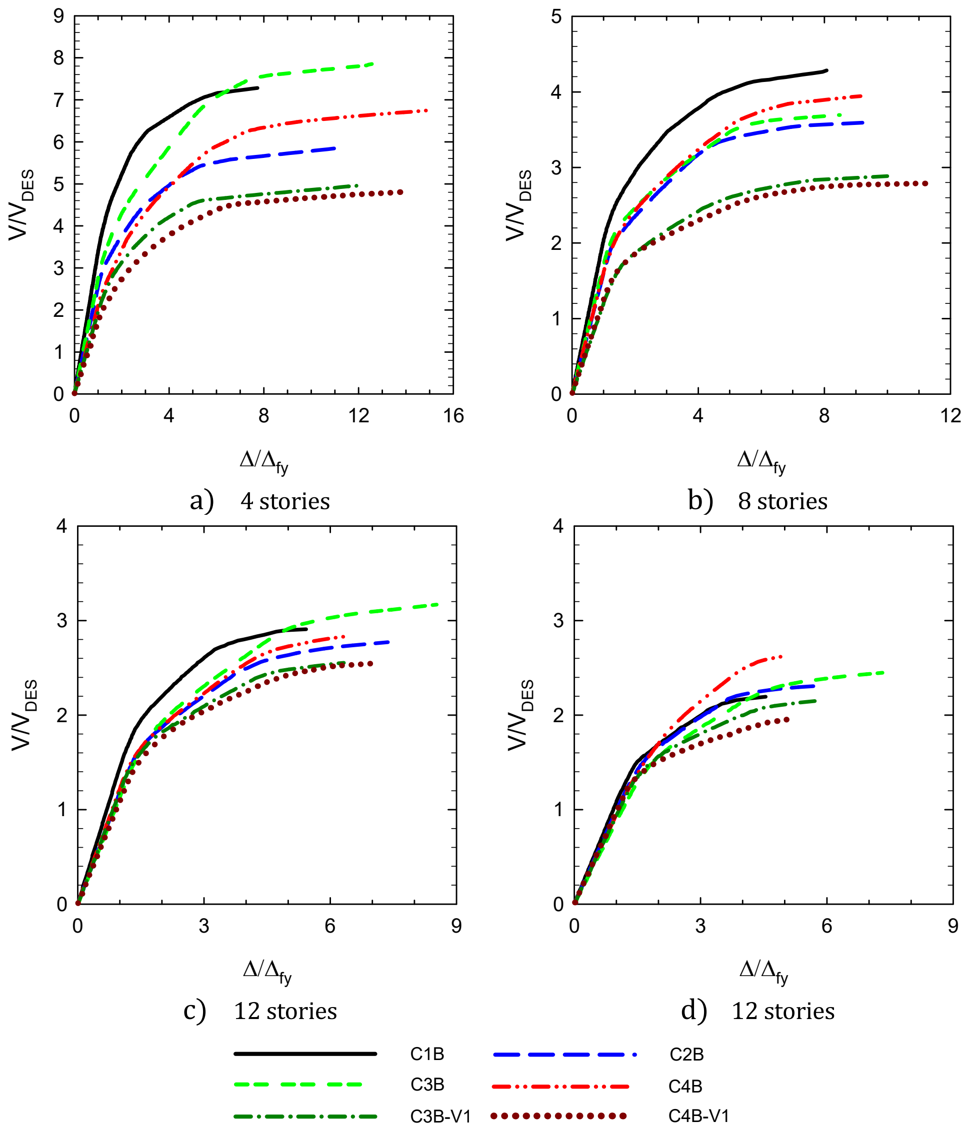


Figure 9: Normalized base shear vs global drift curves for the models under study.

- For the eight-story models, if models which have all bays braced are considered only, it is observed that for multi-bay frames (2, 3 and 4 bays) the ductility capacity is slightly greater than for one-bay fames.
- As the number of stories increases (12 and 16 story models), it is more notorious that ductility increases as the number of bays increases from one to three. Nevertheless, four-bay models have smaller ductility than for two-bay and three-bay models, but always greater than those obtained for one-bay models. This is because for 12 and 16-story models, the computed global drifts values associated to the first plastic deformation for the structure (or first yielding, Δ_{fy}), increases as the number of bays increases, which is opposite to what is observed in four and eight-story models, where this value has just a little variation when the number of bays increases (Fig. 10). Because of this, in some cases, although from the global capacity curves (Fig. 8), it is observed that the deformation capacity increases as the number of bays increases, in the normalized curves (Fig. 9) the same effect is not always observed, since for similar peak drifts values, an increase in the Δ_{fy} value leads to a decrease of the ratio Δ/Δ_{fy}.

