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SEISMIC RESPONSE OF CODE-DESIGNED MEDIUM-RISE SLENDER, MOMENT-RESISTING FRAME STEEL BUILDINGS IN SOFT SOILS

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ABSTRACT

The study of 8-story and 21-story slender, special moment resisting framed steel buildings (SMRF-SB) is presented. Buildings were designed according to the seismic provisions of Mexico's Federal District Code (RCDF) for a maximum story drift ratio close to the limiting drift Δ=1.2% established in the code for buildings designed according to what it is known as "the main body of the code". As buildings do not satisfy the limiting slenderness ratio H/L ≤2.5 established in RCDF for regular structures and other three requirements of structural regularity, then, the *Q´* factor allowed in the code to reduce seismic forces for design purposes was affected by a 0.8 reduction factor, as established in RCDF ($Q'_{irregular}=0.8Q'_{regular}$). Buildings were designed for the soft soil conditions of the lakebed region of Mexico City. Nonlinear dynamic analyses of representative frame models of subject structures were conducted. Several recorded and simulated accelerograms associated to the design spectra of RCDF for the lakebed region were used for the nonlinear dynamic analyses. Story drift ratios associated to the design according to RCDF 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. It can be concluded that although peak story ductility demands are within what it is assumed in the code, and the structural yielding is consistent with a weak-beam, strong-column failure mechanism, the peak dynamic story drift angles considerably surpassed the story drift limit established in the code. This condition has to be revised, particularly from a consistent seismic design methodology viewpoint, because underestimating design drift angles may cause: (1) very important damage to nonstructural elements and, (2) have a negative impact in the review of building separations to prevent a potential structural pounding.

Introduction

Eleven conditions of regularity that building structures must satisfy to be designed as regular buildings are defined in the seismic provisions of Mexico's Federal District Code (RCDF)

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[i.e., Tena-Colunga 1999, NTCS-2004 2004]. If one or more of these eleven regularity conditions are not fulfilled, then, the building is classified as an irregular building. According to NTCS, the reductive seismic force factor Q' has to be reduced by 20% for the design of irregular buildings ($Q'_{irregular}$=0.8$Q'_{regular}$), which must be designed for higher forces but still be checked to comply with the story lateral drift criteria specified for regular buildings, that is, lateral deformations obtained from the analyses must be multiplied by Q in both cases, as schematically depicted in Fig. 1. The 0.8 reduction factor was proposed 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. 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 (1999). These regularity conditions are based on what it was learned worldwide after major earthquake events. Some of them are mostly based on good engineering judgment, experience and common sense, rather than in detailed analytical or experimental research.

The regularity conditions remained unchanged until 2004, as some changes for the definition and design of irregular structures are included in the seismic provisions for RCDF-2004. Mostly, the original eleven regularity conditions remain the same. However, the statement devoted to prevent a soft story condition was redefined and now it is more conservative compared to previous versions taking into account, among other material, recent research findings summarized in Tena-Colunga (2003 and 2004). Among the changes in the design process are that if one building does not satisfies one regularity condition, then $Q'_{irregular}$=0.9$Q'_{regular}$ must be used for the design. If two or more regularity conditions are not satisfied, $Q'_{irregular}$=0.8$Q'_{regular}$. If a building has: (a) 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) a well-defined soft story condition, this is, former regularity condition 10 of RCDF-93 is not fulfilled (Tena-Colunga 1999), then the building must be classified as strongly irregular and use $Q'_{strongly\text{-}irregular}$=0.7$Q'_{irregular}$.

The studies conducted in slender steel, moment-resisting framed buildings that were designed to fulfill the seismic provisions of RCDF-2004 are summarized in this paper. These irregular buildings do not satisfy two regularity conditions defined by the code. Therefore, they were designed according to the specified provisions for irregular buildings and for the lakebed zone of Mexico City. The design criteria for the buildings of reference and some of the most important results obtained from nonlinear dynamic analyses are briefly summarized in following sections.

