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Assessment of Redundancy Factors for the Seismic Design of Special Moment Resisting Reinforced Concrete Frames

Abstract

A parametric study devoted to assess the impact of increasing the structural redundancy in ductile reinforced concrete (RC) moment framed buildings is presented. Among the studied variables were the number of stories and the number of bays. Studied models were 4, 8, 12 and 16-story frames with a story height h=3.5 m (11.5 ft). Nonlinear static analyses were used to evaluate numerically redundancy factors. Based on the results of this research and previous studies reported in the literature, it can be concluded that it is justified to account directly structural redundancy in the design by using a redundancy factor, as proposed and done in some international building codes.

Keywords

structural redundancy, redundancy factor, seismic codes, reinforced concrete moment frames, ductility, overstrength.

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1 INTRODUCTION

Nowadays, building construction in large cities worldwide is dominated by architectural needs of providing larger spaces in relatively reduced land spaces because of the high prices for the land in business and residential districts. Big cities in very active seismic regions are not exempt of this tendency. Often, building developers want to implement similar solutions than the ones they used in non-seismic regions, including architectural and structural projects. Therefore, it is common today in big cities of active seismic regions that several new building projects based upon moment frames do have fewer frames with fewer bays, this is, buildings have weakly-redundant structural systems under lateral loading.

The practice of using weakly redundant structures in seismic regions is not entirely new. It has been used for decades, as a solution for architectural needs related to land space constraints. It is worth noting that the seismic performance of such buildings during past earthquakes has been poor. In particular, buildings where one-bay frames are used in the slender direction have had poor performances during past earthquakes. Besides being weakly redundant, this structuring also favors amplified earthquake responses because of the global slenderness for the building and the slenderness for the plan. Just as illustrating examples, an acknowledging that the following buildings have other structural deficiencies in addition to the lack of redundancy, one could make reference to the severe damage observed in buildings Petunia (Figure 1a) and Laguna Beach (Figure 1) at Caracas, Venezuela, during the July 29, 1967 Caracas Earthquake (Web Berkeley 2010, Tena 2010), or the collapse of Juárez Apartment Building Complex in Mexico City (Figure 2), during the September 19, 1985 Michoacán Earthquake. Juárez apartment buildings were also slender in plan and elevation (Figure 2a), and they were weakly-redundant in the slender direction (one-bay frames only); they finally collapsed in that direction (Figure 2b).

(a) Petunia buildings, with one line of defense (masonry walls) in the short and slender direction. Large plan aspect ratio

(b) Laguna Beach building, with infill frames in the short and slender direction. Large plan aspect ratio.

Figure 1: Weakly-redundant buildings in the short and slender direction that experienced important structural damage during the July 29, 1967 Caracas Earthquake. Pictures taken from the Karl Steinbrugge collection (Web Berkeley 2010).

(a) Structural system, Juárez Apartment Building Complex.

(b) Collapse of building C-4.

Figure 2: Collapse of Building C-4 of the Juárez Apartment Building Complex in Mexico City during the 1985 Michoacán earthquake. Pictures and images taken from http://www.arqred.mx/blog/tag/multifamiliar-juarez.

In Figure 3 it is shown another good example on how vulnerable weakly-redundant structures are: the partial collapse of a steel moment frame building with one-bay frames in the slender direction, which occurred during the January 17, 1995 Kobe Earthquake in Japan. It can be observed that the steel building collapsed in the weakly redundant direction, among other reasons, for the apparent pounding with neighboring structures.

It has been learned from experiences of past earthquakes, from analytical and experimental studies that ductility and redundancy are of paramount importance in helping structures to avoid collapses during strong earthquakes, particularly when earthquake demands considerably surpass those assumed in their design. Whereas in the last two decades ductility capacity has received most of the attention of researchers and building code committees worldwide, the impact of redundancy has been oversight. There are just few research studies available (Feng and Moses 1986, Frangopol and Curley 1987, Fu and Frangopol 1990, Paliou *et al.* 1990, Bertero and Bertero 1999, Whittaker *et al.* 1999, Song and Wen 2000, Husain and Tsopelas 2004, Tsopelas and Husain 2004, Tena-Colunga 2004, Fallah *et al.* 2009) 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 (UBC-97 1997, IBC-2006, ASCE-7 2010) and recently in Mexico (MOC-2008 2009, Tena-Colunga *et al.* 2009). Therefore, there is a need to further evaluate the impact of redundancy in the seismic design and behavior of structural systems, as well as recommendations currently available in some design guidelines and building codes.

Figure 3: Collapse of a weakly redundant steel building in the slender direction during the 1995 Kobe Earthquake. Picture taken from the web site http://www.eqe.com/publications/kobe/kobe.htm.

The results of a parametric study devoted to assess the impact of increasing the structural redundancy in ductile reinforced concrete (RC) special moment-resisting framed buildings as defined in Mexican codes is presented in following sections, as well as the assessment of the redundancy factor currently proposed in MOC-2008 code (MOC-2008 2009, Tena-Colunga *et al.* 2009).

2 PREVIOUS RESEARCH

Despite the fact that redundancy has been acknowledged important for satisfactory performances of many structures during past earthquakes, it has received very little attention from the research community in past decades. Most of the available studies have a probabilistic focus (Feng and Moses 1986, Frangopol and Curley 1987, Fu and Frangopol 1990, Paliou *et al.* 1990, Bertero and Bertero 1999, Song and Wen 2000, Fallah *et al.* 2009), a mixed-one, both deterministic and probabilistic (Bertero and Bertero 1999, Whittaker *et al.* 1999, Husain and Tsopelas 2004, Tsopelas and Husain 2004) or only deterministic (Tena-Colunga 2004).

Bertero and Bertero (1999) studied the effects of redundancy on the probability of structural failure, emphasizing its relationships with the overstrength and ductility ratios. They discussed about the inherent difficulties that exist in defining and quantifying the effects of redundancy in the particular context of earthquake-resistant design and proposed two different definitions of redundancy. Among their very interesting results and observations, they concluded that although redundancy can result in several beneficial effects on the earthquake response, a component of force reduction factors of building codes due to redundancy cannot be established independently of the overstrength and ductility of the structural system.

Whittaker *et al.* (1999) mentioned that in 1986 Bertero recommended four lines of strength- and deformation-compatible vertical seismic framing in each principal direction of a building as the minimum necessary 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 highlighted the need to use elements of similar strength and stiffness to all lines of vertical seismic framing in a building to ensure that each frame contributes somewhat equally to the response of the building in a design earthquake. Based upon very simple calculations, they preliminary proposed draft values for a redundancy factor, which were not intended for implementation in seismic codes or guidelines, but "to stimulate discussion among design professionals and researchers, and to promote research and study".

Song and Wen (2000) studied the redundancy of dual systems and special moment resisting frames (SMRF) in terms of system reliability under SAC project ground motions. They considered as variables the structural configuration (number and layout of shear walls and moment frames), the ductility capacity and the uncertainties in demands and capacities. 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.