In Figures 10-12, full symbols are used to plot the results for models where all bays are braced, whereas open symbols are used to plot the results for models where only the exterior bays are braced.

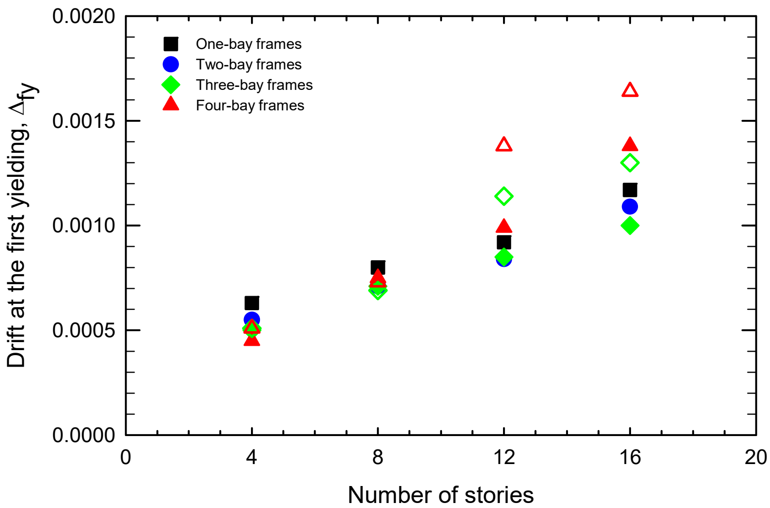


Figure 10: Variation of the global drift at the first plastic deformation (or first yielding) as a function of the number of stories.

Regarding strength, in most cases, buildings where one-bay frames are used have greater values than those obtained for multi-bay frames (2, 3 and 4 bays), with the exception of the 16-story models, where for the frames with all braced bays, the strength capacity increases as the number of bays increases.

From the results presented in this section, except for the eight-story models, the beneficial effect of increasing the number of lines of defense (number of bays) in the ductility capacity is evident.

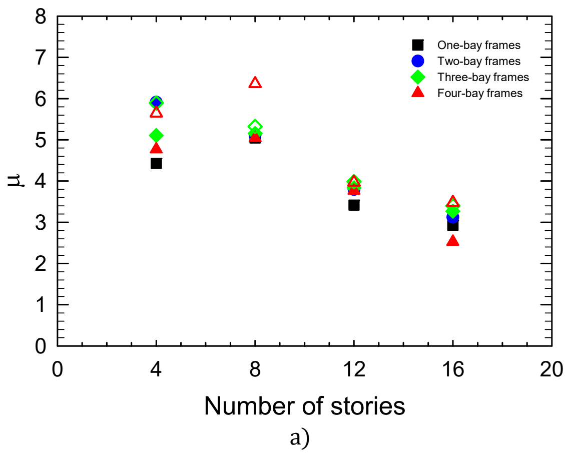
5.4 Developed Ductility and Overstrength

Peak global ductility capacities ($\mu_{global} = \Delta/\Delta_y$) were determined from the global pushover curves of the whole buildings (normalized base shear *vs* average drift computed from the roof displacement over the height of the structure). They are plotted for each studied model in Figure 11a. The equivalent story drift at yielding (Δ_y) was computed from a bilinear idealized curve of the actual force-displacement response curve defined according to what it is already proposed in the literature (Newmark and Hall 1982, FEMA-273 1997). Also, overstrength capacity (Ω) was assessed from the global pushover curves, and were defined as the ratio of peak base shear strength to the design base shear ($\Omega = V_u/V_{des}$, Fig. 11b).

It can be observed from Figure 11a that for all the four-story and eight-story models, the developed global ductility capacities are greater than the deformation demands assumed in the original design ($\mu_{global} > Q = 4$), whereas for the 12-story and 16-story models, $\mu_{global} \leq Q$, regardless of the number of bays considered (Figure 11a).

