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ASSESSMENT OF REDUNDANCY IN THE SEISMIC DESIGN OF MOMENT RESISTING REINFORCED CONCRETE FRAMES

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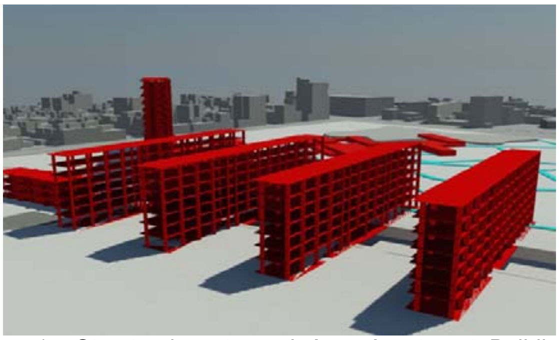
ABSTRACT: The results of a parametric study devoted to assess the impact of increasing the structural redundancy in ductile reinforced concrete (RC) moment framed buildings are presented. Among the studied variables were the number of stories and the number of bays. Studied models were 4, 8, 12 and 16-story 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.

1. Introduction

Currently, 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 practice. 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 at Caracas, Venezuela, during the July 29, 1967 Caracas Earthquake (Tena 2010), or the collapse of Juárez Apartment Building Complex in Mexico City (Fig. 1), during the September 19, 1985 Michoacán Earthquake. Juárez apartment buildings were also slender in plan and elevation (Fig. 1a), and they were weakly-redundant in the slender direction (one-bay frames only); they finally collapsed in that direction (Fig.1b).

It has been learned from experiences of past earthquakes and 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 (i.e., Bertero and Bertero 1999, Husain and Tsopelas 2004, Tsopelas and Husain 2004, Tena-Colunga 2004) 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, 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.



a) Structural system, Juárez Apartment Building Complex

b) Collapse of building C-4

Fig. 1 - 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.

The results of a parametric study devoted to assess the impact of increasing the structural redundancy in ductile reinforced concrete (RC) moment framed buildings 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. Building Codes

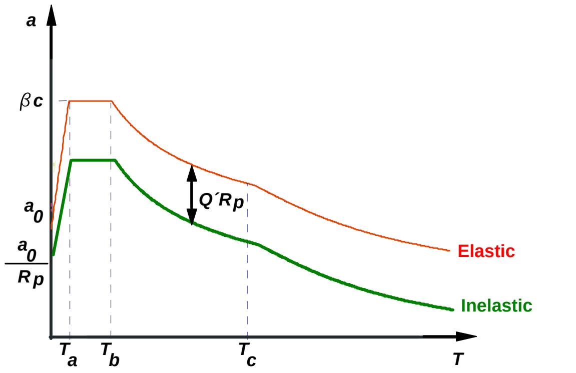
To the authors´ knowledge, the first building code to directly include a redundancy factor (ρ) for the seismic design of buildings was the 1997 UBC Code (UBC-97 1997). The original proposal of UBC-97 has changed in the most recent recommendation of US Codes (ASCE-7 2010). For space constraints, a summary of the recommendations of US building codes is not offer, but it is available in Spanish elsewhere (Tena-Colunga and Cortés-Benítez 2014). In the following section it is summarized the recommendation of Mexican codes, as they are the subject of evaluation of this study.

2.1. MOC-2008

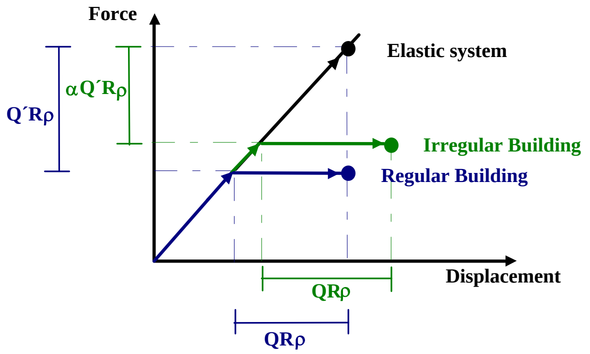
In MOC-2008 (MOC-2008 2009, Tena-Colunga *et al.* 2009) the redundancy factor (ρ) is taken into consideration at the time of defining spectral design forces (Fig. 2a). In fact, ρ is a factor that basically corrects the previous assessment of the overstrength factor (R in Mexican codes) and the ductility factor (Q in Mexican codes), as depicted in Fig. 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, Fig. 1).