Husain and Tsopelas (2004) and Tsopelas and Husain (2004) presented a method, based on pushover analysis, to quantify the deterministic and probabilistic effects of redundancy on the strength of structural systems. They proposed two indices, the redundancy strength index and the redundancy variation index, which were evaluated for plane reinforced concrete frames with different stories (3, 5, 7 and 9), different number of vertical lines of resistance or bays (1, 2, 4 and 6), and various theoretical beam ductility capacity ratios (1.5 to 16). They concluded that the strength redundancy modification factor, R_R, 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 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.

The method proposed by Husain and Tsopelas (2004) and Tsopelas and Husain (2004) is robust from a research-oriented viewpoint. However, from an every-day design practice viewpoint, it has the disadvantage of requiring to perform detailed nonlinear analyses and using probabilistic concepts, which makes their method no very appealing for most practicing engineers, even today. Practicing engineers still prefer to use simple global design parameters that could be easily defined in terms of simple geometric and structural variables, such as the number of frames resisting in the direction of interest, and the number of bays that compose each moment frame.

3 BUILDING CODES

To the author´s knowledge, the first building code to directly include a redundancy factor (ρ) for the seismic design of buildings was the 1997 UBC Code (UBC-97 1997). The original proposal of UBC-97 has changed in the most recent recommendation of US Codes (IBC-2006 2006, ASCE-7 2010).

3.1 ASCE-7

In ASCE-7 (2013), the redundancy factor (ρ) is taken into consideration at the time of assessing the horizontal seismic load effect, E_h, as:

$$E_h = \rho Q_E \tag{1}$$

where Q_E is defined as the effects of horizontal seismic forces from V (total design lateral force or shear at the base) or F_p (the seismic force acting on a component of a structure). The corresponding basic load combinations for strength design and allowable stress design are established in section 12.4.2.3 of ASCE-7 (2010).

As observed, according to ASCE-7, the redundancy factor is taken into consideration to amplify or diminish seismic lateral forces based upon the seismic design category, which is a classification assigned to a structure based on its occupancy and the severity of the design earthquake ground motion at the site, as described in detail in sections 11.6, 11.7 and 11.8 of ASCE-7 (2010). In general, seismic design categories A to C are given to common structures where the earthquake hazard is not high and soil site effects are not very important, whereas seismic design categories D to F are set for structures where the earthquake hazard is higher and soil site effects are important.

Two values are proposed for the redundancy factor in ASCE-7 (2010): a) ρ=1, for all the cases identified in section 12.3.4.1 (among them, structures assigned to seismic design categories B and C), and the exemptions described in section 12.3.4.2 for structures assigned in seismic design categories D to F and, b) ρ=1.3 for structures assigned in seismic design categories D to F, according to section 12.3.4.2. Therefore, the redundancy factor ρ in ASCE-7 is used to magnify horizontal seismic forces in structures found to be in seismic design categories of greater risk, unless one of the two following conditions are met (exemptions to use ρ =1 in seismic design categories D to F):

a) Each story resisting more than 35% of the base shear in the direction of interest, granted it comply with the requirements set in ASCE-7 Table 12.3-3 for the identified lateral-force resisting elements.
b) Structures that are regular in plan at all levels provided that the seismic force-resisting systems consist of at least two bays of seismic force-resisting perimeter framing on each side of the structure in each orthogonal direction at each story resisting more than 35% of the base shear. The number of bays for a shear wall shall be calculated as the length of shear wall divided by the story height or two times the length of shear wall divided by the story height for light-framed construction.

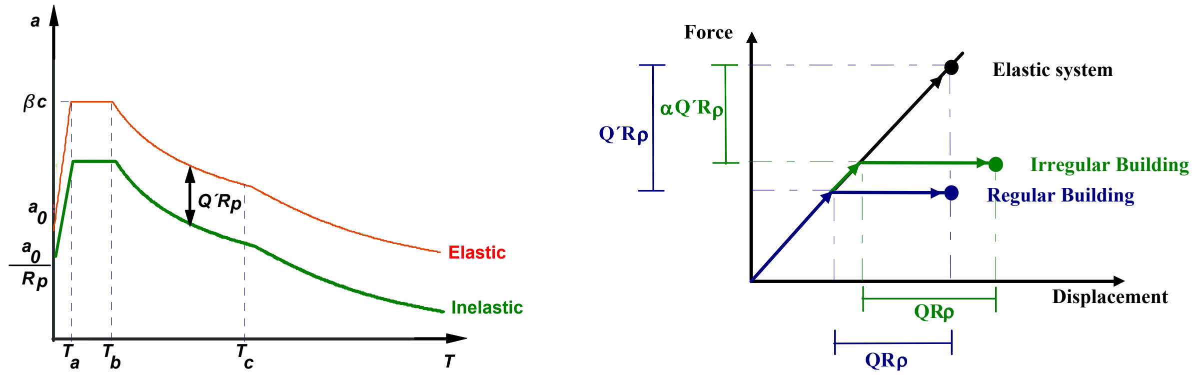
From these definitions, the following general observation is done to ASCE-7 recommendations for the redundancy factor ρ. The lack of redundancy is only penalized for the design of structures where the earthquake hazard is high, according to its seismic design category (D to F). However, the lack of redundancy is not taken into account for the design of structures with one-bay frames in other seismic design categories (A to C). It would be very valuable to know why the ASCE-7 Committee took that decision, but to the authors´ knowledge, there are not documents available that provides specific comments for this topic.

3.2 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 (Figure 4a). In fact, 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 depicted in Figure 4b, 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 30 years (for example, Figures 1 to 3).

The proposed values for ρ in MOC-2008 code are the following:

a) $\rho = 0.8$ 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) $\rho = 1.0$ 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) $\rho = 1.25$ 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).



(a) Schematic representation of inelastic acceleration design spectrum.

(b) Schematic illustration of the design procedure for regular and irregular buildings.

Figure 4: General seismic design criteria for building structures for MOC-2008.

As one can observe, 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 in buildings that among other deficiencies, have one-bay frames (i.e., Figures 1 to 3). Numerical collapses of such structures designed according to modern building codes have also been reported (Tena-Colunga 2004). In addition, smaller overstrength factors have been reported in the literature for such frames.

The structural systems where $\rho = 1.0$ was proposed in MOC-2008 correspond to those considered in most of the consulted studies to define target values for the overstrength factor R. The requirement of having at least two-bay frames or equivalent structural systems was established based upon analyzing the results obtained in previous research studies were redundancy was studied (Husain and Tsopelas 2004, Tsopelas and Husain 2004, Tena-Colunga 2004), and one of ASCE-7 (2010) exemptions for seismic design categories D to F, identified as exemption "b)" in the previous section. The proposal for $\rho = 1.25$ was based in some recent studies where parallel frames of these characteristics have been studied and where higher overstrength factors were obtained (Tena-Colunga *et al.* 2008). It is also worth noting that the values of ρ may vary in each main orthogonal direction.

The assessment of the ρ factor for a given structure is illustrated with the buildings which plans are depicted in Figure 5. For the building plan depicted in Figure 5a, $\rho = 0.8$ should be taken in the Y direction as it has eight parallel one-bay frames, whereas in the X direction, $\rho = 1.0$ because it has two parallel seven-bay frames. In contrast, for the building plan depicted in Figure 5b, $\rho = 1.0$ should be taken in the Y direction as it has eight parallel two-bay frames, whereas in the X direction, $\rho = 1.25$ because it has three parallel seven-bay frames.

