Non-linear step by step seismic response and Push-over analysis for reinforced concrete buildings in Mexico City

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Abstract

Based on previously designed 9 and 17 level reinforced concrete buildings of ductile frames, non-linear static analysis with increased monotonically lateral loads (Push-over) are made in order to determine collapse, and the buildings' responses against the inelastic seismic analysis results are compared with the SCT-EW-85 record. This is designed with the Main Body (A case) and with the A Appendix (B case) conditions of the Seismic Technical Norms of the Mexico City Construction Code (RDF-04), satisfying the maximum story distortion limits of the service and collapse conditions; the buildings (offices) are in the III_b compressible seismic zone with soil dominant period T_s = 2 seconds.

1 Introduction

It is well known that seismic engineering presents important advances in countries where the seismic movements are of great intensity. In Mexican earthquakes from September 1985, several investigations have been made with the aim of understanding the seismic phenomenon and to give solutions to avoid human losses and to reduce damage to structures. In this work, the non-linear seismic (Push-Over) and the dynamic inelastic step by step behavior in 9 and 17 level buildings with reinforced concrete frames located in the compressible zone of México City are determined and compared. The design is made with the RDF-04 Construction Code [1], according to the service and failure limit states

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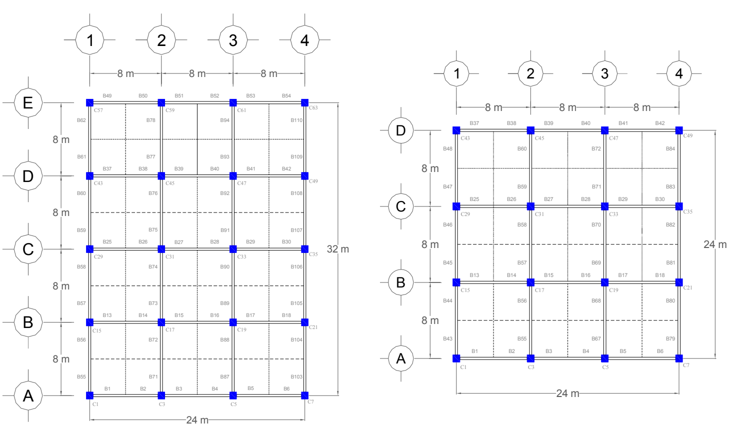
established in its corresponding Complementary Techniques Norms (CTN-Seismic and CTN-Concrete). The structures were designed as follows: A) with the design spectrum of the Main Body (MB) of the CTN-Seismic (seismic zone III_b); B) with the design spectrum according to the soil dominant period, defined in the A Appendix of the CTN-Seismic (T_s= 2 seconds). The inelastic seismic responses in time history were analyzed with the records representative of the maximum compressible soil damage in Mexico City during the earthquake of September 1985. For the Push-over analyses, the responses under different lateral load distributions were calculated and the collapse mechanisms, base shear force-roof lateral displacement relationships and the maximum demands of local and global ductility were determined. The influence of the possible over-resistance sources is included. With the results of this work it is shown that there are no important variations between A (Main Body) and B (A Appendix) cases. The A Appendix criterion allows one to determinate displacement of the conditions of service and collapse to which the structure may be subject.

2 Elastic response calculation

2.1 Structure description and design procedure

The structures are 9 and 17 level office buildings of reinforced concrete, symmetric and regular in plan and elevation. They have a foundation rigid box. The structural system is formed with reinforced concrete frames by beams and columns rigidly connected by a solid slab of 10 cm thick. The concrete is class 1 with f'_c= 250 kg/cm^2 and elastic modulus E_c= 14,000 times square root of f'_c and the longitudinal and transverse steel with f_y= 4,200 kg/cm^2. The 9 level building has friction piles and the 17 level building has point piles; for the first one the base flexibility effects are considered. Figs. 1 and 2 show the main details in the



9 Levels **17 levels**

Figure 1: Structural plant-type (dimensions in meters), 9 and 17 level buildings.

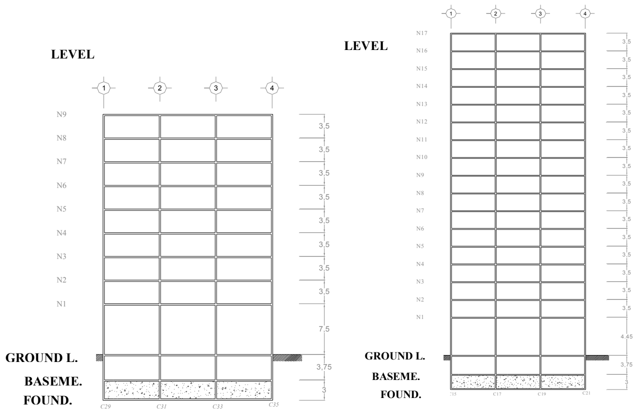


Figure 2: Transversal cuts (dimensions in meters), 9 and 17 level buildings.