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

a) ρ = 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) ρ = 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) ρ = 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

Fig. 2 - 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., Fig. 1). 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. 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 Fig. 3. For the building plan depicted in Fig. 3a, $\rho = 0.8$ 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 Fig. 3b, $\rho = 1.0$ 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 Fig. 3b is more redundant than the building plan depicted in Fig. 3a. 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 3a 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 Fig. 2b. 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: $\alpha Q'R\rho = (0.7)Q'R(1.25) = 0.875Q'R \geq 1.0$. Nevertheless, the code committee for MOC is considering establishing in the next version that, for buildings with strong irregularity condition, $\rho \leq 1.0$.

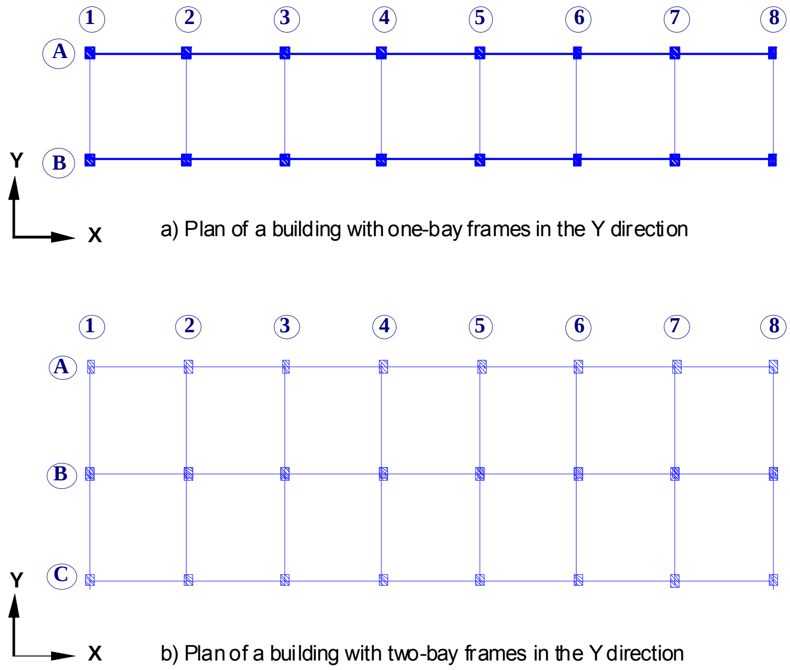


Fig. 3 - Sample buildings to illustrate the assessment of the ρ factor of MOC-2008

3. 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 Fig. 4, 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 land space 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). 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).

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 Fig. 5. It is worth noting that steel reinforcements vary for interior and exterior beams and columns, particularly for taller buildings, as reported in Table 1.

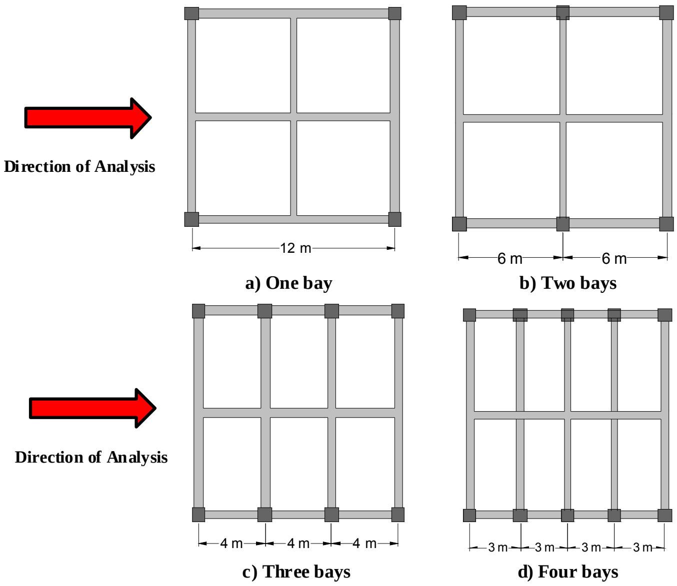


Fig. 4 - Plan layout for the subject buildings of interest. Squares indicates the location of columns (dimensions in meters)