Subject Buildings

Two slender, SMRF steel buildings 8-story (IR5B) and 21-story (IRB6) in height were designed as apartment buildings according to the design spectra of what it is know as the "main body" of Mexico's Federal District Code (RCDF) for soft soil conditions (Fig. 2). Their corresponding building plans and 3D-ETABS models are depicted in Figs. 3 and 4. IR6B building is 66 m in height; the first story is 4 m in height and the remaining 20 stories alternate their height: even stories are 3.4 m in height whereas odd stories are 2.8m in height. IR5B building is 41 m in height; the first story is 6 m in height and the remaining stories have 5 m in height. In fact, IR5B building is a less-irregular, more simplified version of IR6B building.

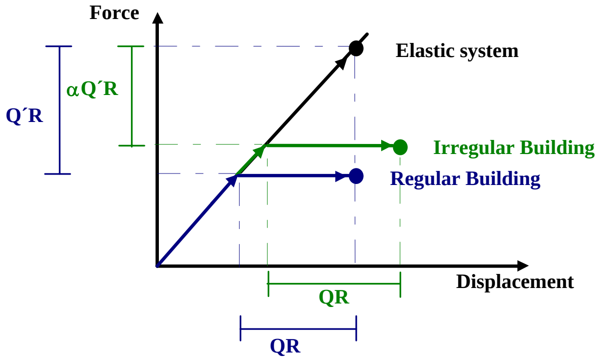

Figure 1.Schematic illustration of the design procedure for regular and irregular buildings according to RCDF code.

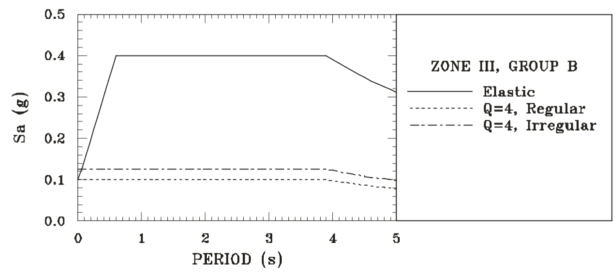

Figure 2."Elastic" and inelastic design spectra for the soft soils of the lakebed zone of RCDF code, according to "the main body of the code".

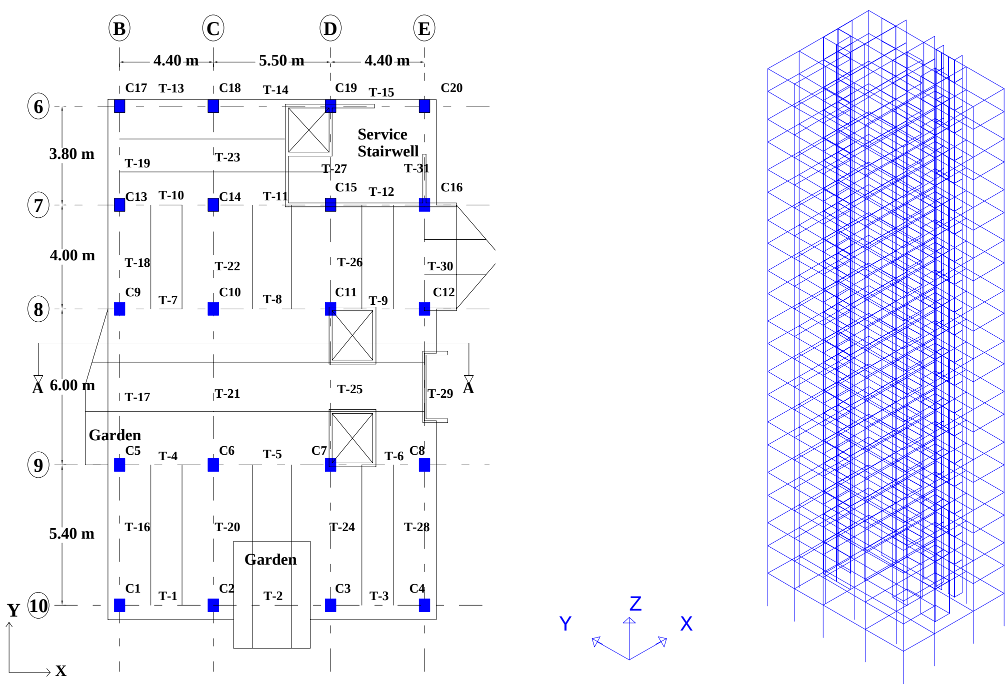

Figure 3. Plan and 3D ETABS model for IR6B building (dimensions in meters).