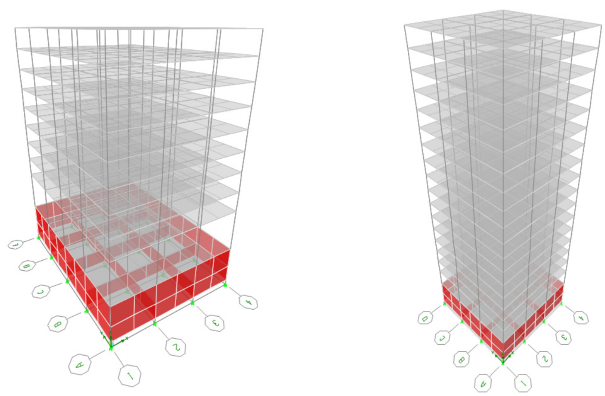


Figure 3: Mathematical model for 9 and 17 level buildings.

plan and the elevation of the buildings under study; fig. 3 presents the mathematical models analyzed with the computer. The column and beam resistances were designed for the last mechanic elements of the spectral modal dynamic analysis, considering the three-dimensional structural behavior more than the vertical load effects (dead loads and live loads) and second order effects, and for the ductile frame special requirements were established by CTN-Concrete, due to the use of seismic behavior factor Q= 3

2.2 Periods of vibration

Table 1 compares the three periods of vibration modes (directions X, Y and θ) of the 9 and 17 level buildings respectively. There are no differences between the A and B cases, as expected. For the 17 level building, due to the symmetry of the structure, periods in the direction X and Y are the same. Fig. 4 shows the design spectra of the RDF-04 of the Main Body (Zone III_b) and the A Appendix (T_s= 2 seconds), the elastic and inelastic response spectra of record SCT-EW-85

Table 1: Vibration periods for 9 and 17 level buildings, A and B cases.

Period (seconds)	A and B cases: 9 levels			A and B cases: 17 levels	
	X	Y	θ	X= Y	θ
T_1	1.502	1.460	1.132	1.752	1.346
T_2	0.563	0.551	0.420	0.643	0.518
T_3	0.335	0.332	0.254	0.374	0.306

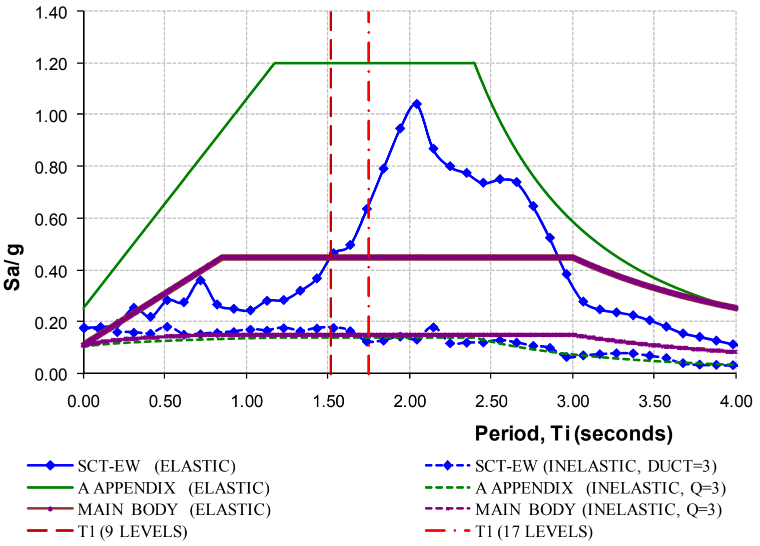


Figure 4: Vibration fundamental period locations of the 9 and 17 level structures, regarding the design spectra (Q = 1 y 3) and the SCT-EW-85 accelerogram response spectra (μ = 1 y 3).

(critical viscous damping of 5%), as well as the location of the critical period of the 9 level building (T_{1x}= 1.502 seconds) and 17 level building (T_{1x}= 1.752 seconds).