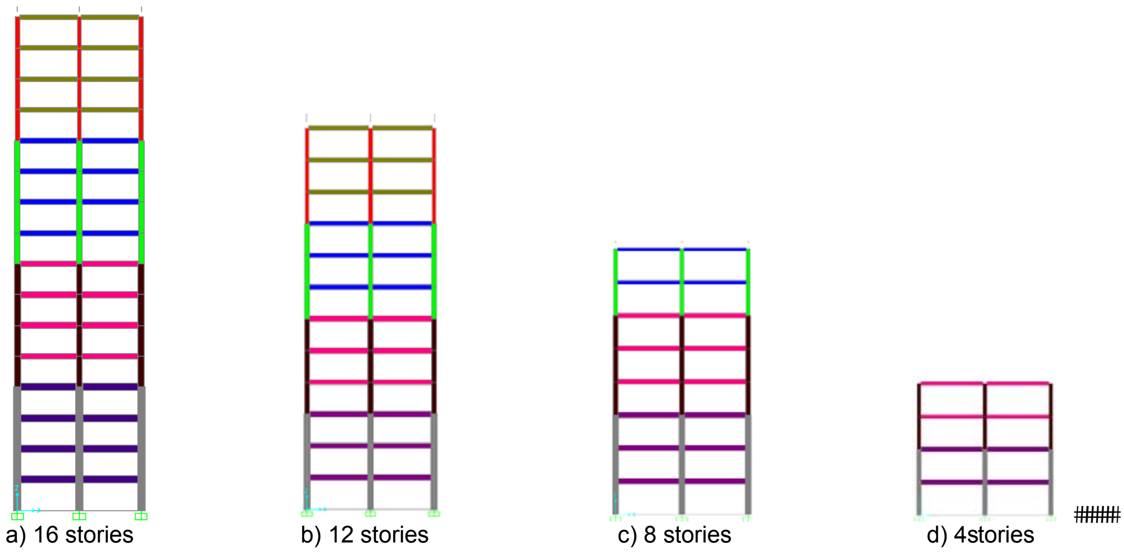


Fig. 5 - Schematic representation of changes of cross sections for beams and columns for the studied models.

Table 1 - Summary for the design of 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

The assumed 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/cm2) 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. 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 Δ=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 (Fig. 4). It can also be observed from Table 1 that in order to insure a ductile behavior for beams and columns, 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. 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.

4. 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 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, (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 is successfully implemented in the construction site.

4.1. 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 Fig. 6. 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 ϕ/ϕ_u >1.0 (in theory, the element completely failed).