The philosophy behind the redundancy factor ρ proposed in MOC-2008 is illustrated in this simple example. *A-priori*, most structural engineers would agree that the building plan depicted in Figure 5b is more redundant than the building plan depicted in Figure 5a. Most seismic codes worldwide do not recognize directly this fact for their seismic design but MOC-2008 (2009). As stated earlier, according to ASCE-7 (2010), the building plan depicted in Figure 5a would only be penalized if it is classified in seismic design categories D to F, and each story resist less than 35% of the base shear in the direction of interest.

It is worth noting that in MOC-2008 code, the design of irregular buildings is penalized using a corrective reduction factor α that modifies the ductility-based force reduction factor Q' (R in US codes), as depicted in Figure 4b. According to MOC-2008, the value of α depends on the degree of irregularity (MOC-2008 2009, Tena-Colunga *et al.* 2009). For buildings found to possess a strong irregularity condition (soft and weak stories or strong torsional coupling), the value for α is 0.7. Therefore, for such buildings, apparent redundant plan configurations are also punished in the design. The effective reduction factor would be: $\alpha Q'R\rho$=(0.7)$Q'R$(1.25)= 0.875$Q'R$≥1.0. Nevertheless, the code committee for MOC is establishing in the next version (under peer review) that, for buildings with strong irregularity condition, $\rho \leq 1.0$.

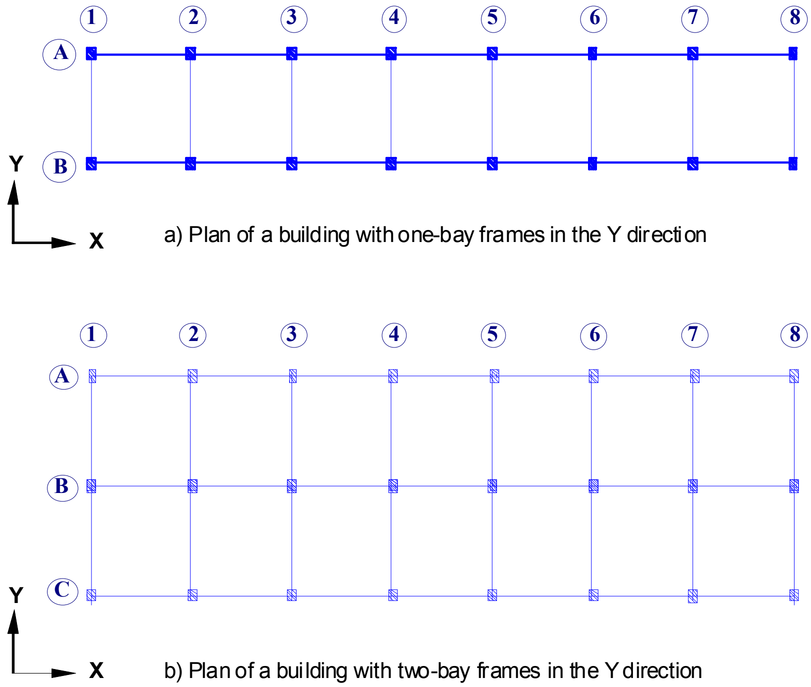


Figure 5: Sample buildings to illustrate the assessment of the ρ factor of MOC-2008.

4 SUBJECT BUILDINGS

The main objective of the research reported herein was to perform a formal assessment of the redundancy factor ρ as proposed in MOC-2008 code (MOC-2008 2009, Tena-Colunga *et al.* 2009) for reinforced concrete special moment resisting frames (RC-SMRFs).

For this purpose, reinforced concrete special moment resisting frames (RC-SMRFs) buildings regular in plan and elevation were initially considered. Studied buildings have the following general characteristics: a) the total width for the plan of the building in the direction of interest (where redundancy was evaluated) was L_{TOT}= 12m (39.4 ft), as depicted in Figure 6, b) the typical story height was h=3.5 m (11.48 ft), c) 4, 8, 12 and 16 stories were considered and, d) 1, 2, 3 and 4 bays were considered. A fixed total width L_{TOT} was considered in this study as it is frequent that for a building project in a big city, available land spaces are generally fixed and constrained in that sense. Then, architects and structural engineers have to decide whether they use one-bay frames or multi-bay frames in one given direction. Besides, Husain and Tsopelas (2004) have already shown the benefits of redundancy when considering that all bays have the same length L and, obviously, if there are no landspace constrains, why do structural engineers would allow architects to use one-bay frames in a given direction?

To have a general benchmark of comparison (for example, avoid a design spectrum dependency), all buildings were designed for a base shear V=0.10W, where W is the total weight for the structure for seismic design. All buildings were designed to fulfill the requirements established by Mexican codes, including all load combinations for seismic loading (MOC-2008 2009, Tena-Colunga *et al.* 2009), and the review of service limit states, strength and detailing requirements for all RC structural elements (Tena-Colunga *et al.* 2008, NTCC-2004 2004). The static method of analysis allowed in MOC-2008 was used, where it is assumed that mass accelerations vary linearly with height; 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 (Figure 4a), as described elsewhere (MOC-2008 2009).

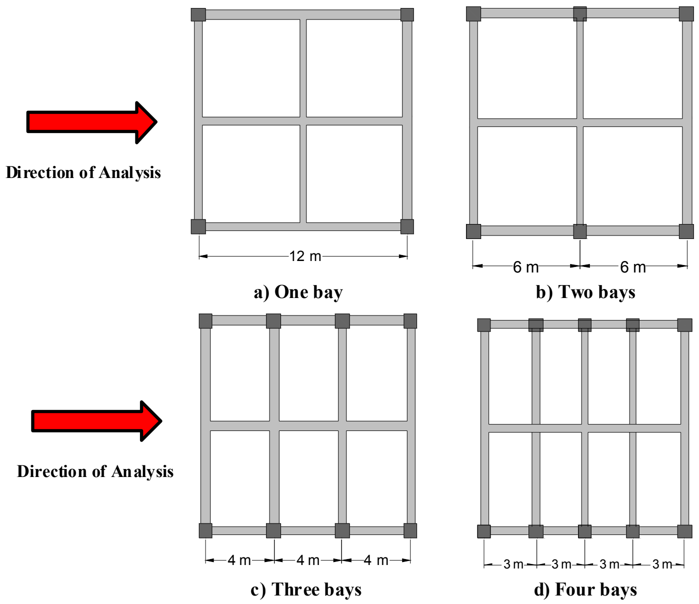
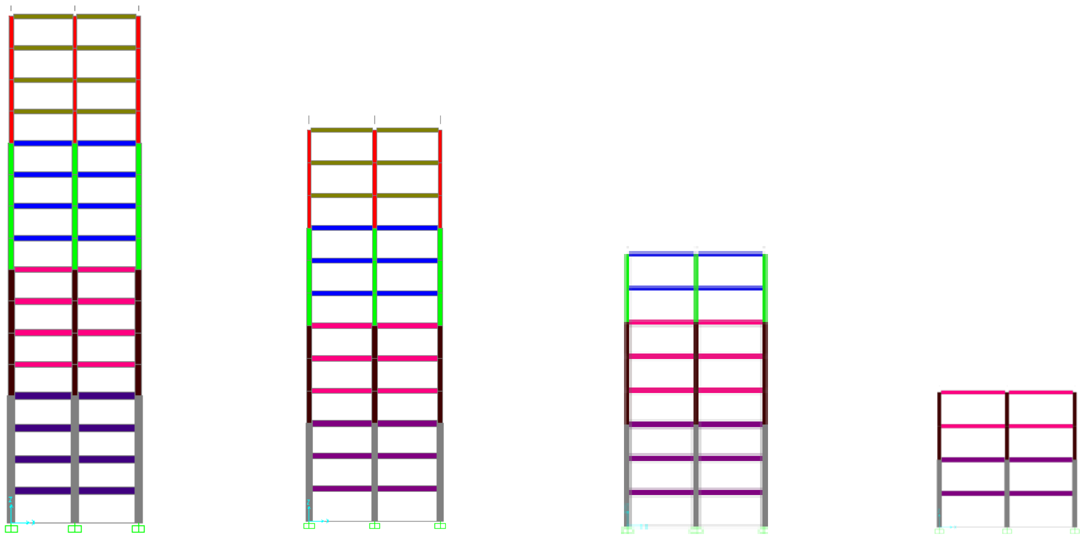