The structural system for both buildings for earthquake loading is composed of special moment resisting frames (SMRFs). Therefore, a seismic modification factor *Q*=4 was assumed for their design. IR6B building does not satisfy the following two conditions of structural regularity as address by RCDF code (i.e., Tena-Colunga 1999):

- Condition 2, slenderness: the ratio of the height of the building to the smallest plan dimension exceeds 2.5 (H/L_2=66/15.9=4.5>2.5).
- Condition 6, diaphragm discontinuities: There are diaphragm openings greater than 20 percent the plan dimension of the structure in the parallel direction. In addition, the open areas introduce important asymmetries in plan (Fig. 3).

Therefore, according to RCDF-2004, the $Q´$ factor for design must be affected by a 0.8 reduction factor ($Q'_{irregular}$=0.8$Q´_{regular}$). The resulting inelastic design spectrum for these irregular buildings is depicted in Fig. 2.

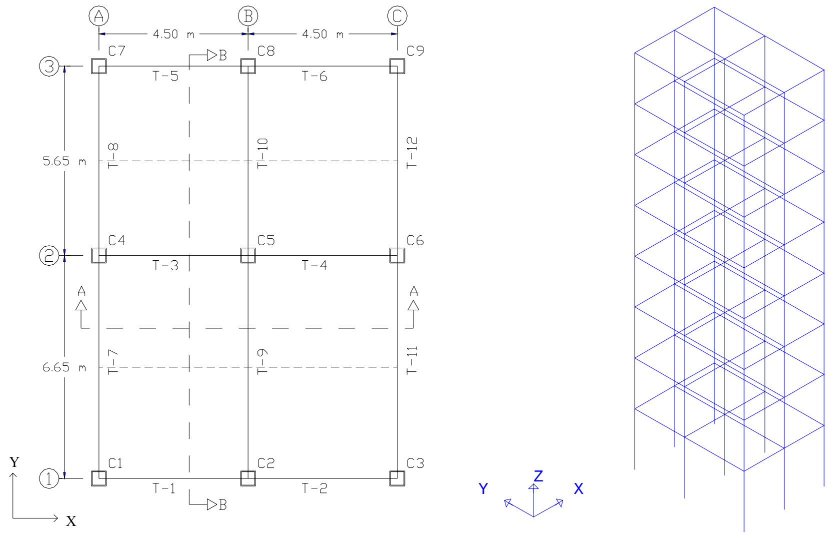

Figure 4. Plan and 3D ETABS model for IR5B building (dimensions in meters).

Buildings were designed according to the seismic provisions of RCDF for a maximum story drift ratio close to the limiting drift Δ=1.2% established by the code for buildings designed according to what it is known as "normal procedure of the main body of the code" (Figs. 5 and 6). The "normal procedure of the main body of the code" is the design procedure prescribed since 1987 that basically endorses an equivalent "elastic" design spectrum which is already reduced for overstrength in a somewhat obscure way, as described elsewhere (Rosenblueth *et al.* 1989).

It is worth noting that design procedures as conceptually transparent as possible are included in recent Mexican seismic codes (i.e., Tena-Colunga *et al.* 2009), among them, Appendix A of RCDF-2004. In such procedures: (a) the parameters that were taken into account to assess the earthquake hazard and define the elastic design spectra are clearly presented and, (b) the sources that can be accounted for reducing the design spectra for the collapse prevention limit state are defined. Nevertheless, the "normal procedure of the main body of the code" is still the most frequently used in Mexican design practice, so it is important to assess the effectiveness of structural designs made with such procedure.

Both buildings were analyzed and pre-designed with the help of ETABS and its Steeler module. The lateral stiffness of concrete masonry walls of the elevator's core was included in the modeling. Final cross sections for beams and columns were defined after reviewing that they complied with the design requirements of the structural steel guidelines of RCDF-2004, which are similar to those addressed in LRFD code. All elements were supposed to be made of A-36 steel. Columns are of square box cross sections, whereas beams are W sections. According to one traditional design practice of many structural engineers in Mexico, cross sections for beams and columns were typified for a number of stories, as shown in Tables 1 to 3 for the final designed sections. Resulting dynamic properties for the design are summarized in Table 4.