2.3 Maximum story drifts

The 9 level building subject to an earthquake in both directions accomplishes with the permissible limit of 0.012 specified in the Main Body (A case), and it is slightly above the limit of 0.04 of the A Appendix (B case) earthquake in the X direction (see fig. 5); the collapse limit of 0.03 was not a condition to govern the behavior of the design of the building. The 17 level building accomplishes with both permissible limits, be it a little above the condition of service of A

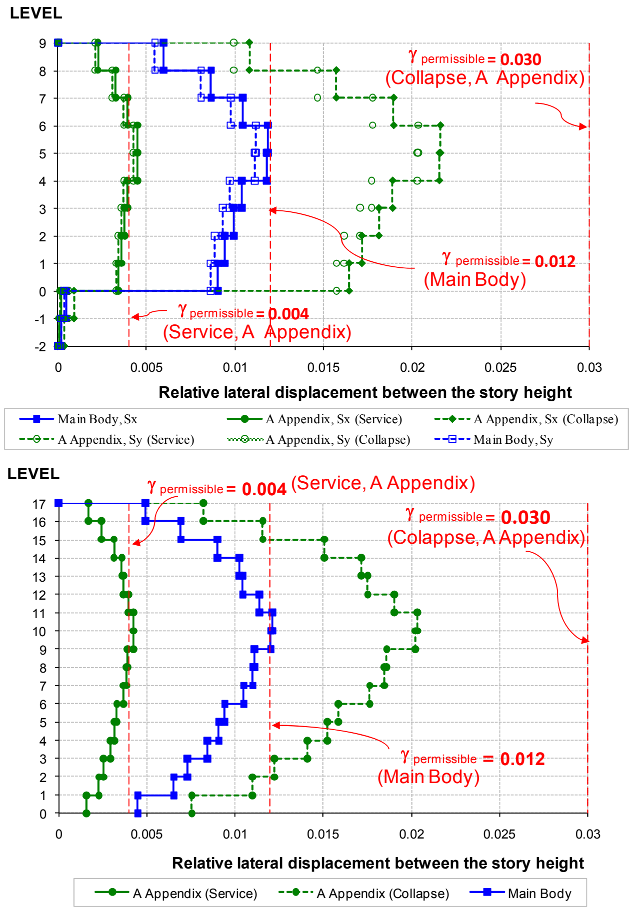


Figure 5: Relative lateral displacement between the story height ratios (story drifts), X and Y directions earthquake, spectral modal dynamic seismic analysis, 9 and 17 level buildings, A and B cases.

Appendix; the story drifts in this structure are closer to the collapse limit than those of the 9 level model (see fig. 5).

3 Step-by-step calculation of the inelastic response

Step-by-step inelastic dynamic responses are determined by using the SCT accelerogram, E-W component, registered on September 19th 1985. The responses were calculated for the internal structural axes, the C axis for the 9 level building and B axis for the 17 level building; such frames were "calibrated" to ensure static and dynamic behavior similar to the three-dimensional model. The results of the step-by-step analyses are shown based on the following nomenclature: MB-ELAS: Main body-elastic; MB-INE-N: Main body-inelastic-nominal resistances; MB-INE-OR: Main body-inelastic-over resistances; AA-ELAS: A Appendix-elastic; AA-INE-N: A Appendix-inelastic-nominal resistances; AA-INE-OR: A Appendix-inelastic-over resistances.

3.1 Maximum horizontal displacements and demands of global ductility

In all cases of the 9 level experimental inelastic behavior, the maximum lateral displacements are very similar, with the exception of the case that is designed with the A Appendix and nominal resistances, where the maximum horizontal displacement is bigger than the A case (MB-INE-N); the inelastic responses with over-resistance effects are virtually the same in both A and B cases. Generally the over-resistance models have lower demands for global ductility, μ_G (see table 2); the inelastic responses of the relative horizontal displacement between a high story with over-resistance effects are practically the same in both A and B cases. In the 17 level models, with the exception of the B case structure with nominal resistances (AA-INE-N), the μ_G values are higher in the A case; the global ductility demands of the A and B cases with nominal resistances are very similar; the same goes for the A and B cases with over-resistances. In no case was μ_G over 3, equal to the value of the seismic behavior factor (Q) with which the elastic design spectrum of the A and B cases were reduced.

Table 2: Calculated global ductility maximum demands comparison of 9 and 17 level buildings, inelastic step-by-step seismic analysis (nominal and over-resistances), A and B cases.