Figure 6: Plan layout for the subject buildings of interest. Squares indicates the location of columns (dimensions in meters).

According to one traditional design practice of many structural engineers in Mexico, the cross sections for beams and columns were typified every M stories, being careful in providing symmetric reinforcement (strength) when defining typical sections in plan and avoiding stiffness irregularities in elevation. The proposed changes of sections for the studied buildings are schematically illustrated in Figure 7. It is worth noting that steel reinforcements vary for interior and exterior beams and columns, particularly for taller buildings, as reported in Table 1.



(a) 16 stories (b) 12 stories (c) 8 stories (d) 4stories

Figure 7: Schematic representation of changes of cross sections for beams and columns for the studied models.

Model	Δ_{max} (%)	ρ^{-}_{beams} (%)	ρ^{+}_{beams} (%)	$\rho_{columns}$ (%)	Model	Δ_{max} (%)	ρ^{-}_{beams} (%)	ρ^{+}_{beams} (%)	$\rho_{columns}$ (%)
M1-4LC	1.65	0.81-1.10	0.41-0.58	1.0-1.3	**M1-12LC**	2.8	1.10-1.25	0.44-0.76	1.2-1.5
M2-4LC	1.3	0.59-0.69	0.33-0.35	1.2-1.4	**M2-12LC**	1.85	1.01-1.19	0.79-0.95	1.2-1.3
M3-4LC	0.9	0.46-0.63	0.32-0.42	1.3	**M3-12LC**	1.8	0.95-1.09	0.87-0.99	1.2-1.5
M4-4LC	0.9	0.64-0.73	0.52-0.58	1.3	**M4-12LC**	1.2	0.97-1.17	0.93-1.13	1.2-1.5
M1-8LC	2.5	0.94-1.18	0.48-0.66	1.0-1.3	**M1-16LC**	2.95	1.00-1.24	0.54-0.89	1.2-1.3
M2-8LC	1.4	0.76-0.92	0.42-0.58	1.0-1.3	**M2-16LC**	2.2	1.09-1.29	0.88-1.21	1.0-1.5
M3-8LC	1.2	0.76-1.03	0.51-0.85	1.2-1.3	**M3-16LC**	1.75	0.93-1.22	0.83-1.19	1.2-1.6
M4-8LC	1.2	0.79-1.04	0.69-0.92	1.2-1.3	**M4-16LC**	1.35	0.92-1.22	0.92-1.12	1.2-2.2

Table 1: Summary for the design of the studied models

The compressive strength for the concrete was f''_c =250 kg/cm^2 (3,551 psi). The elastic modulus for the concrete was estimated as $E_c = 14000\sqrt{f'_c}$ (in kg/cm^2) or $E_c = 4400\sqrt{f'_c}$ (in MPa). Grade 60 steel (f_y=4,200 kg/cm^2) was used for longitudinal and transverse reinforcement. For the columns of all building models, square cross sections were used with a uniform distribution of the longitudinal reinforcement satisfying commercial bar sizes and all detailing requirements of Mexican codes (NTCC-2004 2004). Beams were analyzed and designed as doubly-reinforced T sections in flexure. Gross section properties for the concrete elements were used for stiffness modeling, for all the reasons described in detail in previous works (Tena-Colunga *et al.* 2008). 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 $\mathit{\Delta}$=0.030 (Δ=3%) allowed by MOC-2008 for SMRFs (MOC-2008 2009, Tena-Colunga *et al.* 2009).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.

To complete a glance picture for the overall designs, peak design story drifts (Δ_{max}) and design ranges for the reinforcement ratios for the columns ($\rho_{columns}$) and beams (ρ^{+}_{beams} and ρ^{-}_{beams}) for all the RC-SMRFs building models are summarized in Table 1. It is worth noting that the following notation is used to identify the models in Table 1: *Mi-jLC*, where *i* identify the number of bays and *j* the number of stories. It can be observed from Table 1 that the smallest peak story design drift ratios are obtained for the four story models, because gravity load combinations ruled the design of most elements, beams in particular. As expected, the highest story design drift ratios were generally obtained for the less redundant models (one or two-bay models), as a consequence that their corresponding bay widths are larger (Figure 6).

It can also be observed from Table 1 that in order to insure a ductile behavior for beams and columns from a theoretical viewpoint, special attention was paid in the design process to warrant that steel reinforcement ratios for beams would be mostly below 1.3%, and between 1% (minimum) to 1.6% for columns. All detailing requirements for the longitudinal and transverse steel reinforcement and the ultimate to nominal yield strength ratio (f_u/f_y) established in Mexican codes (NTCC-2004 2004) for RC-SMRFs were also satisfied. To help illustrate the required designed cross sections for columns, exterior columns at the first story varied from 80x80 cm (M1-4LC) and 60x60 cm (M4-4LC) for the 4-story models to 140x140 cm (M1-16LC) to 110x110 CM (M4-16LC) for the 16-story models.

5 NONLINEAR ANALYSES

Nonlinear static analyses (pushover) were conducted for each model under study. All elements (columns and beams) were modeled to monitor the possibility of developing a nonlinear behavior. P-Δ effects were considered in the analyses. For simplicity, the code-based design lateral load distribution profiles (which account for higher modes for flexible structures) were also used in the pushover analysis.

The following assumptions were done for computing nominal capacities for RC beams and columns: (1) the concrete was modeled using a suitable nonlinear modeling of the stress-strain curve for the reinforcement steel was considered. The concrete confinement model selected in this study is the well-known modified Kent-Park model (Park *et al.* 1982) and the stress-strain curve for the reinforcement steel is one proposed for rebars produced in Mexico which is based on the original Mander model (Andriono and Park 1986), (2) the "real" or actual distribution of the steel reinforcement according to the final design was considered and, (3) the contribution of the slab reinforcement in the resisting bending moments of beams was included in the assessment of overstrength capacities. 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 of Mexican codes (NTCC-2004 2004) is successfully implemented in the construction site.

