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# EVALUATION OF THE SEISMIC RESPONSE OF SLENDER, SETBACK RC MOMENT-RESISTING FRAME BUILDINGS DESIGNED ACCORDING TO THE SEISMIC GUIDELINES OF A MODERN BUILDING CODE

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## SUMMARY

This paper presents the study of two irregular (setback and slender) reinforced concrete (RC), moment-resisting framed buildings that were designed to fulfill the seismic provisions of the Mexican code. The fourteen-story irregular buildings, that do not satisfy four regularity conditions defined by the code, were designed according to the specified provisions for irregular buildings and for the lakebed zone of Mexico City. The limiting story drift angle $\Delta$=1.2% established by the ruling code was used for the design of the buildings in the slender direction. The impact of having single-bay frames instead of multiple-bay frames in the slender direction in the seismic performance of the buildings is also studied. Several recorded and simulated accelerograms associated to the design spectra for the lakebed region of Mexico City were used for nonlinear dynamic analyses. Story drift ratios associated to the original design were compared with peak dynamic story drift angles computed from nonlinear dynamic analyses. Structural yielding was studied and associated to hysteretic, deformation and strength demands. The results obtained from nonlinear dynamic analyses suggest that the slender direction of setback buildings with one-bay frames is vulnerable if the buildings are designed close to the limiting drift angle $\Delta$=1.2% established by Mexican codes because the yielding mapping favors non-ductile failure mechanisms that can be triggered by P-$\Delta$ effects.

## INTRODUCTION

The seismic provisions of Mexico's Federal District Code RCDF-93 [1,2], which was the ruling code in Mexico City until January 29, 2004, where the new RCDF-2004 code was finally approved by the Governor of Mexico's Federal District after 3.5 years of evaluation by the city Council, define eleven conditions of regularity that building structures must satisfy to use directly the reductive seismic force factor $Q'$ as $Q$ ($Q'=Q$) in the design of buildings [1,2]. If one or more of these eleven regularity conditions are not fulfilled, then, response modification factors $Q$ have to be affected by a 0.8 reduction factor to compute $Q'$ factors ($Q'=0.8Q$). Therefore, irregular buildings must be designed for higher forces but still be checked to comply with the lateral story drift criteria specified for regular buildings (this is, lateral deformations obtained from the analyses must be multiplied by $Q$ in both cases). The 0.8 reduction factor

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was based on intuition and experience, as there were no studies to justify this value. It is felt that irregularities can affect the nonlinear dynamic behavior of structures in several ways, so, in some cases, the 0.8 reduction factor might be safe enough but in other instances it might not. This could be the case of buildings with several irregularities.

The eleven regularity conditions were formerly introduced in 1987 Mexico's Federal District Code, and are described in English language in Tena-Colunga [2]. These regularity conditions are based on what was learned worldwide after major earthquake events and some of them are mostly based on good engineering judgment, experience and common sense, rather than in detailed analytical or experimental research.

There are some structural irregularities that were studied by Mexican researchers in some detail, particularly the soft story condition [3]. However, these studies were not addressed to evaluate the overall design process, the response modification factors and drift limits allowed by the RCDF code of the time. Therefore, a research project was started in Mexico to evaluate the adequacy of RCDF-93 response modification factors ($Q$) and other design criteria for irregular buildings, both for reinforced concrete (RC) and structural steel [4-6]. Buildings were designed according to the provisions of RCDF-93 for the lakebed and hill zones of Mexico City and for a range of story drift angles in order to evaluate the limits established by the code.

The regularity conditions remained unchanged until January 2004, as the new seismic provisions for RCDF-2004 include some changes for the definition and design of irregular structures. Mostly, the original eleven regularity conditions remain the same [2], but the statement devoted to prevent a soft story condition was redefined and now is more conservative than in previous versions, taking into account, among other material, recent research findings summarized in Tena-Colunga [5,6] and Tena-Colunga *et al.* [4]. Among the changes in the design are that if one building does not satisfies one regularity condition, then $Q'$=0.9$Q$ must be used for the design. If two or more regularity conditions are not satisfied, $Q'$=0.8$Q$. If a building (a) has a strong torsional irregularity evaluated in terms of a static eccentricity greater than 20 percent of the plan dimension in the given direction of analysis ($e_s$>0.20L) or, (b) has a well-defined soft story condition, this is, violates former regularity condition 10 of RCDF-93 [2]; then, the building must be classified as strongly irregular and use $Q'$=0.7$Q$.

This paper summarizes the studies conducted in two setback and slender reinforced concrete (RC), moment-resisting framed buildings that were designed to fulfill the seismic provisions of RCDF-93. The fourteen-story irregular buildings, that do not satisfy three regularity conditions defined by the code, were designed according to the specified provisions for irregular buildings and for the lakebed zone of Mexico City. The limiting story drift angle $\Delta$=1.2% established by the ruling code was used for the design of the buildings in the slender direction. The impact of having single-bay frames instead of multiple-bay frames in the slender direction in the seismic performance of the buildings is also studied. Some of the findings of this research study were taken into account for the seismic design provisions for irregular structures of RCDF-2004. The design criteria for the buildings of reference and some of the most important results obtained from the nonlinear dynamic analyses will be briefly summarized in following sections

## SUBJECT SETBACK BUILDINGS

The particular study of two RC setback buildings, which do not comply with more than one regularity conditions according to RCDF-93, is presented. The general description of the buildings and design considerations are summarized in following sections. For both buildings the structural system for earthquake loading is composed of special moment resisting frames (SMRFs), so a reduction force factor $Q'$=0.8$Q$=3.2 was used for the design. The material properties considered for design was concrete with

$f'_c$=250 kg/cm$^2$ (25.48 MPa) and reinforcement steel with $f_y$=4200 kg/cm$^2$ (428.1 MPa). Young's 8modulus was taken as $E = 14000\sqrt{f'_c}$ (kg/cm$^2$), considering a concrete type I according to RCDF-93. Buildings were designed according to seismic provisions of RCDF-93 [1-2] for zone III (lakebed soft soil of Mexico City), complying with all the requirements established by the reinforced concrete guidelines of RCDF-93 for SMRFs. For stiffness purposes, interior beams were considered as T sections and exterior beams as L sections, considering the contribution of the slab for stiffness purposes. The equivalent flange width was taken according to the recommendations available in the RC guidelines of RCDF-93. Also, the joint was considered as being only 50% effective as a rigid end zone. The details of the design can be consulted in Tena-Colunga [5].

**IR2A Building**

IR2A building is a fourteen-story setback building (with vertical and horizontal irregularities), as depicted in Fig. 1. The building does not comply with four regularity conditions of RCDF-93 (conditions 2, 3, 7 and 8). The building is 50.5 m tall with a typical story height of 3.5 m but the ground level, where the story height is 5 m. The floor system consists of a 12 cm thick RC flat slab supported by beams in its perimeter.