Table 1. Designed sections for columns of IR5B and IR6B buildings					
IR5B building			**IR6B Building**		
Story	**Columns**	**Dimensions (inches)**	**Stories**	**Columns**	**Dimensions (inches)**
1-2	C1-C9	18 x 18 x 0.625	1-4	C1-C20	28 x 28 x 1.125
3-8	C1-C9	18 x 18 x 0.5	5-10	C1-C20	24 x 24 x 0.9375
			11-16	C1-C20	20 x 20 x 0.75
			17-21	C1-C20	16 x 16 x 0.625

Table 2. Designed sections for beams of IR6B building					
Stories	**Beams**	**Section**	**Stories**	**Beams**	**Section**
1-9	T-1, T-3 to T-15	W24x131	10-12	T-1, T-3 to T-15	W24x131
	T-2	W24x146		T-2	W24x146
	T16 to T-31	W24x103		T-16 to T-31	W24x94
13	T-1, T-3 to T-15	W24x117	14-16	T-1, T-3 to T-15	W24x117
	T-2	W24x131		T-2	W24x117
	T-16 to T-31	W24x94		T-16 to T-31	W24x94
17	T-1 to T-15	W24x94	18-21	T-1 to T-15	W24x94
	T-16 to T-31	W24x94		T-16 to T-31	W24x76

Table 3. Designed sections for beams of IR5B building		
Stories	**Beams**	**Section**
1-3	T-1 to T-12	W16x89
4-6	T-1 to T-12	W16x67
7-8	T-1 to T-12	W14x43

Table 4. Dynamic characteristics of IR5B and IR6B buildings								
Mode	**IR5B (W_{I5B}=558.3 Ton)**				**IR6B (W_{I6B}=5704.5 Ton)**			
	T (s)	Modal mass (%)			T (s)	Modal mass (%)		
		Y	X	θ		Y	X	θ
1. First mode of translation	1.530	79.86	0.00	0.00	1.646	3.59	63.10	5.79
2. First mode of translation	1.440	0.00	79.74	0.08	1.615	70.08	3.50	0.05
3. First torsional mode	1.020	0.00	0.06	81.03	1.197	0.07	5.22	69.04
4. Second mode of translation	0.537	12.04	0.00	0.00	0.566	4.03	9.47	0.83
5. Second mode of translation	0.499	0.00	12.63	0.01	0.559	9.92	4.10	0.30
6. Second torsional mode	0.361	0.00	0.01	11.41	0.430	0.02	1.89	11.32

The design drift envelopes in both orthogonal directions as well as the design stress ratios for beams and columns (as computed by the Steeler software) are shown in Figs. 5 and 6 for IR6B and IR5B buildings respectively. It can be observed from Fig. 5 that although the design drifts in the X direction are closer to the drift limit Δ=1.2% established by the code ("RDF-b") than the design drifts in the Y direction, the resulting design stress ratios in the Y direction of the building are notoriously higher than in the X direction. In contrast, the design drifts and stress ratios obtained for IR5B building are more balanced in both orthogonal directions (Fig. 6). Nevertheless, it is worth noting that the design strategy of typifying cross sections for beams and columns for a number of stories, as often done in Mexican design practice, leads to the over-design of many structural members and then an important overstrength may be developed.