Case		Δ_Y (cm)		Δ_{max} (cm)		μ_G	
		9 levels	17 levels	9 levels	17 levels	9 levels	17 levels
A	MB-N	16.29	21.08	33.72	43.83	**2.07**	**2.08**
	MB-OR	19.70	30.97	33.47	50.61	**1.70**	**1.63**
B	AA-N	14.34	19.24	37.57	42.85	**2.62**	**2.23**
	AA-OR	20.52	29.01	33.23	47.01	**1.62**	**1.62**

3.2 Base shear force-roof lateral displacement relationship

When an inelastic behavior is present, it is observed that as the structure dissipates a greater amount of seismic energy, the responses have a greater hysteretic area, and there are further reductions in the base shear force and the roof horizontal displacement. Figs. 6 and 7 compare the base shear force-roof lateral displacement relationship of the 9 and 17 level buildings, B case (A Appendix). The hysteretic curves of the 9 level models of the B case present light wider, which means the light has a larger distribution in the nonlinear range and higher energy dissipation; the over-resistance cases tend to behave elastically. The 17 level models of the B case tend to have a larger hysteretic

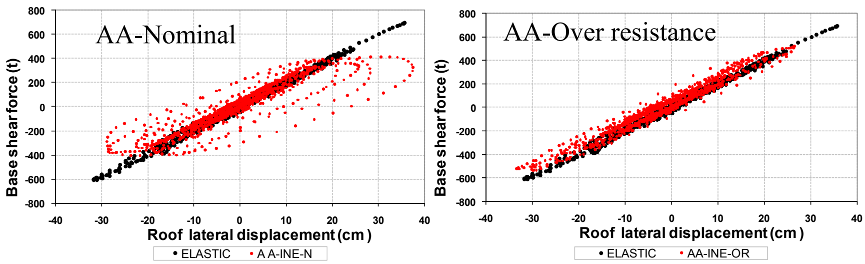


Figure 6: Base shear force–roof lateral displacement relationships, elastic and inelastic step-by-step seismic analysis (nominal and over-resistances), 9 level structures, B case (A Appendix).

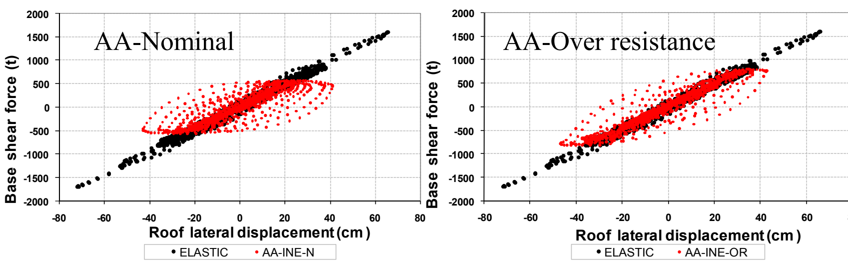


Figure 7: Base shear force–roof lateral displacement relationships, elastic and inelastic step-by-step seismic analysis (nominal and over-resistances), 17 level structures, B case (A Appendix).

area, and over-resistance cases show a light inelastic behavior, although in a less important way than the condition of nominal resistance.

3.3 Global distribution of plastic hinges and demands of local ductility

In 9 and 17 level buildings, in the A and B cases, the overall distribution of plastic hinges had a general tendency to the failure mechanism known as a “beam” type; that is, the plastic hinges are present in most of the beams and in only some columns, which is consistent with the design philosophy of “strong column-weak beam” of the RDF-04 code. Figs. 8 and 9 have global distributions of plastic hinges in the 9 level buildings with and without over-resistances. The shades of gray described different times in which each plastic hinge was introduced, from the dark gray color in the beginning until the end of the most intensive phase of the record SCT-EW-85, using in these cases the step-by-step analyses. The distributions of plastic hinges for the 9 level models with nominal resistance are very similar to the B case in that the bottom of all the columns on the ground floor yielded; in the A case only one of them does. Cases of over-resistance distributions are similar in both cases, although with a slightly higher number of yielded elements in case B. There were no significant differences between the A and B cases of the 17 level buildings or the 9 level buildings.

Fig. 10 shows the maximum local ductility demands developed in the beams for the A and B cases of the structural axes of the 9 and 17 level buildings in this study; the columns behave almost in the elastic range, with maximum demands of less than 1.5. The demands of local ductility in the beams of the 9 level models, for the B case, are higher than the A case, regardless of the type of resistance, something similar happens in the demands of local ductility in the columns. The maximum demands for the nominal cases are values of between 4 and 6 in the beams, whereas in the columns the values are less than 2; with the over-resistance effects such that the maximum demands in the beams are from 2 to 3 and the columns behave elastically, for practical purposes. The maximum demands of local ductility in the beams and columns in the 17 level models tend to be higher in the B case, no matter what type of resistance. The maximum value in the beams varies between 3 and 7 for models with nominal resistance and between 1.5 and 3 when over-resistance effects are considered.