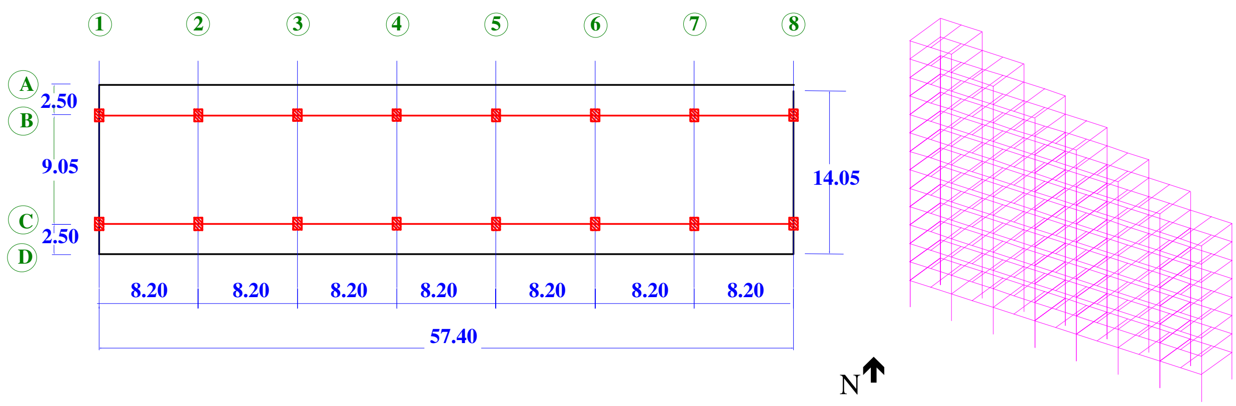



Figure 1. Plan and 3D ETABS model for IR2A building (dimensions in meters).

The final design is composed of beams of rectangular cross sections measuring 50x80 cm ($\rho^+$ = 0.0035 to 0.0062, $\rho^-$ =0.0054 to 0.0135) and 50x90 cm ($\rho^+$ = 0.006 to 0.0072, $\rho^-$ = 0.0122 to 0.0134) in the E-W direction and 45x105 cm ($\rho^+$ = 0.0027 to 0.0113, $\rho^-$ = 0.0038 to 0.0130) in the N-S direction. The shear reinforcement at beam-ends varies from $\rho_{sh}$=0.0043 to $\rho_{sh}$=0.0095 in the E-W direction and $\rho_{sh}$=0.0046 to $\rho_{sh}$=0.0097 in the N-S direction. The sizing of the beams was controlled by flexure, whereas the shear reinforcement was generally controlled by the special requirements for confinement of beams of SMRFs.

Columns are of rectangular cross section, having their longer dimension parallel to the N-S direction, and measuring 70x110 cm in stories 7 to 14 with a longitudinal steel reinforcement ratio $\rho$=0.0105 and a transverse shear reinforcement ratio of $\rho_{sh}$=0.0165 (columns C1), and 70x130 cm in stories 1 to 6 with longitudinal steel reinforcement ratios $\rho$=0.0100 (column C2) and $\rho$=0.0212 (column C3) and transverse shear reinforcement ratios $\rho_{sh}$=0.0153 (column C2) and $\rho_{sh}$=0.0178 (column C3). The sizing of the columns was controlled by shear and drift requirements, whereas the shear reinforcement was generally obtained by applying the stringiest confinement provisions for SMRFs detailing.

3D Etabs models were used for design purposes using modal-spectral dynamic analyses according to the seismic provisions of RCDF-93 code. The dynamic elastic characteristics of IR2A (first six mode shapes) are summarized in Table 1. It can be observed that the structure is more flexible in the N-S direction than in the E-W direction. The first global mode shape is basically a translational mode in the N-S direction coupled with rotation because of the irregularities, whereas the second global mode shape is a translational mode in the E-W direction only, as the irregularities do not introduce eccentricities in the E-W direction. The third global mode is basically a rotational mode highly coupled with translations in the N-S direction.

**Table 1. Dynamic characteristics of IR2A building**

| Mode | Direction | Period (s) | Modal Mass (%) | | |
|---|---|---|---|---|---|
| | | | N-S | E-W | Rotation |
| 1. First mode of translation (coupled) | N-S | 1.381 | 54.78 | 0.00 | 9.74 |
| 2. First mode of translation | E-W | 1.282 | 0.00 | 82.20 | 0.00 |
| 3. First torsional mode (coupled) | Rotation | 0.889 | 24.84 | 0.00 | 45.65 |
| 4. 2nd mode of translation | E-W | 0.487 | 0.00 | 10.39 | 0.00 |
| 5. 2nd torsional mode (coupled) | Torsion | 0.436 | 6.68 | 0.00 | 30.96 |
| 6. 2nd mode of translation (coupled) | N-S | 0.316 | 6.12 | 0.00 | 1.54 |

The maximum story drift angles associated to this design were $\Delta$=1.15% in the N-S direction at story 7 and $\Delta$=0.8% in the E-W direction, within the design drift angles established by RCDF-93.

**IR2C Building**

IR2C is a setback building, twin structure of IR2A building, which has an additional bay in the short (slender) direction, as depicted in Fig. 2. The building does not comply with four regularity conditions of RCDF-93 (conditions 2, 3, 7 and 8). The building is 50.5 m tall with a typical story height of 3.5 m but the ground level, where the story height is 5 m. The floor system consists of a 12 cm thick RC flat slab supported by beams in its perimeter.

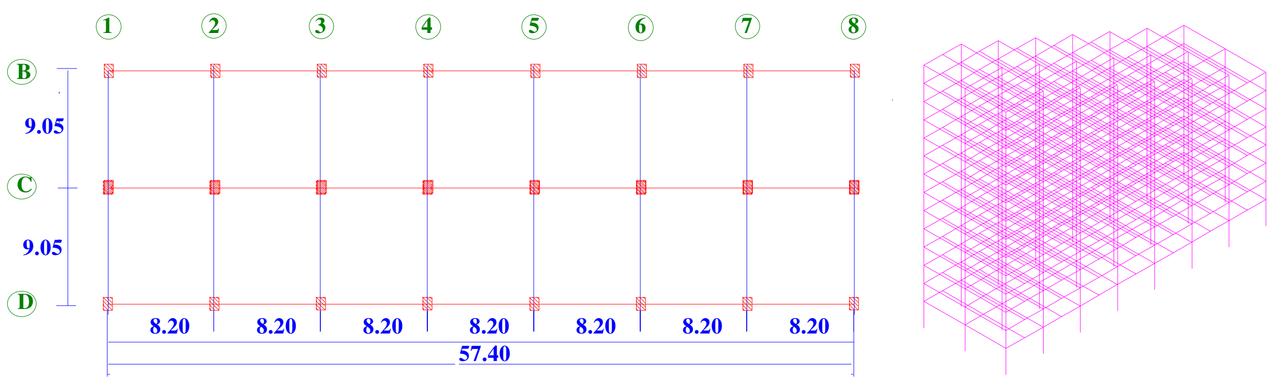



Figure 2. Plan and 3D ETABS model for IR2C building (dimensions in meters).

The final design is composed of beams of rectangular cross sections measuring 50x90 cm ($\rho^+$ = 0.0027 to 0.0073, $\rho^-$ = 0.0036 to 0.0126) and 50x100 cm ($\rho^+$ = 0.007 to 0.0099, $\rho^-$ = 0.0087 to 0.0134) in the E-W direction and 50x100 cm ($\rho^+$ = 0.0033 a 0.0120, $\rho^-$ = 0.0043 a 0.0134) in the N-S direction. The shear reinforcement at beam-ends varies from $\rho_{sh}$=0.0043 to $\rho_{sh}$=0.0095 in the E-W direction and $\rho_{sh}$=0.0046 to $\rho_{sh}$=0.0097 in the N-S direction. The sizing of the beams was controlled by flexure, whereas the shear reinforcement was generally controlled by the special requirements for confinement of beams of SMRFs.