The observed variation of ductility capacity and overstrength shown in Figures 11a and 11b, where the larger the number of stories (slenderness ratio), the smaller the ductility and overstrength, are consistent with the results of previous research studies of this particular structural system (Godínez-Domínguez and Tena-Colunga 2010).

From the results corresponding to the four-story models, where the final design is strongly influenced by the gravitational loads and the strength/stiffness required balances (Eq. 4), it is observed that the global ductility does not increases as the numbers of bays increases. Nevertheless, it is worth noting that multi-bay frames developed higher ductility capacities when compared to one-bay frames (Fig. 11a).



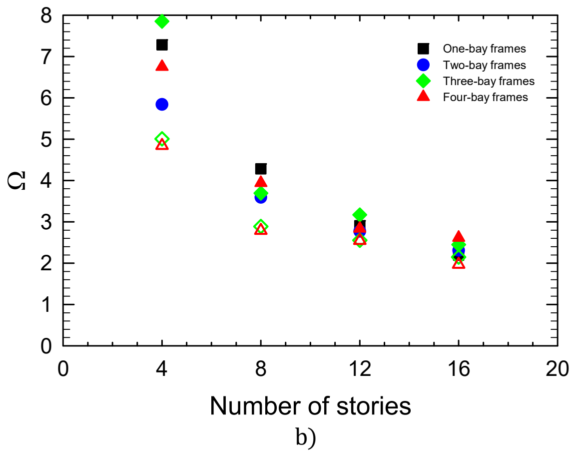


Figure 11: a) Global ductility (μ) and, b) overstrength (Ω) developed for the models under study.

From the results obtained for eight-story models, where earthquake lateral loading started to rule the design of most structural members, it can be observed that, in general, ductility increases as the number of bays increases. In this case, the three-bay and four-bay models have the higher ductility capacities. The same observation can be done for 12-story models. For 16-story models, where lower ductility capacities are obtained respect to those obtained for models of smaller height, it is observed again the beneficial effect of increasing the number of lines of defense (more redundant frames), because in all cases, the ductility capacities for multi-bay models are higher than one-bay models. However, for this particular height, unexpectedly, the smallest ductility capacity was obtained in the four-bay model C4B-16S, where all bays are braced. This is because, as it can be seen from global capacity curves (Fig. 8d), although in model C4B-16S a ultimate drift capacity comparable to the more ductile models is achieved, the corresponding drift at yielding, Δ_y, is greater than those obtained for the rest of the 16-story models (8.1% higher than the next lower value and 47.3% greater than the minimum registered), so that lower ductility is achieved. In general, the value of the global drift at yielding, Δ_y, increases as the number of bays increases, and also increases as the height of the frame increases.

As commented above, it is important to note that, for all heights, higher ductilities are obtained for dual systems (models where only the exterior bays are braced) respect to those obtained for models where all bays are braced.

Regarding overstrength, if only models with all bays braced are considered, it was observed that for most cases, frames with one-bay, three-bays and four-bays have higher values than those obtained for two-bay models, except for 16-story models, in which the overstrength increases as the number of bays increases. As it was expected, for models with all bays braced, higher overstrength are developed respect to those obtained for models where only the exterior bays are braced (Figs. 9 and 11b).

6 ASSESSMENT OF REDUNDANCY FACTORS

The adopted criterion in this study to assess the redundancy factors, which is briefly described in following section, is fully based on the previously research conducted by Tena-Colunga and Cortés-Benítez (2015), where a parametric study devoted to assess the impact of increasing the structural redundancy for the seismic design of ductile moment resisting reinforced concrete frames was developed, as it was described in previous sections.

It is clear from the results presented in previous sections that redundancy impacts in different proportions ductility and strength capacities for RC-MRCBFs (Fig. 11), which it is not yet considered in MOC-2008 code (MOC-2008 2009, Tena-Colunga *et al.* 2009). Therefore, two different redundancy factors were assessed taking into account both the current definition of MOC-2008 and the criterion proposed by Tena-Colunga and Cortés-Benítez (2015): ϱ_μ to assess the impact of redundancy in the ductility capacity, and ϱ_Ω to assess the impact of redundancy in the strength capacity.