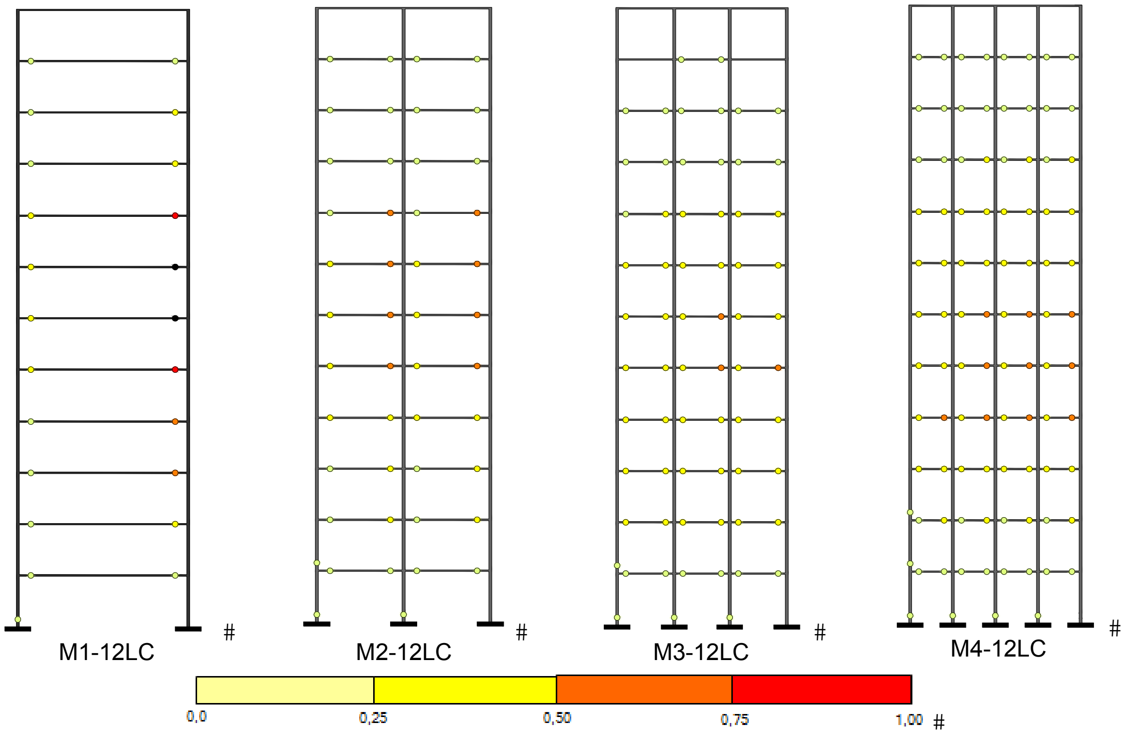


Fig. 6- Inelastic yielding mappings for the 12 story models

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4.2. 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. For space constraints, the results obtained for all models under study are not shown (i.e., Tena-Colunga and Cortés-Benítez 2014). As expected, it was observed from those curves that the elastic stiffness for the studied models increases as the number of bays increases. Therefore, from this perspective, it was difficult to qualitatively assess the impact of having more bays (more redundancy) 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 ($\Delta_{FIRST\text{-}YIELD}$), 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 Fig. 7. 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. 7. For the 4-story models (Fig. 7a), 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 influenced 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.