Columns are of rectangular cross section, having their longer dimension parallel to the N-S direction, and measuring 70x110 cm in stories 5 to 14 with a longitudinal steel reinforcement ratio $\rho$=0.0105 (column C1) and $\rho$=0.0171 (column C2) and a transverse shear reinforcement ratio of $\rho_{sh}$=0.0148 (column C1) and $\rho_{sh}$=0.0203 (column C2), and 75x135 cm in stories 1 to 6 with longitudinal steel reinforcement ratios $\rho$=0.0130 (column C3), $\rho$=0.0203 (column C4) and $\rho$=0.0250 (column C5) with transverse shear reinforcement ratios $\rho_{sh}$=0.0180 (columns C3 and C4) and $\rho_{sh}$=0.0229 (column C5). The sizing of the columns was controlled by shear and drift requirements, whereas the shear reinforcement was generally obtained by applying the stringiest confinement provisions for SMRFs detailing.

3D Etabs models were used for design purposes using modal-spectral dynamic analyses according to the seismic provisions of RCDF-93 code. The dynamic elastic characteristics of IR2C (first six mode shapes) are summarized in Table 2. Similar observations can be made with respect to IR2A building, noting that IR2C is a slightly more rigid structure than IR2A, particularly in the N-S direction.

**Table 2. Dynamic characteristics of IR2C building**

| Mode | Direction | Period (s) | Modal Mass (%) | | |
|---|---|---|---|---|---|
| | | | N-S | E-W | Rotation |
| 1. First mode of translation (coupled) | N-S | 1.299 | 54.46 | 0.00 | 13.02 |
| 2. First mode of translation | E-W | 1.265 | 0.00 | 82.07 | 0.00 |
| 3. First torsional mode (coupled) | Rotation | 0.899 | 26.42 | 0.00 | 48.34 |
| 4. 2nd mode of translation | E-W | 0.488 | 0.00 | 10.59 | 0.00 |
| 5. 2nd torsional mode (coupled) | Rotation | 0.439 | 5.85 | 0.00 | 26.12 |
| 6. 2nd transnational mode (coupled) | N-S | 0.331 | 5.47 | 0.00 | 0.89 |

The maximum story drift angles associated to this design were $\Delta$=1.02% in the N-S direction at story 3 and $\Delta$=0.8% in the E-W direction, within the design drift angles established by RCDF-93.

**Table 3. Characteristics of some of the selected acceleration records**

| | 09/19/85 Records | | | | Artificial Records | | |
|---|---|---|---|---|---|---|---|
| | SCT-EW | SCT-NS | TEX-EW | TBOM-NS | S05-EW | S56-EW | S84-EW |
| **Duration (s)** | 160 | 160 | 100 | 150 | 250 | 250 | 250 |
| **Max. $S_a$ (g)** | 1.0 | 0.66 | 0.49 | 0.75 | 0.86 | 1.44 | 1.03 |
| **$T_{site}$ (s)** | 2.01 | 2.05 | 1.62 | 1.92 | 2.15 | 2.20 | 1.38 |

## ACCELERATION RECORDS

Accelerograms recorded in Mexico City at stations SCT, TEX and TBOM during the September 19, 1985 Michoacán earthquake ($M_S$=8.1) in the lake bed zone (zone III), as well as artificial records for a postulated $M_S$=8.1 subduction earthquake obtained for the lake-bed stations S05, S56 and S84 (installed after the 1985 earthquake) were used for the nonlinear dynamic analyses. These accelerograms are associated to the design spectra of RCDF-93. The main characteristics of some of the selected records are summarized in Table 3. The reported peak pseudoacceleration ($S_a$) corresponds to the one obtained for a 5% damped elastic response spectra and occurs for the reported site period ($T_{site}$).

## NONLINEAR DYNAMIC ANALYSES

Nonlinear dynamic analyses were performed for all buildings using the acceleration records presented in Table 3 that correspond to the seismic zone that buildings were designed for. DRAIN-2DX program was used for the nonlinear dynamic analyses. Two types of modeling were used, depending on the characteristics for the structural system: (a) a representative frame (frame B, Fig. 3) for IR2A and IR2C buildings in their main longitudinal direction (E-W), as these frames have the same geometry, sections and reinforcement, as well as the mode shape is completely uncoupled in that direction and, (b) 2D models that account for the interaction among frames in the short direction (N-S), as shown in Fig. 4 for IRA building and in Fig. 5 for IR2C building. These models account for the transmition of forces among frames due to the diaphragm action. The rigid diaphragm action is modeled with link elements (rigid elastic axial rods) that transmit lateral loads from one frame to another without dissipating energy by any means (damping, hysteresis, etc.).

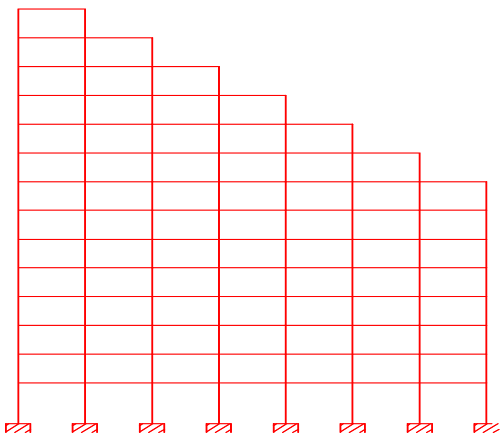


Figure 3. Equivalent Frame model (B) used for IR2A and IR2C buildings in the E-W direction.

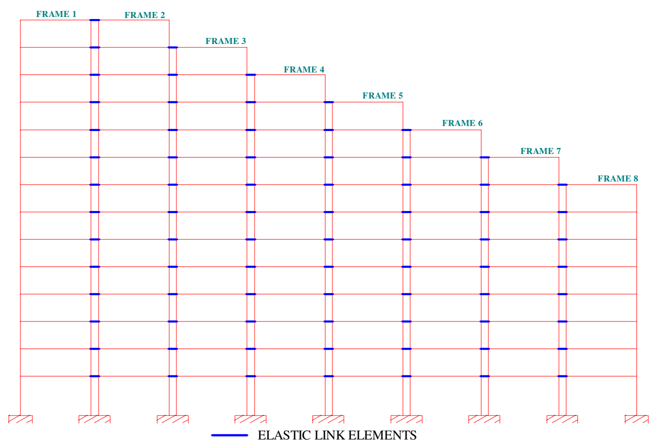



Figure 4. 2D model for IR2A building in the N-S direction that accounts for the interaction among resisting frames in that direction.