Therefore, in order to assess ϱ_Ω according to the current definition of MOC-2008, the developed overstrength $\Omega_{\#\text{bay-N}}$ obtained for one-bay or multi-bay frames (# varies from 1 to 4 in this study) for the N story model (N =4, 8, 12 and 16 in this study) was normalized with the developed overstrength $\Omega_{2\text{bay-N}}$, obtained for the two-bay frame for the same N story model (Eq. 5). The results obtained for ϱ_Ω are shown in Fig. 12a. In the same way, to assess ϱ_μ according to the current definition of MOC-2008, developed ductilities $\mu_{\#\text{bay-N}}$ and $\mu_{2\text{bay-N}}$ (defined similarly) were used (Eq. 6). The results obtained for ϱ_μ are shown in Fig. 12b.

$$\rho_{\Omega} = \frac{\Omega_{\#bay\text{-}N}}{\Omega_{2bay\text{-}N}} \tag{5}$$

$$\rho_{\mu} = \frac{\mu_{\#bay\text{-}N}}{\mu_{2bay\text{-}N}} \tag{6}$$

It is clear from Eqs. 5 and 6 that for two-bay models, $\varrho_\Omega = \varrho_\mu = \varrho = 1.0$, as currently defined in MOC-2008 code.

Comparing the assessed values for ϱ_Ω with respect to the proposed ϱ values in MOC-2008 (Fig. 12a), it is observed that the three-bay and four-bay models do not usually reach the proposed value ϱ =1.25. In fact, the only value greater than that was ϱ_Ω =1.34 for the three-bay four-story model (C3B-4S); and for the rest of cases ϱ_Ω reaches a maximum value of 1.16 (for the four-bay 16-story model, C4B-16S). For one-bay models, where ϱ = 0.8 it is proposed in MOC-2008; it was clearly observed that ϱ_Ω decreases as the number of stories increases. That is, ϱ_Ω decreases as earthquake loading started to rule the design of most structural members. In this case, ϱ_Ω decreases from a computed value of 1.19 for the eight-story model to a minimum value ϱ_Ω =0.95 for the 16-story model. Therefore, it can be concluded that from the strength viewpoint, in RC-MRCBFs where buckling is allowed in steel bracing members, redundancy has a smaller impact than the one anticipated in MOC-2008 code.

The above results reasonably agree with those reported by Tena-Colunga and Cortés-Benítez (2015), where the same structural redundancy factors were evaluated for ductile moment resisting reinforced concrete framed buildings (RC-MRFs), reporting a minimum value ϱ_Ω = 0.9 for one-bay frames, and a maximum value ϱ_Ω = 1.20 for three-bay and four-bay models. However, the tendency of the results obtained in this study is opposite to the previous reported for RC-MRFs, because in RC-SMFs, it was found that ϱ_Ω tends to increase when the number of bays and stories increases.