The results obtained from pushover analyses were processed so the last step taken into consideration would correspond to a "real" deformation, instead of taken the results of the last step before numerical instabilities crash program runs. For this purpose, plastic curvatures for beams and columns obtained for a given step of pushover analysis were compared to theoretical moment-curvature curves obtained with BIAX (Wallace and Moehle 1989). Results were processed just until the time step where it was assured that plastic curvatures barely surpassed those obtained with BIAX. This strategy is reasonable, as usually one should ignore from few to several steps at the end of pushover analyses, where numerical instability trigger and therefore, misleading results about the deformation capacity for the structure under study are usually obtained in exchange.

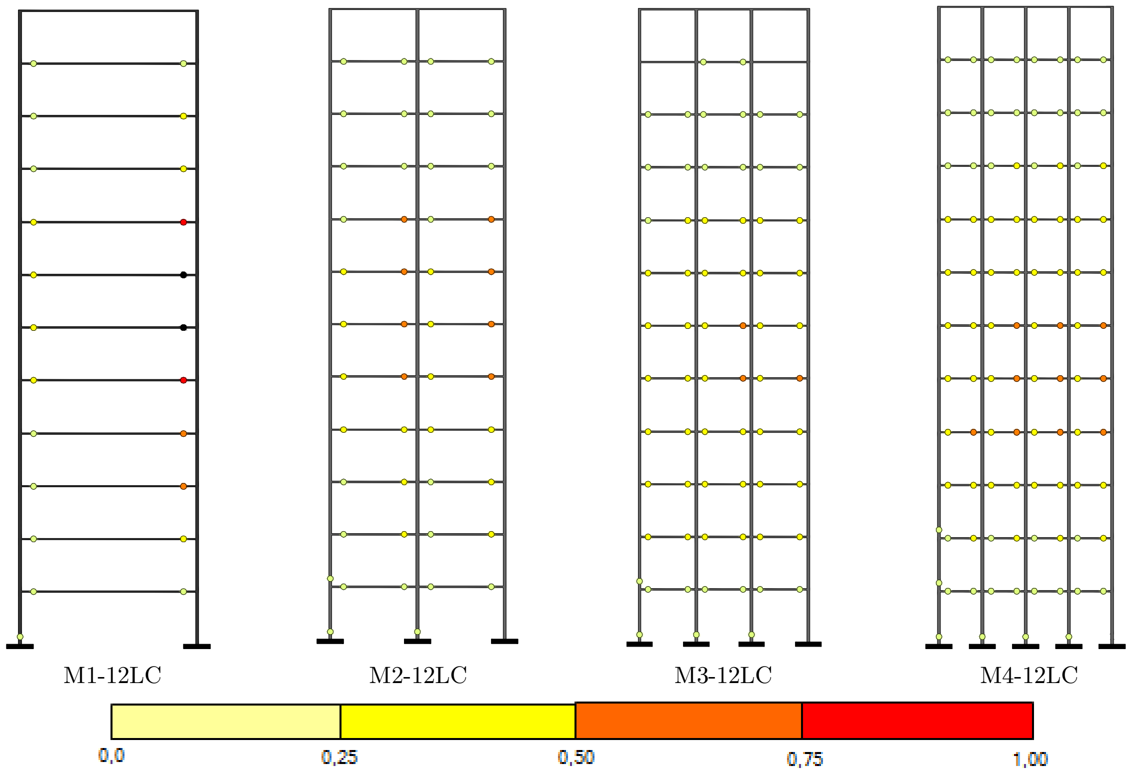


Figure 8: Inelastic yielding mappings for the 12 story models

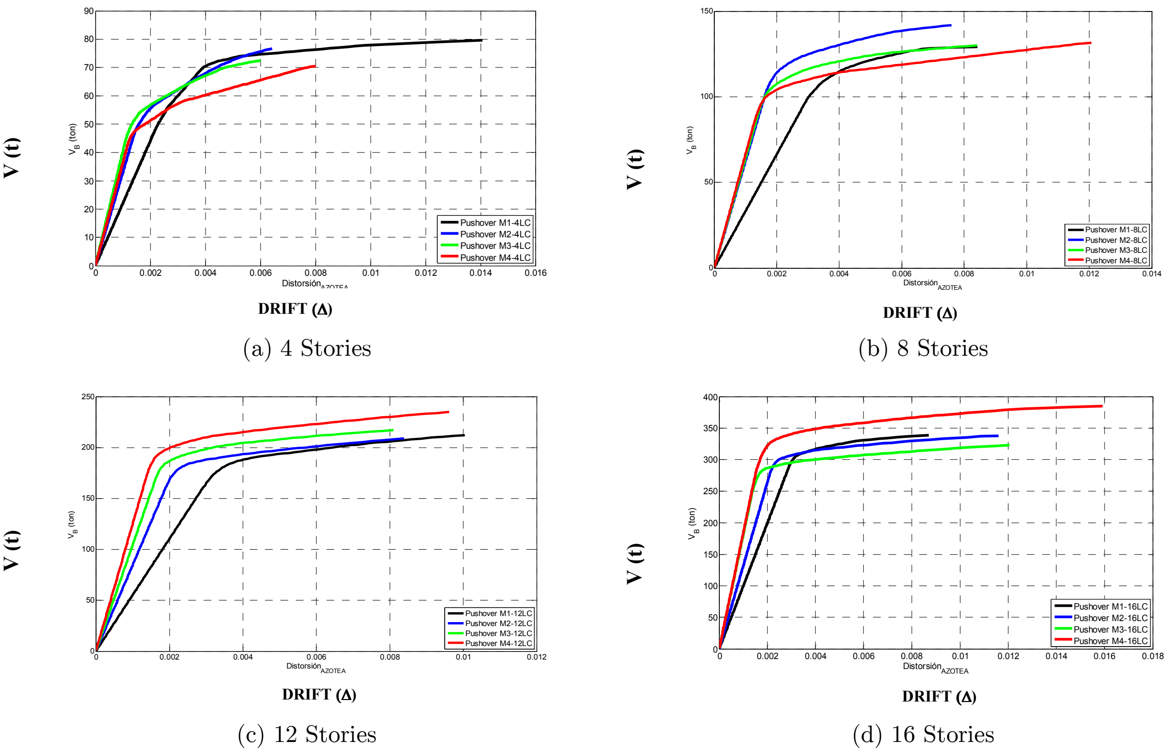
5.1 Yielding mappings

In order to check that the weak beam - strong column design philosophy for RC-SMRFs was achieved, yielding mappings corresponding to the load step where the collapse mechanism is formed were obtained, as shown in Figure 8. A hot color scale was defined to highlight the inelastic demands for beams and columns. No color identifies elastic responses. A mild yellow color identifies nonlinear responses after yielding and up to a reparable damage state ($\phi/\phi_u \leq 0.25$). Strong yellow is used for moderate nonlinear responses ($0.25 < \phi/\phi_u \leq 0.5$). Orange is used for important nonlinear responses ($0.5 < \phi/\phi_u \leq 0.75$). Red is used for nonlinear responses on the descending branch of moment-curvature curves ($0.75 < \phi/\phi_u \leq 1.0$). Black is used when $\phi/\phi_u > 1.0$ (in theory, the element completely failed).