Because IR2A and IR2C buildings are made of reinforced concrete, the following assumptions were taken for computing the nominal capacities of beams and columns: (1) the concrete was modeled with the equivalent stress block established by the concrete norms of RCDF-93 (NTCC-95), (2) the “real” or actual distribution of the reinforcement steel according to the final design was considered and, (3) an elastic perfectly-plastic behavior of the reinforcement steel, as established by NTCC-95, was assumed. These assumptions are consistent with the minimum overstrength associated to current RCDF-93 provisions. Overstrength due to concrete confinement and a suitable nonlinear modeling of the stress-strain curve for the reinforcement steel was only 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 recently proposed by Rodríguez and Botero in 1994 for rebars produced in Mexico.

P-Δ effects were included in the nonlinear analyses. Soil-structure interaction was not included as the purpose of this study is to evaluate the code provisions regarding the *Q* factors, drift angles, strength and stiffness criteria for irregular structures alone; therefore, the study should be done without introducing other variables that may affect these code criteria.

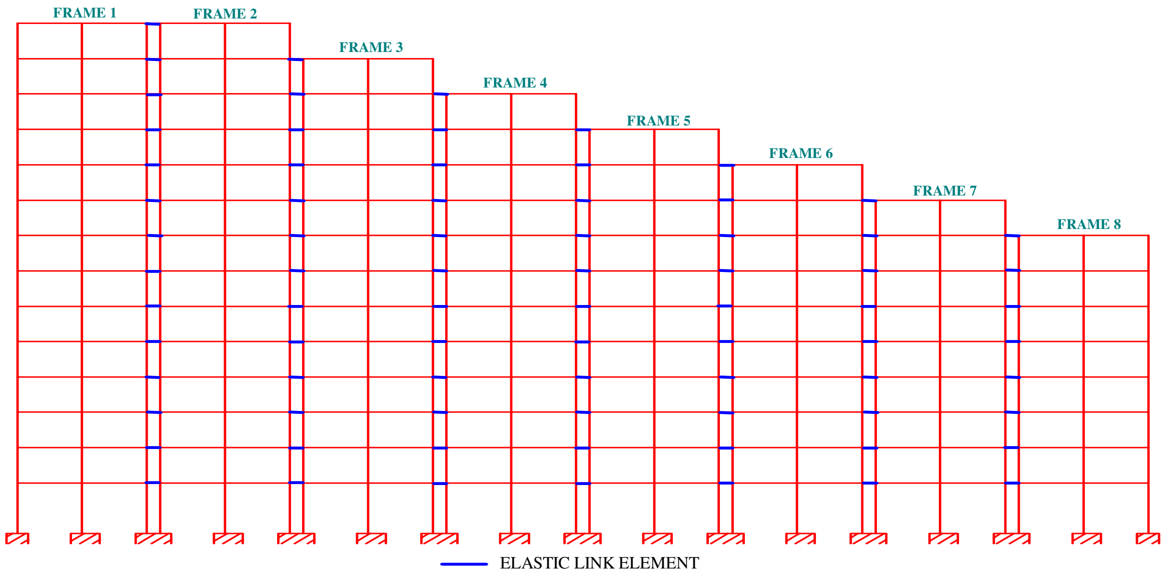



Figure 5. 2D model for IR2C building in the N-S direction that accounts for the interaction among resisting frames in that direction.

Dynamic results processed from the analyses were envelopes for peak dynamic story drift angles ($\Delta = \Delta_i / H_i$), maximum dynamic story shear indexes ($V/W_T$), peak story ductility demands ($\mu$), effective (peak to peak) story shear stiffnesses and story hysteresis curves ($V/W_T$ vs $\Delta$), as well as yielding mapping for time-steps associated to peak dynamic responses and the yielding mapping envelope for all time-steps (detect all elements that responded inelastically at least once in a while).

The critical acceleration records for both IR2A and IR2C buildings were SCT-EW record for the "real" records and S56-EW record for the artificial records. It is worth noting that the artificial S05-EW record also produced higher demands than the SCT-EW record, but this paper prefers to present peak responses for each group of accelerograms (recorded vs artificial).

Some of the most interesting results for both buildings will be shown and discussed in following sections.

**IR2A Building**

Some of the results obtained for the setback frame B in the E-W direction are presented in Figures 6 to 9. The maximum response envelopes for displacement ductility, story drift angles, story shear indexes and effective (peak to peak) story shear stiffnesses for SCT-EW and S56-EW records when nominal strength sources are considered are depicted in Fig. 6 for the E-W direction. It can be observed that peak story ductility demands occur in the first three stories and they slightly surpass the response modification factor $Q$=4 assumed for the design for the SCT-EW record, but $\mu$ considerably surpass $Q$ for the S56-EW record. Peak to peak story shear stiffnesses in these stories are about 25% to 28% of their initial elastic stiffnesses for SCT-EW records, and slightly smaller than that for S56-EW record. Peak dynamic story drift angles surpass those computed for the design in the first three stories, as well as the drift limit $\Delta$=1.2% established by RCDF-93 code.

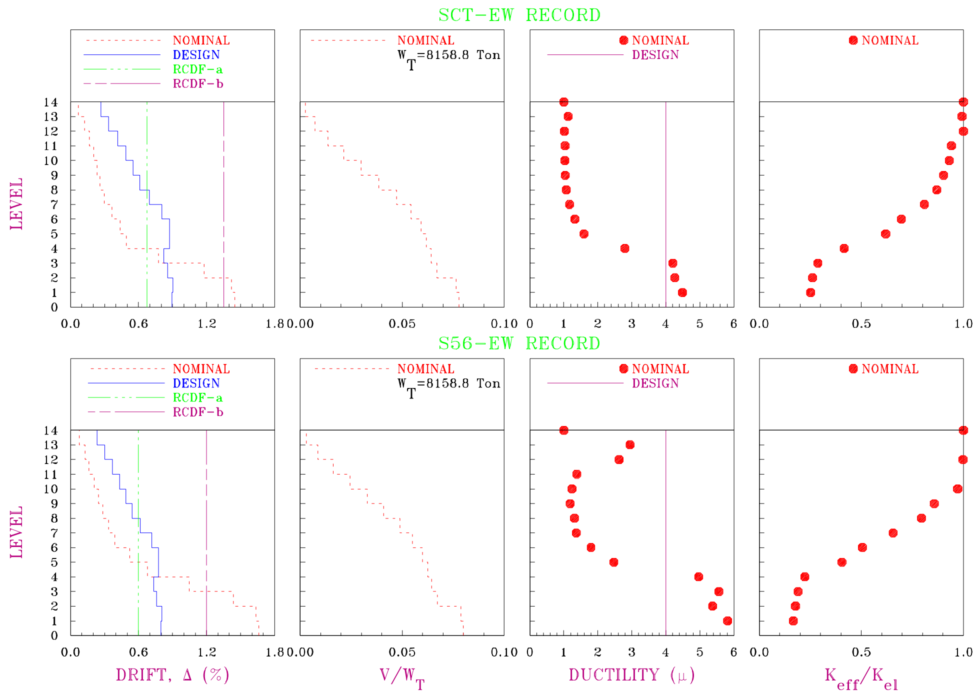



Figure 6. Peak response envelopes for frame B of IR2A building.