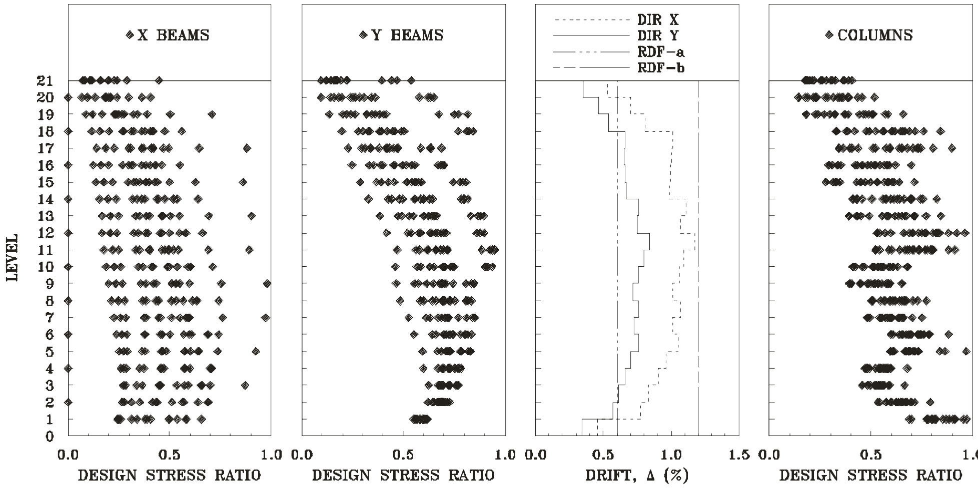

Figure 5. Design drifts and stresses ratios for IR6B building

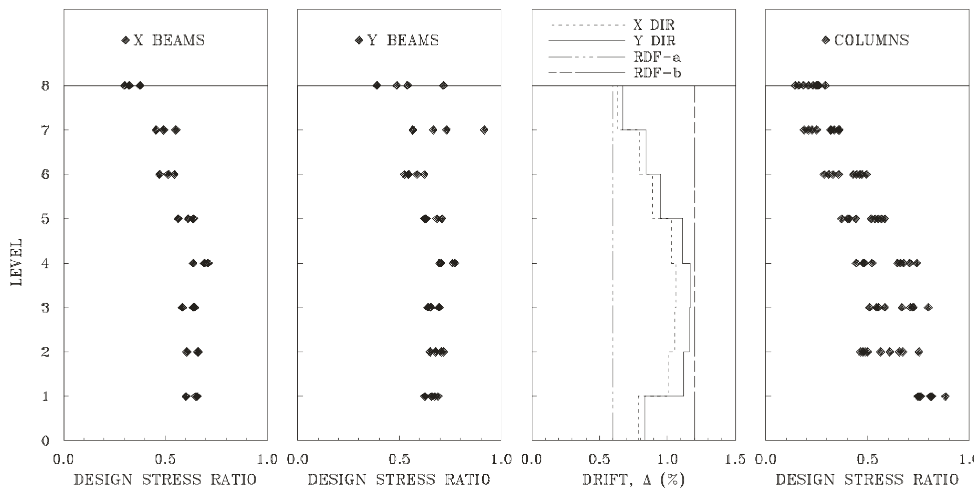

Figure 6. Design drifts and stresses ratios for IR5B building

Acceleration Records

Acceleration records obtained in Mexico City at stations SCT 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. The main characteristics of some of the selected records are summarized in Table 5. The reported peak pseudoacceleration (S_a) corresponds to the one obtained for a 5% damped elastic response spectrum and occurs for the reported site period (T_{site}).

Critical acceleration records for both IR5B and IR2B buildings were SCT-EW record for the “real” records and S56-EW record for the artificial records. In fact, S56-EW record (Fig. 7) was the most demanding one for both buildings.

Table 5. Characteristics of some of the selected acceleration records					
	09/19/85 Records		Artificial Records		
	SCT-EW	TBOM-NS	S05-EW	S56-EW	S84-EW
Duration (s)	160	150	250	250	250
Max. S_a (g)	1.0	0.75	0.86	1.44	1.03
T_{site} (s)	2.01	1.92	2.15	2.20	1.38