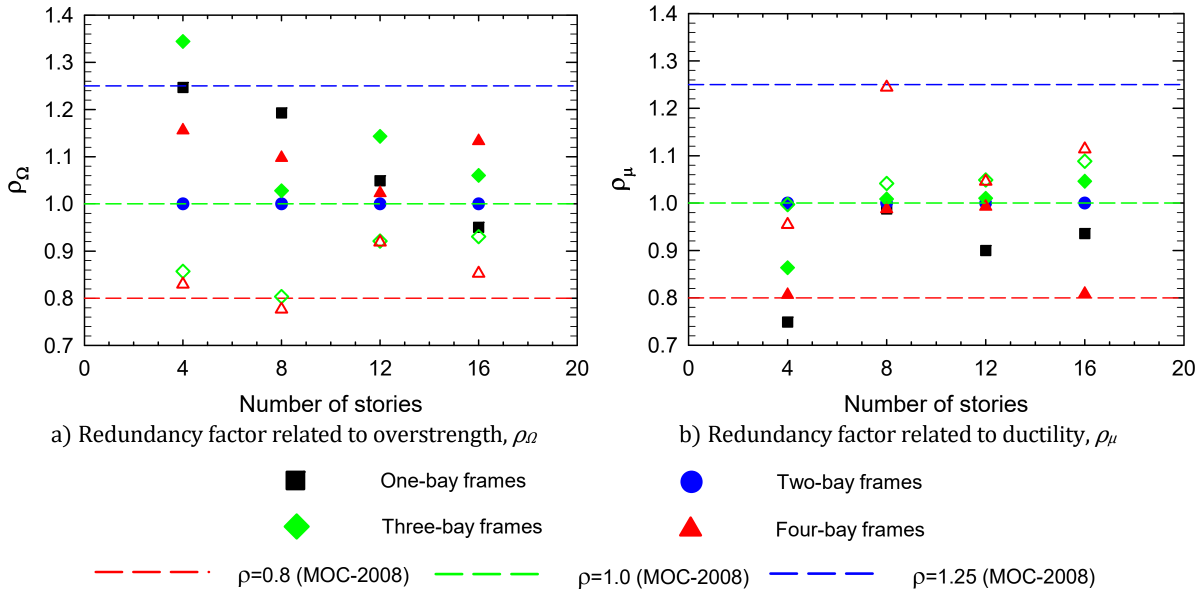


Figure 12: Assessed redundancy factors.

Similar general tendencies are observed for ϱ_Ω (Figure 12a) and ϱ_μ (Figure 12b). For one-bay models, it is observed that assessed values for ϱ_μ are higher than the proposed ϱ =0.8 value in MOC-2008 for the 8, 12 and 16-story models, where earthquake loading usually rule the design of most structural members (with an average value $\varrho_\mu = 0.94$). It is worth noting that a smaller value than the proposed ϱ =0.8 was only obtained for the four-story model ($\varrho_\mu = 0.75$). As expected, the assessed values for ϱ_μ in three-bay and four-bay models are smaller than the proposed ϱ =1.25 value in MOC-2008. The corresponding peak values were $\varrho_\mu = 1.09$ and $\varrho_\mu = 1.24$ for the three-bay and four-bay models, respectively. The above results are smaller than those reported by Tena-Colunga and Cortés-Benítez (2015) for RC-SMRFs. This makes sense, since RC-MRCBFs are generally stronger but less ductile than RC-SMRFs. In addition, due to the ϱ_μ factors were obtained from developed global ductilities, it should be considered that, in general, the computed values for the global drift at yielding (Δ_y), increases as the number of bays increases, thereby the ductility decreases as the number of bays increases ($\mu = \Delta/\Delta_y$).

7 CONCLUDING REMARKS

The results of a parametric study devoted to assess, using nonlinear static analyses, the impact of increasing the structural redundancy in ductile moment-resisting reinforced concrete concentric braced frames structures (RC-MRCBFs) were presented. Steel chevron bracing susceptible to buckling was considered. Among the studied variables were the number of stories (4, 8, 12 and 16) and the number of bays (1, 2, 3 and 4). RC-MRCBFs were assumed to be located in soft soil conditions in Mexico City and were designed for a design base shear ratio V/W=0.10 using a capacity design methodology adapted to general requirements of the seismic, reinforced concrete and steel guidelines of Mexican Codes.

Based upon the limitations of the described research, the following can be concluded from the results obtained in this study:

- As previously reported by Tena-Colunga and Cortés-Benítez (2015) for RC-SMRFs, it was confirmed that strength and deformation capacities of RC-MRCBFs are also impacted by redundancy. Therefore, the current methodology proposed in MOC-2008 (and in some international building codes, i.e. ASCE-7-2010), where redundancy is directly taken into account during the seismic design process using a redundancy factor, ϱ, seem to be adequate.
- When increasing the number of bays (higher redundancy) of RC-MRCBFs, a favorable effect was observed in the structural behavior, because a better distribution of the inelastic deformations along the width and height of the frames was observed, which leads to obtain stable collapse mechanisms.
- In general, for RC-MRCBFs, the impact of redundancy is higher for their strength capacity rather than for their ductility capacity. For models where all bays are braced, the following values were obtained: ϱ_Ω =1.25 and ϱ_μ =0.81 for the four-story models; ϱ_Ω =1.11 and ϱ_μ =0.99 for the eight-story models; ϱ_Ω =1.07 and ϱ_μ =0.97 for the 12-story models and; ϱ_Ω =1.05 and ϱ_μ =0.93 for the 16-story models. It is worth noting that the same impact for ductility and strength is currently considered in the redundancy factor ϱ proposed in MOC-2008 code.

When comparing the proposed redundancy factors for overstrength (ϱ_Ω) and ductility (ϱ_μ) with the ϱ values proposed in MOC-2008, it was observed that, in general, for RC-MRCBFs, in MOC-

2008 the impact of redundancy in both strength and ductility capacities are underestimated for the one-bay frames. Nevertheless, for the three-bay and four-bay frames, the impact of redundancy in both strength and ductility are overestimated in MOC-2008.