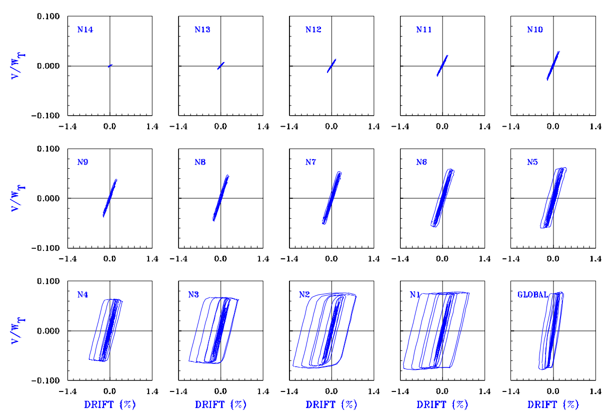


Figure 7. Hysteresis curves for frame B of IR2A building under SCT-EW record.

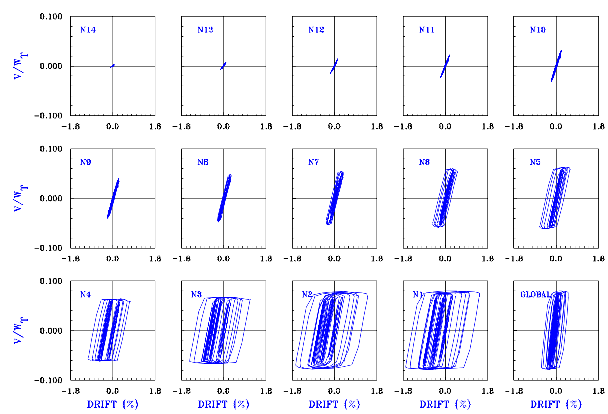
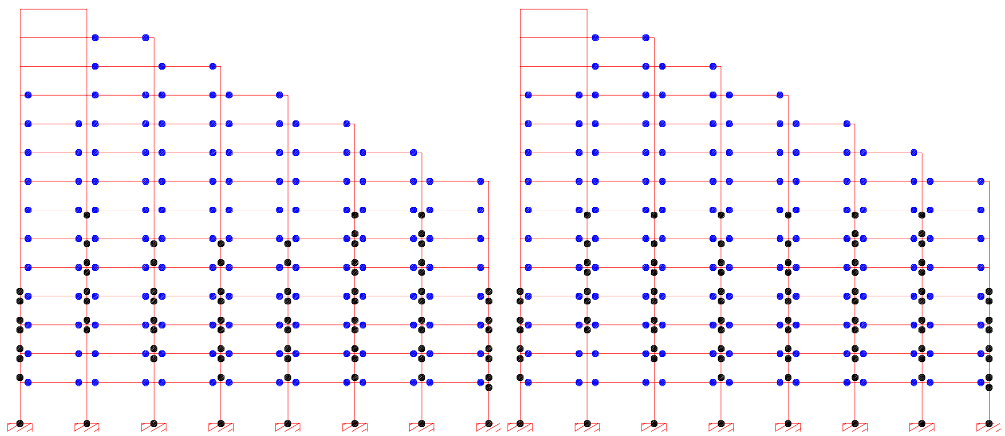


Figure 8. Hysteresis curves for frame B of IR2A building under S56-EW record.

From the hysteresis curves (Figures 7 and 8) one can observe that all stories exhibit a stable behavior with no important strength degradation, despite of the fact that an important number of column yielding occur primarily in the first four stories, although column yielding can also be observed in stories 5 to 7 (Fig. 9). The reason for this is that the magnitude of the computed plastic hinge rotation was small compared to those obtained for the beams. In general, one can conclude that the design is somewhat satisfactory in this direction, granted that overstrength sources due to the confinement of the concrete core and "real" behavior of the reinforcement steel were not considered in these analyses, something that will be evaluated in the near future for these direction.

The results obtained for the slender frames in the N-S direction with the model that accounts for the interaction among frames are shown in Figures 10 to 15. The maximum response envelopes for SCT-EW and S56-EW records when nominal strength sources are considered are depicted in Fig. 10. It can be observed from Fig. 10 that under SCT-EW record, peak story ductility demands surpass the design $Q$ factor in the first six stories, and they are associated to peak to peak shear stiffnesses or 18% to 25% of their initial elastic value. Peak dynamic story drift ratios are in general covered by the design envelope in almost all stories and they do not surpass the drift limit Δ=1.2% established by the code, although there are some stories where the peak drifts are very close to this code limit.



a) Yielding envelope for SCT-EW record b) Yielding envelope for S56-EW record

Figure 9. Yielding mapping for frame B of IR2A building.

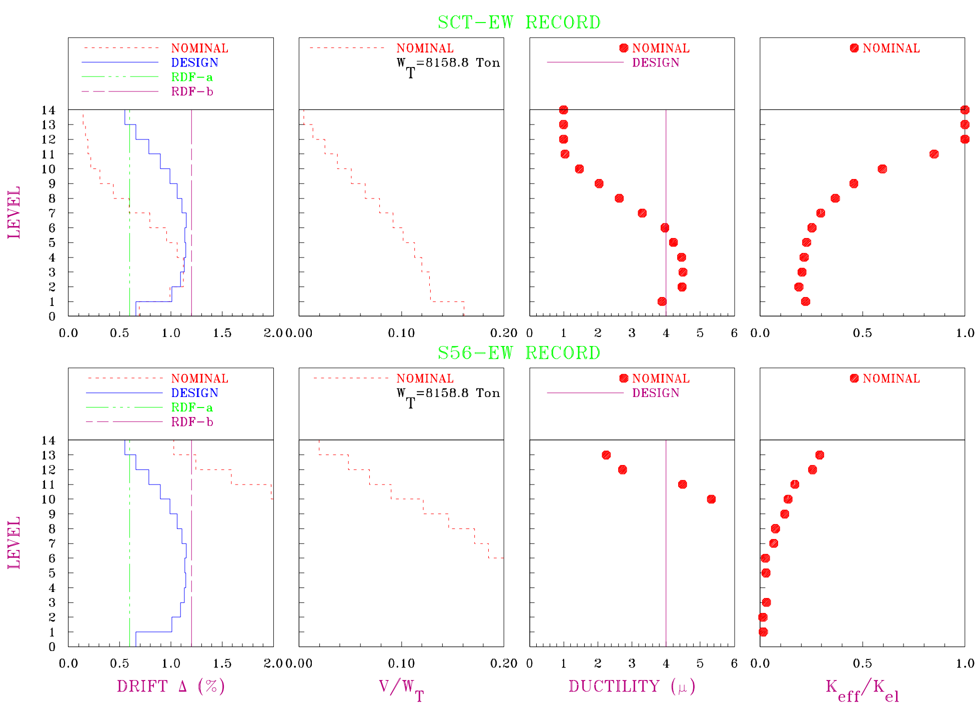



Figure 10. Peak response envelopes for all frames of IR2A building in the N-S direction.

The hysteresis curves for the SCT-EW record exhibit, however, an apparently stable behavior where some important strength degradation and some asymmetric (shifting) behavior of the hysteresis loops in the first four stories are observed (Fig. 11). The observed asymmetric behavior and strength degradation are directly related to the important column yielding observed primarily in the first five stories, although column yielding is also detected in stories six to nine (Fig. 12). In fact, the column yielding mapping in stories 2 and 3 suggest the possibility that a generalized weak story failure mechanism could trigger, would the frames in this direction be subjected to a stronger ground shaking.