5.2 Base shear vs global drift curves

Base shear vs global drift curves (V 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 9. As expected, it can be clearly observed from these curves that the elastic stiffness for the studied models increases as the number of bays increases. Therefore, from this perspective, it is difficult to qualitatively assess the impact of having more bays (more redundancy, Figure 8) in the relative deformation capacity for the system (ductility).

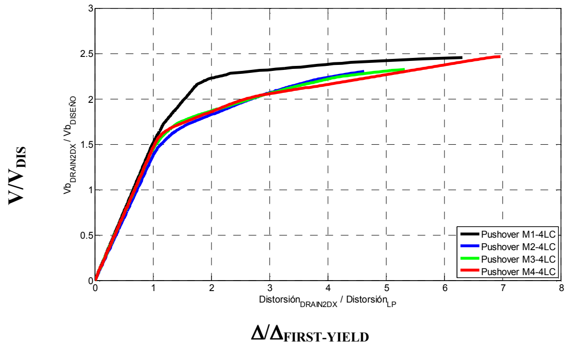
To ease comparisons, obtained global pushover curves were normalized in the following way. Global drifts were normalized with respect to the global drift at the first yielding for the structure ($\Delta_{\text{FIRST-YIELD}}$), which occurs in beams. Base shear was normalized with respect to the assumed designed base shear V_{DIS}=0.10W. Normalized curves are shown in Figure 10. 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.



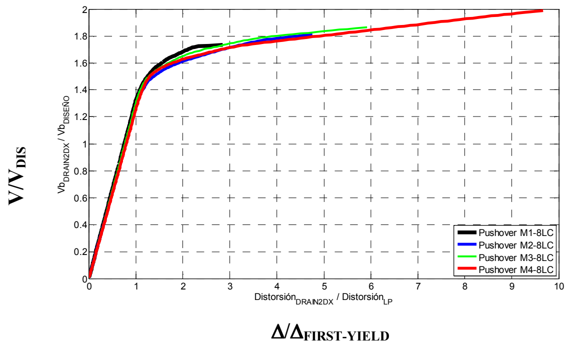
(a) 4 Stories (b) 8 Stories (c) 12 Stories (d) 16 Stories

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

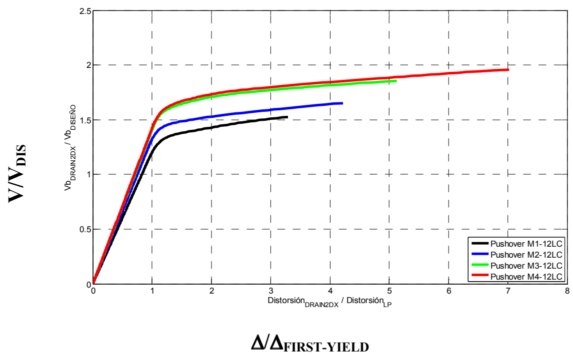
The following observations can be done from the normalized curves presented in Figure 10. For the 4-story models (Figure 10a), it is observed that the one-bay frame (M1-4LC) developed a reasonable strength and deformation capacity. In fact, surprisingly as it may seem, these capacities are even higher than for the two-bay and three bay models. It is worth noting that in model M1-4LC, bending moments in beams due to gravitational loads were relatively high, and this fact impacted the final design in the load combinations for earthquake. Big negative bending moments due to gravitational loads at both beam ends were summed with a negative bending moment due to seismic load at one end and a positive bending moment due to seismic load at the other end. Resulting sums considering alternate seismic loading yielded that a big negative and a very small positive (or even negative) bending moments were obtained for the design of those beams. For ductile RC-SMRFs frames, it is required in international RC building codes that the positive bending moment capacity at beam ends should be at least half the negative bending moment capacity, this is, $M^{+}_{DIS} \geq 0.5M^{-}_{DIS}$. Therefore, for this detailing requirement provided for RC-SMRFs in building codes worldwide, beams for M1-4LC model were "overdesigned" for positive moment. However, this was the reason that allowed this structure to develop an important ductility and strength. One can assume that such deformation and strength capacities would not develop if the frame would have been designed as an ordinary (RC-OMRFs) or intermediate (RC-IMRFs) moment-resisting frame.



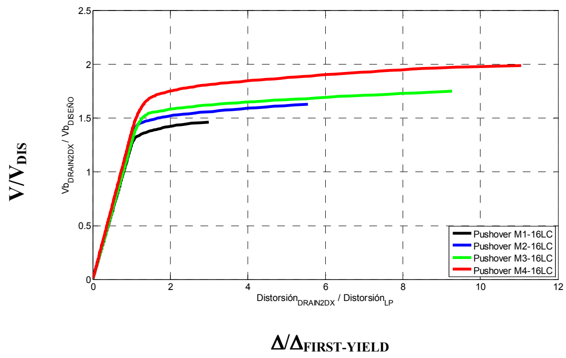
(a) 4 Stories



(b) 8 Stories



(c) 12 Stories



(d) 16 Stories

Figure 10: Normalized base shear vs global drift curves for the models under study.

It can also be inferred from the observation of Figure 10 that earthquake loading started to rule the design of most structural members from eight stories and therefore, more redundant frames (multi-bay frames) exhibited better structural performances than one-bay frames. It is observed for 8-story models (Figure 10b) that for multi-bay frames, the ductility capacity increases more significantly than the strength capacity when compared to one-bay frames. As the number of stories increase, it is more notorious that strength and ductility increase as the number of bays increases, this is, as frames become more redundant (Figure 8). Therefore, it can be concluded from the obtained results that, for the design base shear considered in this study (V/W=0.10), redundancy has a more positive impact for medium-rise RC-SMRFs than for lowrise RC-SMRFs. Also, for RC-SMRFs, possessing a higher redundancy is more important in its ductility capacity than in its strength capacity.

5.3 Ductility and Overstrength

Available ductility and overstrength were obtained from idealized bilinear base-shear vs global drift curves obtained from pushover curves, as schematically depicted in Figure 11.

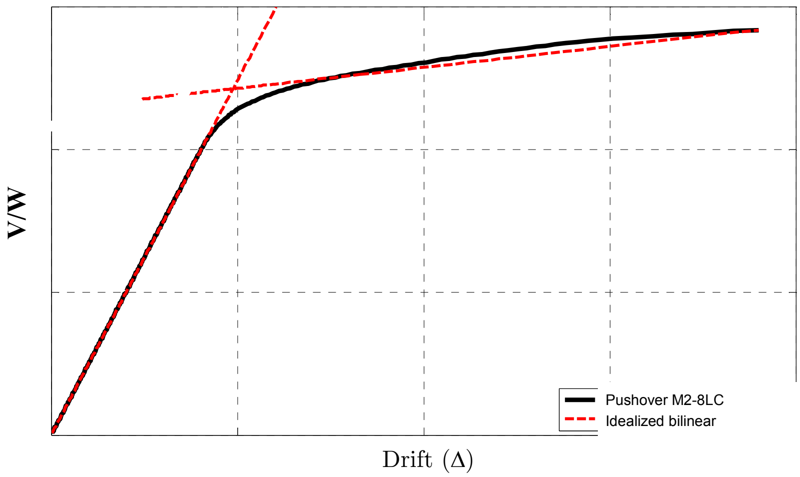


Figure 11: Idealized bilinear base shear vs drift curved obtained from the pushover curve.