For RC-MRCBFs, the proposed redundancy factor for overstrength (ϱ_Ω) tends to decrease as the number of stories increases (or as the earthquake lateral loading started to rule the design of most structural members). In contrast, the proposed redundancy factor for ductility (ϱ_μ) tends to increase as the number of stories increases.

Based on the results of this research, it seems that the redundancy factor ϱ proposed in MOC-2008 code is conceptually in the right direction, but some adjustments would be advisable for improvement, as it seems to depend also on the structural system responsible to resist lateral loads. In this study it was found that the impact of redundancy on the strength capacity of RC-MRCBFs is higher than on their ductility capacity. These results, as expected, are completely opposite to the previously reported for RC-MRCFs structures (Tena-Colunga and Cortés-Benítez 2015), where the impact of redundancy on the ductility capacity is higher than on their strength capacity. It should be considered that only chevron steel bracing has been studied in this research, and perhaps, the assessed redundancy factors may vary if a different bracing arrangement is considered, because as previously reported (Godínez-Domínguez 2014a), overstrength reduction factors (Ω) are dependent on the bracing configuration. Based on these results, and the previous reported in literature, a possible strategy to consider could be the use of different redundancy factors as a function of the structural system, as currently proposed and done in some international building codes for overstrength and ductility reduction factors.

Acknowledgments

The first author gratefully acknowledged financial support of Secretariat of Public Education of Mexico (SEP) as part of the Project No. PROMEP/103.5/13/6999 (UNACH-PTC-136) "Evaluación de factores de reducción por redundancia estructural en sistemas diseñados con base en marcos dúctiles de concreto reforzado con contraventeo metálico tipo chevrón" (in Spanish).

References

ACI 318-14 (2014). Building code requirements for structural concrete (ACI-318-08) and commentary. Farmington Hills. (MI, USA) American Concrete Institute.

Bertero, R. D., Bertero, V. V. (1999). Redundancy in earthquake resistant design. ASCE Journal of Structural Engineering, 125 (1): 81-88.

Bruneau, M., Uang, C. M., Whittaker, A. (1998). Ductile design of steel structures, first edition, McGraw-Hill.

Chopra, A. K., Goel, R. K. (2002). A modal pushover analysis for estimating seismic demands of buildings. Earthquake Engineering and Structural Dynamics, 31: 561-582.

FEMA-273 (1997). NEHRP guidelines for the seismic rehabilitation of buildings. FEMA publication 273. Washington, DC: Federal Emergency Management Agency, October.

Godínez-Domínguez, E. A., Tena-Colunga, A. (2010). Nonlinear behavior of code-designed reinforced concrete concentric braced frames under lateral loading. Engineering Structures, 32: 944-963.

Godínez-Domínguez, E. A., Tena-Colunga, A., Pérez-Rocha, L. E. (2012). Case studies on the seismic behavior of reinforced concrete chevron braced framed buildings. Engineering Structures, 45: 78–103.

Godínez-Domínguez, E. A., Tena-Colunga, A. (2014). Efecto de los modos superiores en la respuesta no lineal de marcos dúctiles de concreto reforzado con contraventeo metálico tipo chevrón. Caso de estudio. Revista Internacional de Ingeniería en Estructuras, 19 (2): 171–181.

Godínez-Domínguez, E. A. (2014a). Influencia de contraventeos concéntricos en cruz o chevrón en el comportamiento no lineal de marcos dúctiles de concreto reforzado. Revista de Ingeniería Sísmica, 91: 1-30 (in Spanish).

Goel, R. K., Chopra, A. K. (2004). Evaluation of modal and FEMA pushover analyses: SAC Buildings. Earthquake Spectra, 20 (1): 225-254.

Husain, M., Tsopelas, P. (2004). Measures of structural redundancy in reinforced concrete buildings. I: Redundancy Indices. ASCE Journal of Structural Engineering, 130 (11): 1651-1658.

IBC-2006 (2006). International Building Code, 2006 Edition. International Code Council, ISBN-13:978-1-58001-302-4.

Kemp, R. A. (1996). Inelastic local and lateral buckling in design codes. ASCE Journal of Structural Engineering, 122 (4): 374-382.