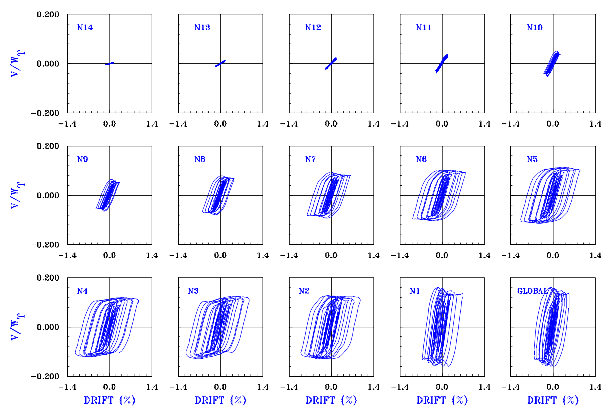



Figure 11. Hysteresis curves for all slender frames of IR2A building in the N-S direction under SCT-EW record.

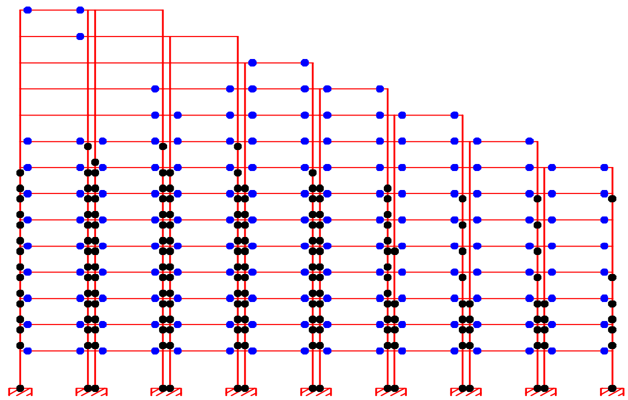


Figure 12. Yielding mapping envelope for all slender frames of IR2A building in the N-S direction under SCT-EW record.

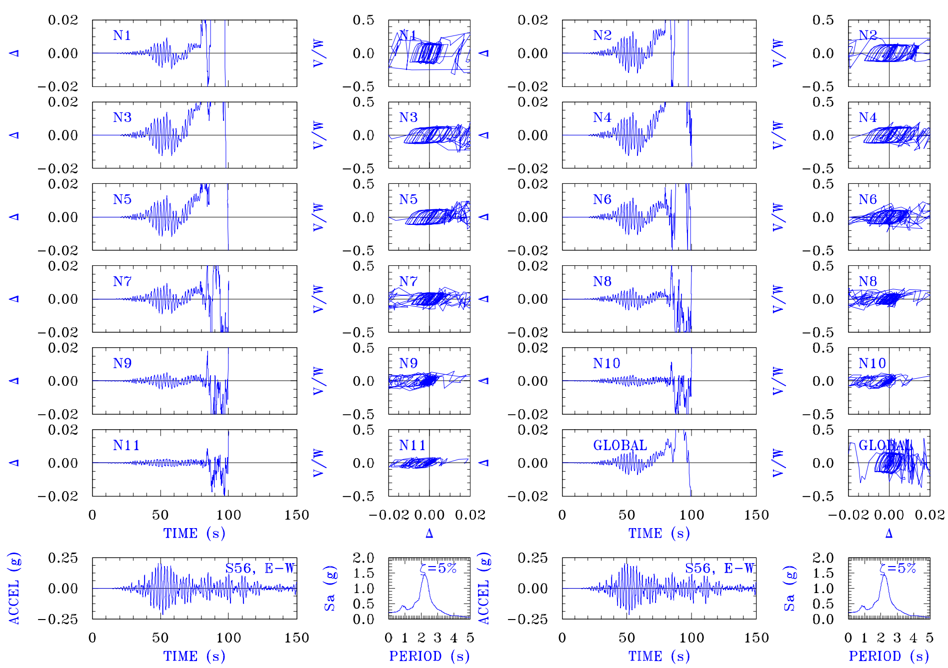



Figure 13. Details of the time history response of the first eleven stories of the slender frames of IR2A building in the N-S direction under S56-EW record, nominal strength.

In fact, one can observe from all envelopes shown in Fig. 10 for S56-EW record that something strange is happening for this record when nominal strength is considered, as peak responses for the first nine stories are not depicted inside the plot region. The reason for this is that, numerically, the building experienced a collapse mechanism, as shown in detail in Figure 13, where the time-step response for the first eleven stories plus the global response is depicted. From the observation of this time-histories and other information, one can conclude that the dynamic instability that lead the structure to collapse was most likely triggered by: (a) P-Δ effects due to the slenderness of the frames in this direction, (b) an insufficient strength and deformation capacity for the columns and, (c) the low redundancy of these frames (one-bay frames). All these factors favored a weak story failure mechanism. It seems that the dynamic instability was unbearable approximately after 80 seconds of shaking and was leaded by the columns of stories 2 and 3. The remaining stories collapsed afterwards, as can be observed in the time histories depicted in Fig. 13. The yielding envelope associated to the numerical collapse obtained under S56-EW record is depicted in Fig. 14, where it is observed a generalized column yielding for the first 10 stories, as well as a generalized beam yielding.

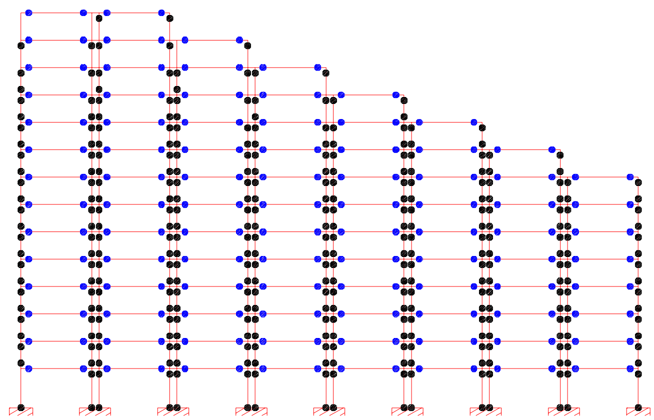


Figure 14. Yielding mapping for all slender frames of IR2A building in the N-S direction under S56-EW.

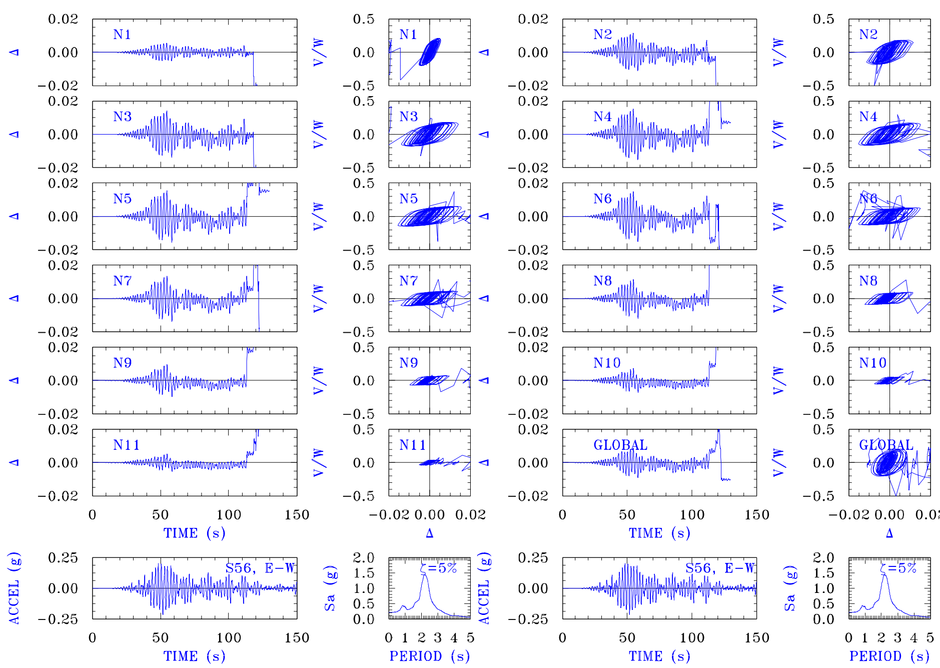



Figure 15. Details of the time history response of the first eleven stories of the slender frames of IR2A building in the N-S direction under S56-EW record, considering overstrength sources.