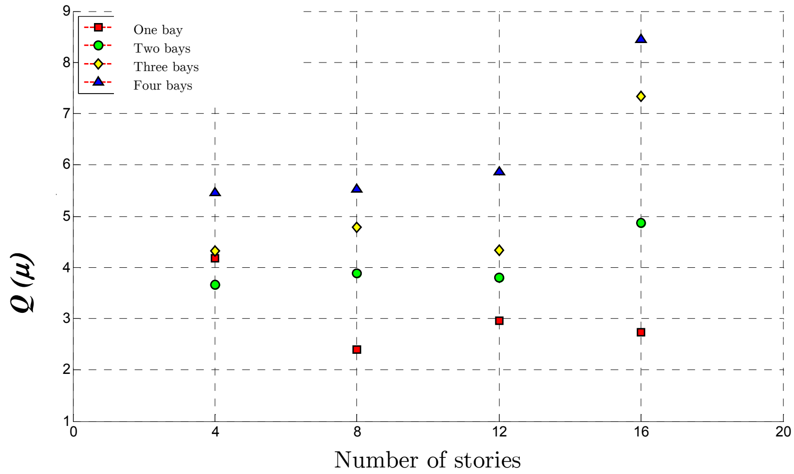


Figure 12: Global ductility developed for the models under study.

Available ductilities for all models under study are shown in Figure 12. It is worth noting that according to Mexican codes, RC-SMRFs could be designed for a global ductility μ=Q=4 (Figure 4). It can be observed that weakly-redundant models (one-bay frames) are not able to develop the peak global ductility Q=4 allowed in Mexican codes for the design, with the exception of the 4-story model (M1-4LC). As discussed earlier, the ductility capacity of the studied RC-SMRFs generally increase as the number of bays and stories increase. One of the reasons behind it is that when more bays are used, the negative and positive bending moment capacities are much closer (this is, for a given beam element, $\rho^+ \approx \rho^-$) and then, the rotation capacity for beams increases at both ends.

Overstrength capacity (Ω) was assessed as the ratio between the peak base shear strength obtained from the pushover curve divided by the design base shear. The available overstrength computed for all models under study are shown in Figure 13. It can be observed from Figure 13 that, with the exception of the one-bay, four-story model (M1-4LC), there is a clear tendency for overstrength. The available overstrength is reduced as: a) the number of bays decreases (redundancy decreases) and, b) the number of stories increases. The difference of the overstrength developed for one-bay frames with respect to the one developed for four-bay frames increases as the number of stories increases. As the impact of gravitational loads was more important in the design of beams for one-bay and two-bay models than for three-bay and four-bay models, it seems reasonable that available overstrength has a smaller variation for four-bay models than for one-bay models, particularly for models eight stories in height or above. In agreement to what it is currently acknowledged in Mexican seismic codes (MOC-2008 2009, Tena-Colunga *et al.* 2009, NTCS-2004 2004), higher overstrength capacities are developed in short-period, lowrise models (four stories), because gravitational loads often rule the design of beam members.

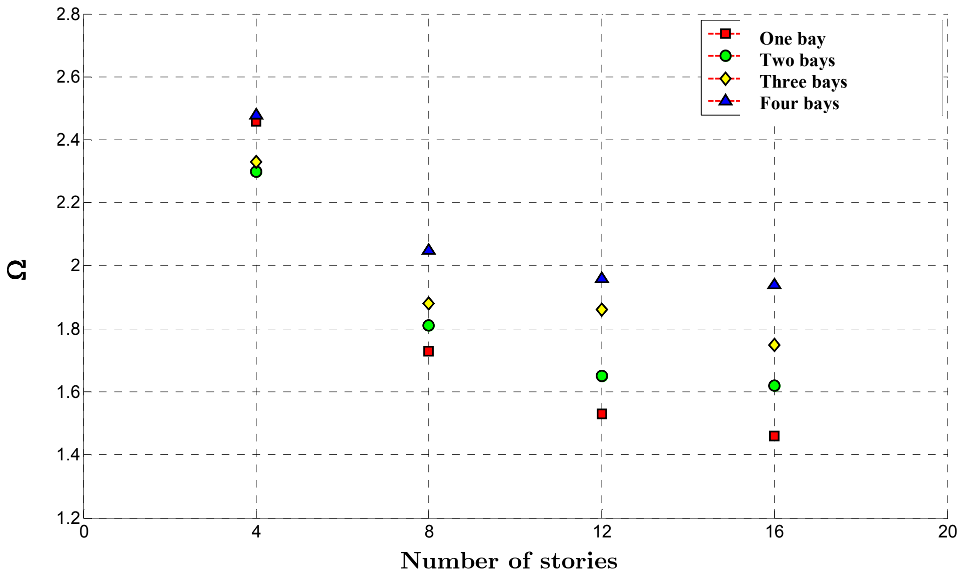


Figure 13: Overstrength (Ω) developed for the models under study.

6 ASSESSMENT OF REDUNDANCY FACTORS

From the results obtained from pushover analyses, it is confirmed that the impact of having more redundant frames increases both the ductility and strength capacity of RC-SMRFs, as currently recognized by MOC-2008 code (MOC-2008 2009, Tena-Colunga *et al.* 2009) with the redundancy factor ρ (Figure 4b). However, it is also clear from the results presented in previous sections that redundancy impacts in different proportions ductility (Figure 12) and strength (Figure 13) capacities for RC-SMRFs, which it is not yet considered in MOC-2008 code. Therefore, two different redundancy factors were assessed taking into account the current definition of MOC-2008: 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 $\Omega_{\#bay\text{-}N}$ 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 $\Omega_{2bay\text{-}N}$, obtained for the two-bay frame for the same N story model, this is:

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

In the same fashion, to assess ρ_μ according to the current definition of MOC-2008, developed ductilities $\mu_{\#bay\text{-}N}$ and $\mu_{2bay\text{-}N}$ (defined similarly) were used:

$$\rho_\mu = \frac{\mu_{\#bay-N}}{\mu_{2bay-N}} \tag{3}$$

It is clear from Eqs. 2 and 3 that for 2-bay models, $\rho_\Omega = \rho_\mu = \rho = 1.0$, as currently defined in MOC-2008 code.

The results obtained for ρ_Ω are shown in Figure 14. It is observed that ρ_Ω increases for three-bay and four-bay models, whereas for one-bay models, ρ_Ω decreases as the number of stories increases. As it was expected, $\rho_\Omega > 1.0$ for three-bay and four-bay models, and $\rho_\Omega < 1.0$ for one-bay models, except the four story model M1-4LC, for the reasons discussed in previous sections. Comparing the assessed values for ρ_Ω with respect to the proposed ρ values in MOC-2008, it is observed that three-bay and four-bay models do not reach the proposed value $\rho = 1.25$. The highest value was $\rho_\Omega = 1.16$ for the four-bay 16-story model M4-16LC. It is proposed in MOC-2008 that ρ=0.8 for one-bay models; however, the smallest computed value was $\rho_\Omega = 0.90$ for the 16-story model M1-16LC. Therefore, it can be concluded that from the strength viewpoint, in RC-SMRFs, redundancy has a smaller impact than the one anticipated in MOC-2008 code. Nevertheless, it seems that this code proposal is conceptually moving into the right direction.