Liao, K. W., Wen, Y. K. (2004). Redundancy in steel moment frame systems under seismic excitations. Report No. UILU-ENG-2004-2010, Department of Civil and Environmental Engineering, University of Illinois at Urbana-Champaign.

MOC-2008 (2009). Manual de diseño de obras civiles, diseño por sismo. Comisión Federal de Electricidad, México, November (in Spanish).

Newmark, N. M., Hall, W. J. (1982). Earthquake spectra and design. Monograph series, Earthquake Engineering Research Institute. Oakland.

NTCC-2004 (2004). Normas Técnicas Complementarias para Diseño y Construcción de Estructuras de Concreto. Gaceta Oficial del Distrito Federal, October (in Spanish).

NTCM-2004 (2004). Normas Técnicas Complementarias para Diseño y Construcción de Estructuras Metálicas. Gaceta Oficial del Distrito Federal, October (in Spanish).

NTCS-2004 (2004). Normas Técnicas Complementarias para Diseño por Sismo. Gaceta Oficial del Distrito Federal, Tomo II, No. 103-BIS, 55-77, October (in Spanish).

Park, R., Priestley, M.J.N., Gill, W. D. (1982). Ductility of square-confined concrete columns", ASCE Journal of Structural Engineering, 108 (4): 929-950.

Prakash, V., Powell, G.H., Fillipou, F. (1992). DRAIN-2DX: Base program user guide. Report No. UBC/SEMM-92/29, Department of Civil Engineering, University of California at Berkeley.

Rodríguez, M., Botero, J.C. (1995). Comportamiento sísmico de estructuras considerando propiedades mecánicas de aceros de refuerzo mexicanos. Revista de Ingeniería Sísmica, SMIS, 49: 39-50.

Song, S.H., Wen, Y.K. (2000). Structural redundancy of dual and steel moment frame systems under seismic excitation. Structural Research Series No. 631, Department of Civil and Environmental Engineering, University of Illinois at Urbana-Champaign, November.

Tapia-Hernández, E., Tena-Colunga, A. (2014). Code-oriented methodology for the seismic design of regular steel moment resisting braced frames. Earthquake Spectra, 30 (4): 1683-1709.

Tena-Colunga, A. (2004). Evaluation of the seismic response of slender, setback RC moment-resisting frame buildings designed according to the seismic guidelines of a modern building code. Proceedings, 13th World Conference on Earthquake Engineering, Vancouver, Canada, Paper No. 2027, CD-ROM, August.

Tena-Colunga, A., Mena-Hernández, U., Pérez-Rocha, L.E., Avilés, J., Ordaz, M., Vilar J.I. (2009). Updated seismic design guidelines for buildings of a model code of Mexico. Earthquake Spectra, 25 (4): 869-898.

Tena-Colunga, A., Cortés-Benítez, J.A. (2015). Assessment of redundancy factors for the seismic design of special moment resisting reinforced concrete frames. Latin American Journal of Solids and Structures, 12: 2330-2350.

Tsopelas, P., Husain, M. (2004). Measures of structural redundancy in reinforced concrete buildings. II: Redundancy response modification factor (R_R). ASCE Journal of Structural Engineering, 130 (11): 1659-1666.

UBC-97 (1997). Uniform Building Code, 1997 edition. International Conference of Building Officials, Whittier, California, Vol. 2.

Wallace, J., Moehle, J. (1989). BIAX: A computer program for the analysis reinforced concrete sections. Report No. UCB/SEMM-89/12, Department of Civil Engineering, University of California at Berkeley.

Wang, C-H., Wen, Y. K. (2000). Evaluation of pre-Northridge low-rise steel buildings Part I, Modeling. ASCE Journal of Structural Engineering, 126 (10): 1160-1169.

Whittaker, A., Hart, G., Rojahn, C. (1999). Seismic response modification factors. ASCE Journal of Structural Engineering 125 (4): 438-444.

ETABS (2013). ETABS Nonlinear Version 9.0.0. Extended 3D analysis of building systems. Computer and Structures, Inc., Berkeley, California.

ASCE 7-10 (2010). Minimum design loads for buildings and other structures. ASCE Standard ASCE/SEI 7-10, American Society of Civil Engineers, ISBN 978-0-7844-1085-1