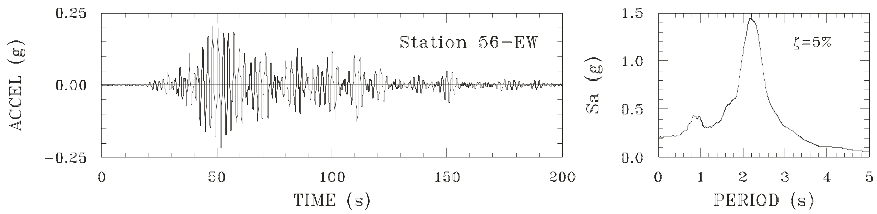

Figure 7. S56-EW Acceleration record and response spectrum for ζ=5%

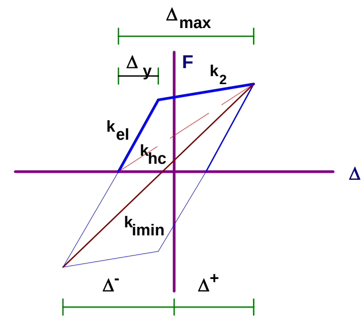

Figure 8. Definitions for half cycles

Nonlinear Dynamic Analyses

Nonlinear dynamic analyses were performed for all buildings using the acceleration records presented in Table 5 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) representative perimeter frames (B and 10, Fig. 3) for IR6B building in its main orthogonal directions (E-W and N-S) and, (b) 2D models that account for the interaction among all frames for IR5B building in its main orthogonal directions. The latter modeling accounts for the transmission of lateral forces among all 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.) or including discontinuities.

Nominal plastic capacities for A-36 steel with a pre-to-post yielding stiffness ratio (k_2/k_1) of 2%, were considered for beams and columns of both models. Additional analyses were performed for IR6B building considering k_2/k_1 of 5% and 10%. An equivalent viscous damping ratio ζ=3% was used for all models, as values of ζ ranging from 2% to 4% have been measured experimentally in existing steel buildings. P-Δ effects were included in the nonlinear analyses. Soil-structure interaction was not included as the purpose of the study was to evaluate code provisions regarding the reduction factors, drift angles, strength and stiffness criteria for irregular structures alone. Therefore, the study should be done without introducing other variables that may interfere these code criteria.

Among the dynamic results processed from the nonlinear analyses were normalized story hysteresis curves (V/W_T vs Δ), as well as yielding mappings for time-steps associated to peak dynamic responses and yielding mapping envelopes for all time-steps (detect all elements that responded inelastically at least once). From story hysteresis curves the following envelopes were also obtained: (a) peak dynamic story drift angles ($\Delta=\Delta_i/H_i$), (b) story drift angles related to the first yielding of resisting elements, usually beams ($\Delta_{first\text{-}yield}$), (c) peak story ductility demands (μ), (d) equivalent story drift at yielding (Δ_y), corresponding to a bilinear plastic envelope of the hysteretic response, (e) maximum dynamic story shear indexes (V/W_T), (f) peak to peak story shear stiffnesses computed from the largest amplitude cycles (k_{eff}), normalized with respect to the elastic story stiffness (k_{el}), (g) Average story secant stiffness of nonlinear half cycles (k_{ave}) normalized with respect to the elastic story stiffness (k_{el}) and, (g) total number of inelastic half cycles (small and large amplitude) for each story. Half cycles were considered for assessing k_{ave} from k_{hc} (Fig. 8) because of the important differences often observed in the amplitude of adjacent positive and negative half cycles due to the variation of the intensity of the ground motion. Finally, the developed overstrength capacity with respect to the design seismic coefficient was also assessed from the processed results.

For space constraints, some of the most important results obtained for the Y direction, the critical direction of analysis of both buildings, are presented and briefly discussed.

Peak response envelopes for the parameters described above (i.e., Fig. 8) for the Y direction of IR5B building under the action of SCT-EW, S05-EW and S56-EW records are presented in Figure 9 when considering k_2/k_1=2%. It can be observed that for all records peak dynamic story drifts (Δ) considerably surpassed the design envelope and the drift limit Δ=1.2% (RDF-b) established by the code. Nevertheless, the nonlinear response is moderately strong and within the building capacity in most stories, as peak story ductility demands (μ) are smaller than 3.0 and k_{ave}/k_{el} is between 0.4 and 0.6 for the most demanded middle stories under the SCT-EW and S56-EW records. The equivalent number of nonlinear cycles in the most demanded stories is 13 under the SCT-EW record and considerably increases to 24 for the S56-EW record. Envelopes for the story drift angles related to the first yielding of structural elements within the story ($\Delta_{first\text{-}yield}$), in this case study always beams are compared to the drift limit Δ_{serv}=0.4% established in Appendix A of NTCS-2004 for elastic response under the service earthquake. As observed in Figure 9, stories start yielding at drifts ranging from 0.65% (first story) to 1.28% (sixth story) under the S56-EW record, considerably higher to the drift limit Δ_{serv}=0.4% established in Appendix A of NTCS-2004. These results are directly related to the important lateral flexibility of the building in the Y direction (T_y= 1.53/8=0.19N, where N is the number of stories). Therefore, the lateral flexibility of the corresponding stories is also high. Surprisingly, it can also be observed in Figure 9 that for each story, $\Delta_{first\text{-}yield}$ can vary depending on the frequency content of the ground motions.