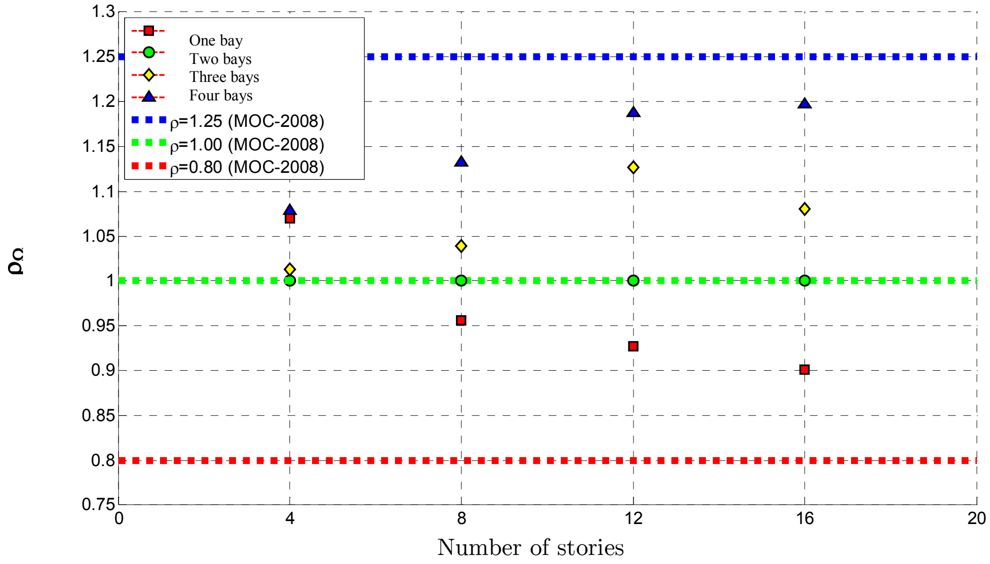


Figure 14: Redundancy factor related to overstrength, ρ_Ω

The results obtained for ρ_μ are shown in Figure 15. Similar general tendencies are observed for ρ_μ (Figure 15) and ρ_Ω (Figure 14). Therefore, similar observations can be done for ρ_μ regarding one-bay, three-bay and four-bay models with respect to the number of stories and with the proposed ρ values in MOC-2008, but ρ_μ values are higher than ρ_Ω. It is worth noting that assessed values for ρ_μ are higher than the proposed $\rho = 1.25$ value in MOC-2008 for the four-bay models, and very close to ρ=1.25 for the three-story models. Taking an average for the three-bay and four-bay models, ρ_μ=1.41 was obtained. It can also be observed from Figure 15 that for one-bay models, ρ_μ is much smaller than ρ=0.8 proposed in MOC-2008. The smallest computed value was ρ_μ =0.56 for the 16-story model M1-16LC. Therefore, it can be concluded that from the ductility viewpoint, in RC-SMRFs, redundancy has a higher impact than the one anticipated in MOC-2008 code.

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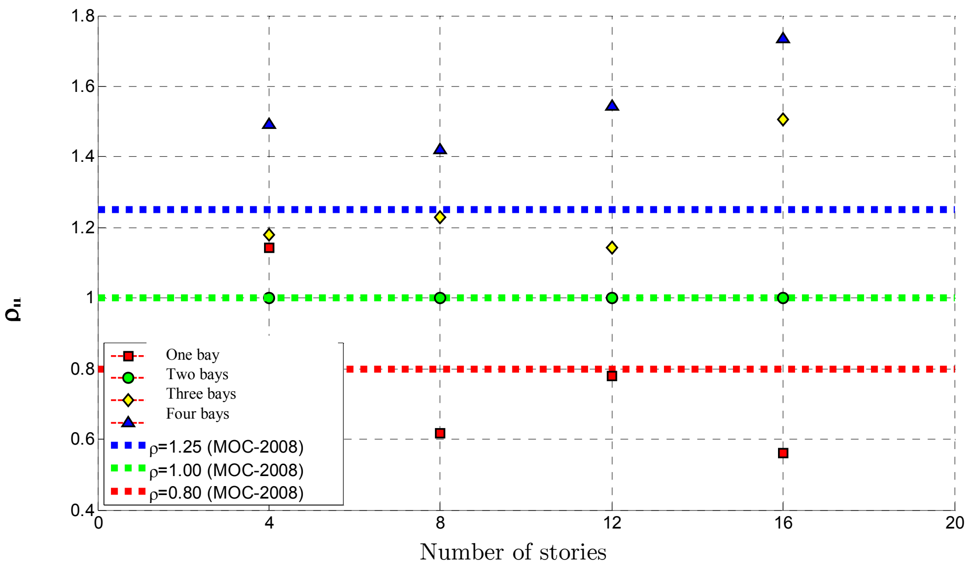

Figure 15: Redundancy factor related to ductility, ρ_μ

7 CONCLUDING REMARKS

The results of a parametric study devoted to assess the impact of increasing the structural redundancy in ductile reinforced concrete moment framed buildings were presented. Among the studied variables were the number of stories (4, 8, 12 and 16) and the number of bays (1, 2, 3 and 4). Buildings were analyzed and rigorously designed as RC-SMRFs according to the guidelines of Mexican codes. Nonlinear static analyses were used to assess redundancy factors.

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

- For a design base shear ratio V/W=0.10, 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 lowrise frames (four stories). The reason is that the impact of gravitational loads was more important in the design of beams for lowrise RC-SMRFs. On the other hand, lateral load combinations often rule the design of most members for medium-rise RC-SMRFs.
- It was confirmed that strength and deformation capacities of RC-SMRFs are impacted by redundancy. Therefore, it should be directly taken into account for a transparent seismic design. This is currently recognized in MOC-2008 code with the redundancy factor ρ.
- In general, for RC-SMRFs, the impact of redundancy is higher for their ductility capacity rather than for their strength capacity. 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, in MOC-2008 the impact of redundancy in strength is overestimated and the impact of redundancy in ductility is underestimated.
- For RC-SMRFs, the proposed redundancy factors for overstrength (ρ_Ω) and ductility (ρ_μ) decrease as the number of stories increases. From the limited results obtained in this study and based upon four-bay models, it seems that these factors tend to reach an upper limit as the number of stories increases.

Based on the results of this research and previous studies reported in the literature, it can be 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.

On this regard, 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. In this study it was found that the impact of redundancy on the ductility capacity of RC-SMRFs is higher than on their strength capacity. Therefore, perhaps it would be convenient to use different redundancy factors to modify strength and ductility. However, among others, the following questions should be first addressed: a) is the described tendency general for all structural systems? One may assume that for braced frames or shear-wall systems, perhaps the strength capacity might increase in a larger proportion with redundancy than for RC-SMRFs, b) is the use of different redundancy factors for strength and ductility practical enough? One may assume that code committee members and practicing engineers would still prefer a fixed, weighted value instead, c) should structural designs ruled by gravitational loads be addressed as an exception of the rule at the time of prescribing code recommendations for a redundancy factor? and, d) would the results obtained from nonlinear dynamic analyses substantially modify the tendencies and assessed values obtained from nonlinear static analyses? Some of these topics are starting to be addressed by this research group.

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