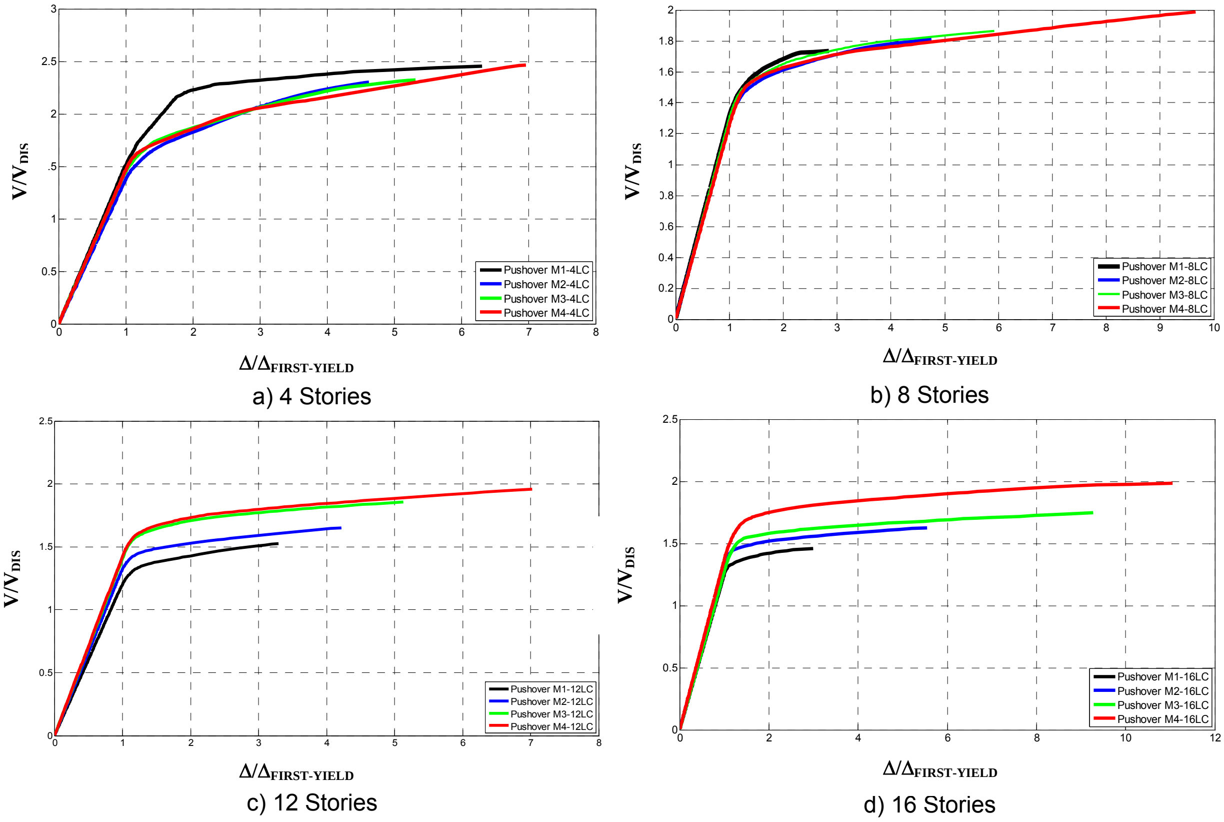

Fig. 7 - Normalized base shear vs global drift curves for the models under study

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It can also be inferred from the observation of Fig. 7 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 (Fig. 7b) 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 (Fig. 5). 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 low-rise RC-SMRFs. Also, for RC-SMRFs, possessing a higher redundancy is more important in its ductility capacity than in its strength capacity.

5. Assessment of Redundancy Factors

From the results obtained from pushover analyses, it was 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 (Fig. 2b). However, it is also clear from the results presented in previous sections that redundancy impacts in different proportions ductility and strength (Fig. 7) 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 Ω_{2bay-N}, obtained for the two-bay frame for the same N story model, this is:

$$\rho_{\Omega} = \frac{\Omega_{\#bay-N}}{\Omega_{2bay-N}} \tag{1}$$

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

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

It is clear from Eqs. 1 and 2 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 Fig. 8. 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, ρ_{Ω} >1.0 for three-bay and four-bay models, and ρ_{Ω} <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 ρ =1.25. The highest value was ρ_{Ω} =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 ρ_{Ω} =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.

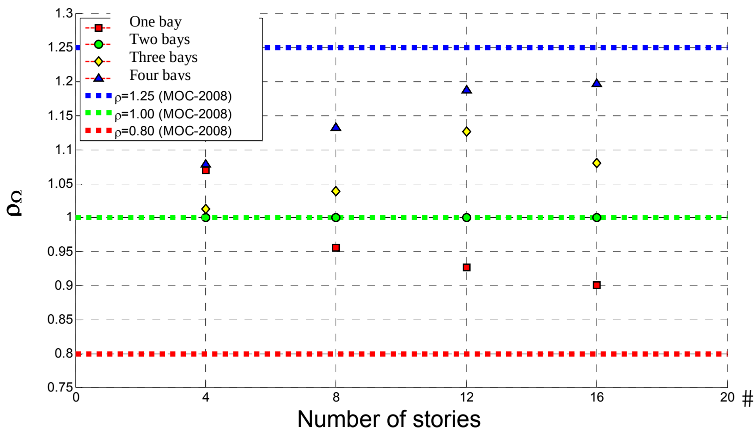

Fig. 8 - Redundancy factor related to overstrength, ρ_{Ω}

The results obtained for ρ_{μ} are shown in Fig. 9. Similar general tendencies are observed for ρ_{μ} (Fig. 9) and ρ_{Ω} (Fig. 8). 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 ρ =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 Fig. 9 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.

6. Concluding Remarks

Based upon the limitations of the described research, the following can be concluded from the results obtained in this study (for space constraints). 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. 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.

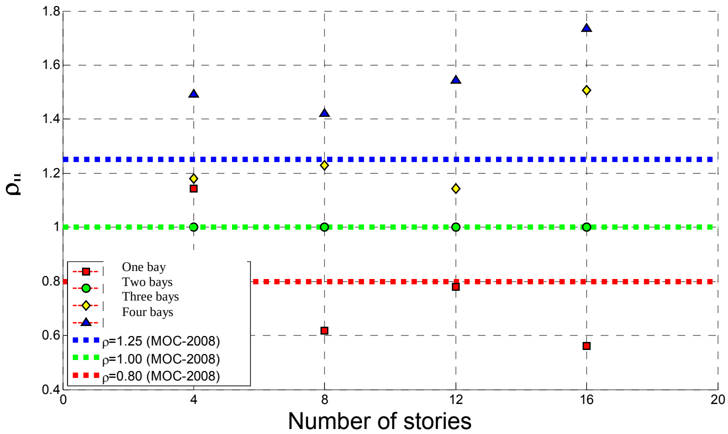

Fig. 9 - Redundancy factor related to ductility, ρ_μ

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