The building has an important lateral strength. The demanded peak base shear capacity was V/W_T=0.451, higher than its design base shear coefficient: 0.125 V/W_T. Therefore, the demanded overstrength (Ω_0 in American codes, R in Mexican codes) was Ω_0=R=3.6, which is notoriously higher than R=2 specified in Appendix A of RCDF-2004 for such structuring, and still higher than Ω_0= 3 specified in ASCE 7-05 (2005). It is worth noting that IR5B building can still develop more overstrength and that this important additional strength is primarily related to the practice of typifying cross sections every M stories (i.e., Fig. 6). It is also worth noting that the nonlinear response is due primarily to beam yielding from stories 1 to 7 and the yield of first story columns at their base. Therefore, a weak-beam, strong-column mechanism is observed, as assumed in the design.

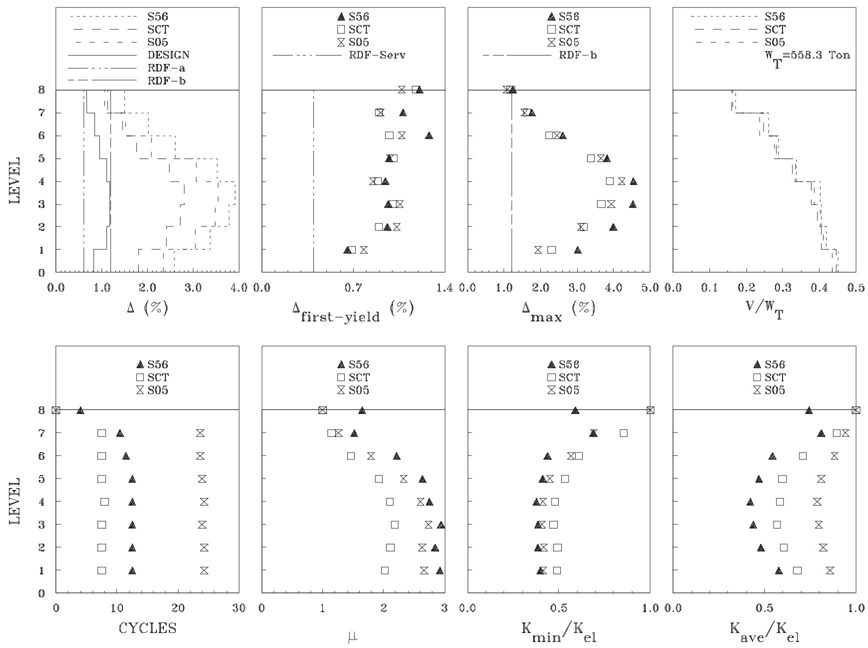

Figure 9. Peak response envelopes for IR5B building in de Y direction, considering k_2/k_1=2%