An important item to evaluate in future studies is the effect of the duration of the ground motion, as the structural collapse was detected when the severity of the acceleration record diminished, this is, after experiencing the stronger pulses of acceleration that caused important damage to the structure.

The structural collapse under S56-EW record was still obtained when overstrength sources were included for beams and columns due to: (a) concrete confinement model using the modified Kent-Park model and, (b) the “real” stress-strain curve for the reinforcement steel, as depicted in Fig. 15. However, it can be observed that the structure was able to resist more loading cycles before the collapse. One can conclude that the major problem for IR2A building in the N-S direction is being slender and have low column redundancy (one-bay frames), factors that caused heavy strength and deformation demands that columns were unable to withstand despite of being detailed fulfilling all the requirements for a SMRF structure. Therefore, these results suggest reviewing the design considerations for one-bay, RC frame structures, as currently there are no specific recommendations or restrictions for the design of this type of structures.

**IR2C Building**

For IR2C building only the results obtained for the slender frames in the N-S direction with the model that accounts for the interaction among frames will be shown and discussed, as this is the most vulnerable direction for the building. The maximum response envelopes for SCT-EW and S56-EW records when nominal strength sources are considered are depicted in Fig. 16.

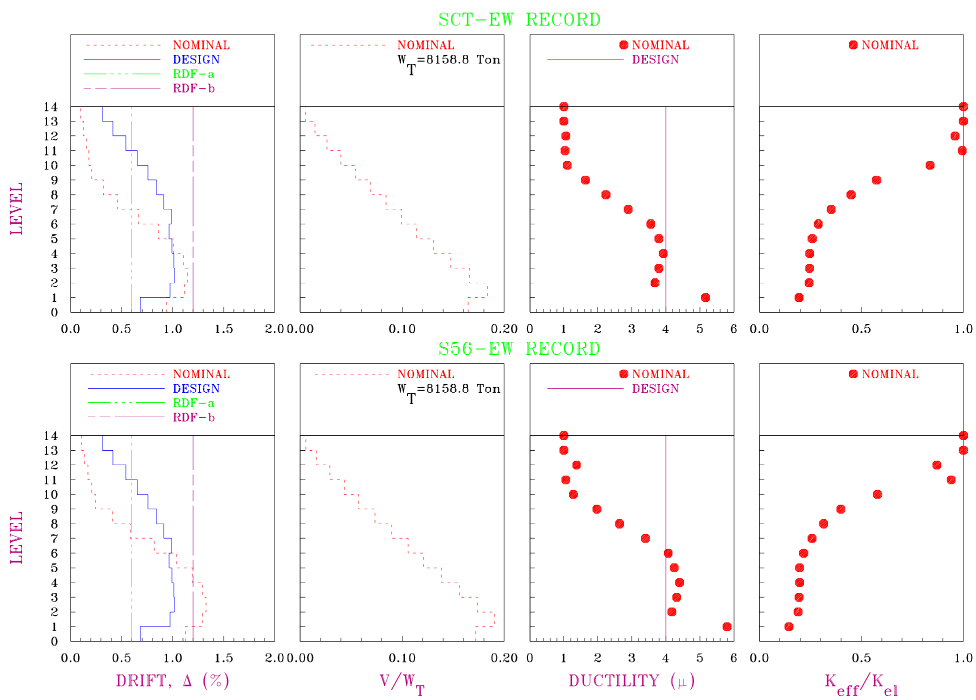



Figure 16. Peak response envelopes for all frames of IR2C building in the N-S direction.

It can be observed from Fig. 16 that under both SCT-EW and S56-EW records, peak story ductility demands are close to the design $Q$ factor in the first six stories, and they are associated to peak to peak shear stiffnesses or 20% to 30% of their initial elastic value. Peak dynamic story drift ratios are in general covered by the design envelope in the upper stories, but they surpass the design envelope in the first six stories. Peak dynamic drift ratios in the bottom stories do not surpass the drift limit Δ=1.2% established by the code for SCT-EW record, but this limit is slightly surpassed in stories 2 to 4 for S56-EW record.

However, the hysteresis curves for the SCT-EW record depicted in Fig. 17 exhibit a stable behavior where some strength degradation is starting to show up, particularly in the first story, where an important number of columns yield at their base (Fig. 18). Column yielding is also detected in stories 2 to 9, primarily in the more slender frames (1 and 2), as shown in Fig. 18.

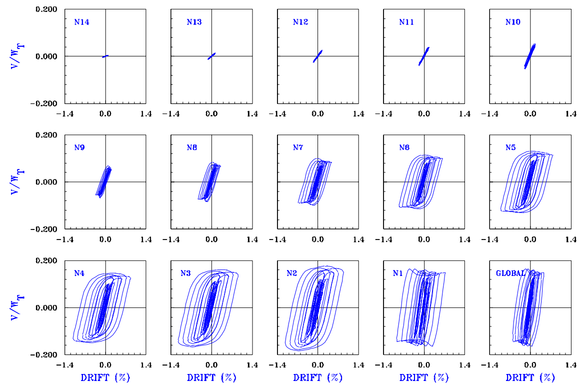



Figure 17. Hysteresis curves for all slender frames of IR2C building in the N-S direction under SCT-EW record, nominal strength.

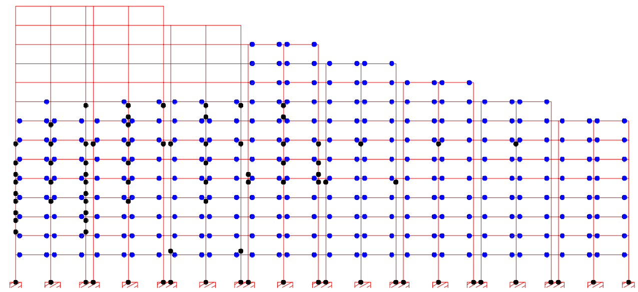


Figure 18. Yielding mapping envelope for all slender frames of IR2C building in the N-S direction under SCT-EW record, nominal strength.

The details of the dynamic response of the slender frames of IR2C building in the N-S direction under the S56-EW record are depicted in Figure 19. It can be observed that although an incipient dynamic instability could start after 80 seconds of response in the first six stories, particularly stories 2 and 3, the higher redundancy of the two-bay frames of IR2C allow the building to recover and survive these critical demands, so at the end of the record the dynamic response is stable (compared to the one-bay frames of IR2A).