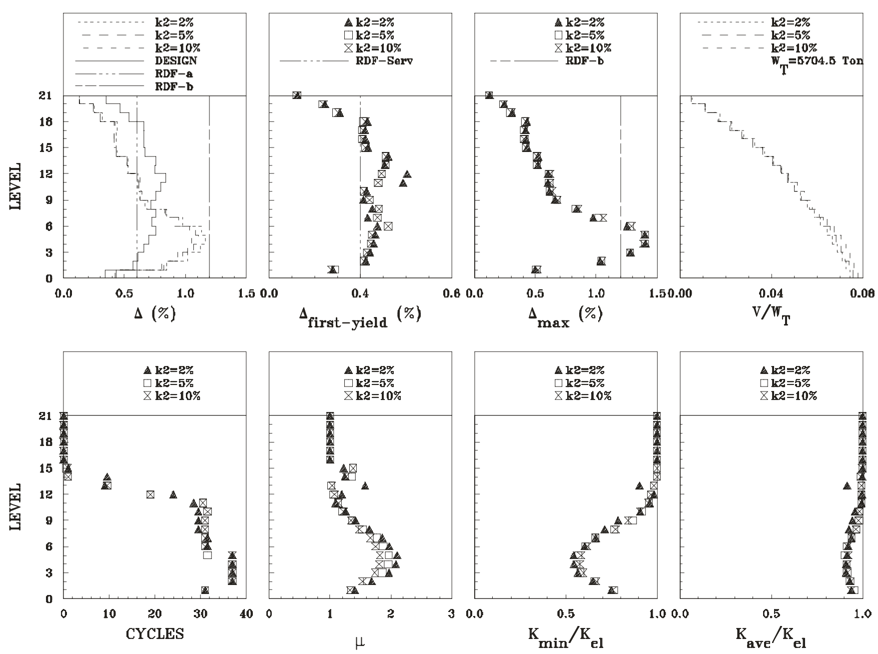

Figure 10. Peak response envelopes for frame B of IR6B building under S56-EW record

Peak response envelopes for frame B of IR6B building under the action of the critical S56-EW records are depicted in Figure 10 when considering k_2/k_1=2%, 5% and 10%. The following observations can be made: (1) peak dynamic story drifts (Δ) considerably surpassed the design envelope and the drift limit Δ=1.2% (RDF-b) established by the code in the lower stories, (2) Although the equivalent number of nonlinear cycles in the most demanded stories are close to 40, the nonlinear response is small and within the building capacity in most stories, as peak story ductility demands (μ) are smaller than 2.0 and the smallest k_{ave}/k_{el} is 0.9 for the most demanded middle stories. That is why the are no significant differences for the results obtained for the different k_2/k_1 ratios that were considered, (3) Stories start yielding at drifts ranging from 0.28% (first story) to 0.61% (twelfth story), and usually higher to the drift limit Δ_{serv}=0.4% established in Appendix A of NTCS-2004, (4) The lateral flexibility of the building in the Y direction is T_y= 1.615/21=0.077N, and that explains why these drifts are smaller than those obtained for IR5B, (6) The frame has an important lateral strength (V/W_T) that ranged from 0.075 to 0.078. The demanded overstrength (Ω_0=R) varied from 2.4 to 2.5, higher than R=2 specified in Appendix A of RCDF-2004 for such structuring despite the fact peak nonlinear demands were relatively small and, (5) a weak-beam, strong-column mechanism is also observed.

Concluding Remarks

The study of the 8-story and 21-story slender, SMRF-SB designed as irregular building according to the seismic provisions of “the main body” of Mexico’s Federal District Code (RCDF) for a maximum story drift ratio close to the limiting drift Δ=1.2% leads to the following observations: (1) although peak story ductility demands are within what it is assumed in the design and the structural yielding is consistent with a weak-beam, strong-column failure mechanism, the peak dynamic story drift angles considerably surpassed the story drift limit established in the code for this design method. This condition has to be revised, particularly from a consistent seismic design methodology viewpoint, because underestimating design drift angles may cause: (1) very important damage to nonstructural elements and, (2) have a negative impact in the review of building separations to prevent a potential structural pounding. Future research works will be devoted to review if the conceptually transparent design procedure included in Appendix A of RCDF-2004 could improve the predicted displacement response.

References

ASCE-7, 2005. *Minimum design loads for buildings and other structures*, ASCE Standard ASCE/SEI 7-05, American Society of Civil Engineers, ISBN 0-7844-0809-2

NTCS-2004, 2004. *Normas técnicas complementarias para diseño por sismo*, Reglamento de Construcciones para el Distrito Federal, Gaceta Oficial del Departamento del Distrito Federal (in Spanish).

Rosenblueth, E., Ordaz, M., Sánchez-Sesma, F. J. and Singh, S. K., 1989. The Mexico earthquake of September 19, 1985-Design spectra for Mexico’s Federal District, *Earthquake Spectra*, 5 (1) 273-291.

Tena-Colunga, A., 1999. International seismic zone tabulation proposed by the 1997 UBC code: Observations for Mexico, *Earthquake Spectra*, 15 (2), 331-360.

Tena-Colunga, A., 2003. Dynamic response of buildings with soft first story designed according to the seismic guidelines of a modern building code, *Proceedings, Response of Structures to Extreme Loading XL-2003*, Toronto, Canada, Paper No. 190 (15-06), CD-ROM, August.

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

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