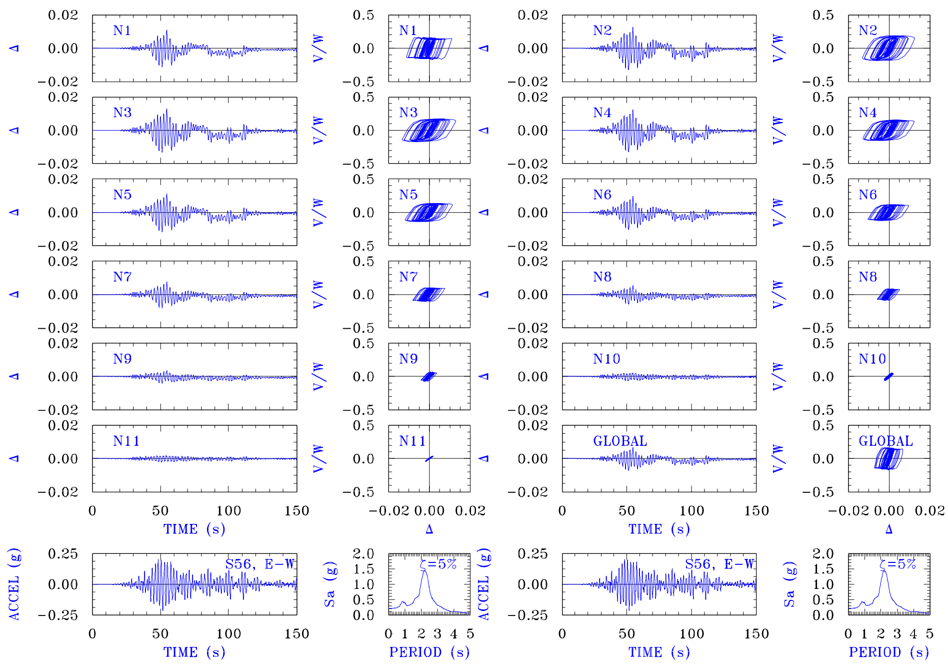



Figure 19. Details of the time history response of the first eleven stories of the slender frames of IR2C building in the N-S direction under S56-EW record, nominal strength.

From the yielding mapping of all elements that work inelastically under the action of S56-EW record (Fig. 20) one can observe that the yielding of the columns is minimized with respect to the one observed for IR2A (Fig. 14), particularly in the stories of reference. Therefore, and after a more detailed analyses of the results, one can conclude that the incipient shifting of the curves shown in Fig. 19 is primarily due to the yielding of beams, so that explains why IR2C building is able to recover and prevent the collapse for P-Δ effects, in contrast with IR2A building, where the important yielding of the columns favored a weak-story collapse mechanism magnified for P-Δ effects.

The results obtained for IR2C building when overstength sources are considered are not shown for space constraints, but it can be said that column yielding is considerably minimized, the ductility demands reduced, and the hysteresis loops are more stable.

It can be affirmed that the dynamic behavior of IR2C setback building in its weak (slender) direction is much more adequate than the one obtained for IR2A building. As both buildings are similar in geometry, in initial (elastic) dynamic properties, and were designed using the same code criteria, one can conclude that the key for survival and the superior performance of IR2C building with respect to IR2A building is the higher redundancy of the frames in the slender N-S direction.

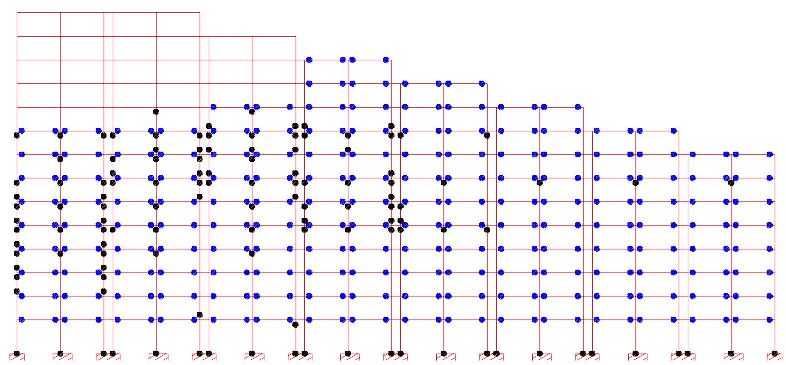


Figure 20. Yielding mapping envelope for all slender frames of IR2C building in the N-S direction under S56-EW record, nominal strength.

## CONCLUSIONS

This paper presented the study of two irregular (setback and slender), fourteen-story reinforced concrete (RC), moment-resisting framed buildings that were designed to fulfill the seismic provisions of the 1993 Mexican code for the soft soil conditions of Mexico City. The irregular buildings do not satisfy four regularity conditions defined by the code and were designed in the slender direction close to the limiting story drift angle Δ=1.2% established by the code. The impact of having single-bay frames instead of multiple-bay frames in the slender direction in the seismic performance of the buildings was studied. Several recorded and simulated accelerograms associated to the design spectra for the lakebed region of Mexico City were used for nonlinear dynamic analyses. Story drift ratios associated to the original design were compared with peak dynamic story drift angles computed from nonlinear dynamic analyses. Structural yielding was studied and associated to hysteretic, deformation and strength demands.

The results obtained from nonlinear dynamic analyses suggest that the slender direction of setback buildings with one-bay frames is extremely vulnerable if the buildings are designed close to the limiting drift angle Δ=1.2% established by Mexican codes because the yielding of columns favors a weak story failure mechanisms that can be triggered by P-Δ effects. The behavior of such structures is much improved when a bay is added in the slender direction (two-bay frames). This simple practice minimizes the yielding of columns with respect to the one observed for the single-bay framed setback building, favoring that most of the nonlinear action concentrate on the yielding of beams, so a weak-beam strong-column mechanism is more likely to happen, therefore, minimizing the risk of a collapse due to P-Δ effects. Therefore, it is clear that the higher redundancy of the building with two-bay frames was the key for their survival, particular when compared with a twin structure with one-bay frames.

Perhaps it is not new to highlight the benefits of having higher redundancy in framed structures. In fact, good structural engineers and professors promote the use of redundant structures for seismic design. Nevertheless, to the author's knowledge, no seismic code worldwide penalizes the seismic design of buildings with single-bay frames in one direction, and perhaps they should. The fact is that there are many buildings with single-bay frames located in seismic zones of the world, many of them are slender and/or have other irregularities, and some of them have suffered heavy structural damage or collapsed during recent strong earthquakes. The geometry of IR2A building (single-bay framed building) was borrowed from the project of a hotel built in Manzanillo, Mexico, with the difference that for these hotel there are RC shear walls in bays 3 and 6. This study highlights the need to research further if the design of one-bay framed structures should be penalized for both regular and irregular structures, of if they should be only penalized for slender structures.

With respect to the setback irregularity, this study did not find any undesirable stress concentration or high yielding demand in the neighborhood of the setback, which one should say, it was a surprising result. However, it is clear for the author of this study that further research is needed before conclude something important on this regard.

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