4 Non-linear static analysis (Push-over)

The non-linear static analysis, with four different distributions of lateral load, was made with and without the over-resistance effects to establish: 1) The SMD

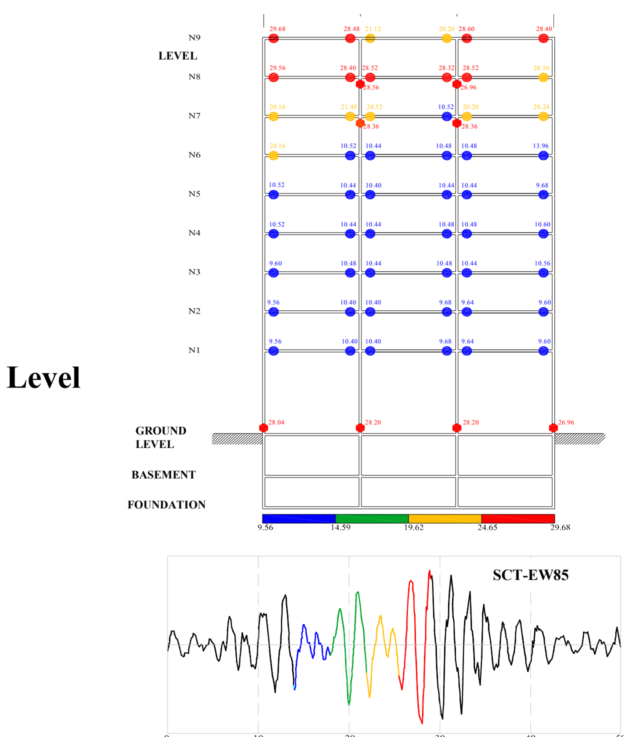


Figure 8: Apparition sequence and global distribution of plastic hinges, 9 level structures, B case with nominal resistances.

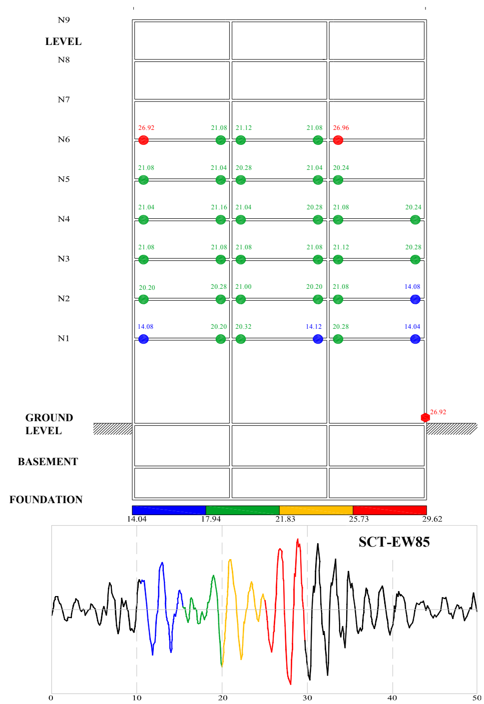


Figure 9: Apparition sequence and global distribution of plastic hinges, 9 levels structure, B case with over-resistances effects.

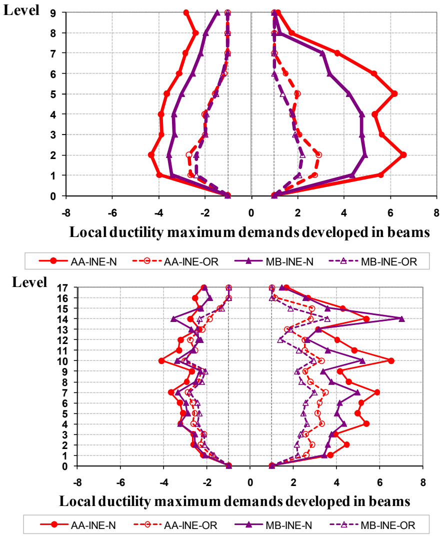


Figure 10: Local ductility maximum demands developed in beams, inelastic step by step analysis with nominal resistances and over-resistances, 9 and 17 level buildings, A and B cases.

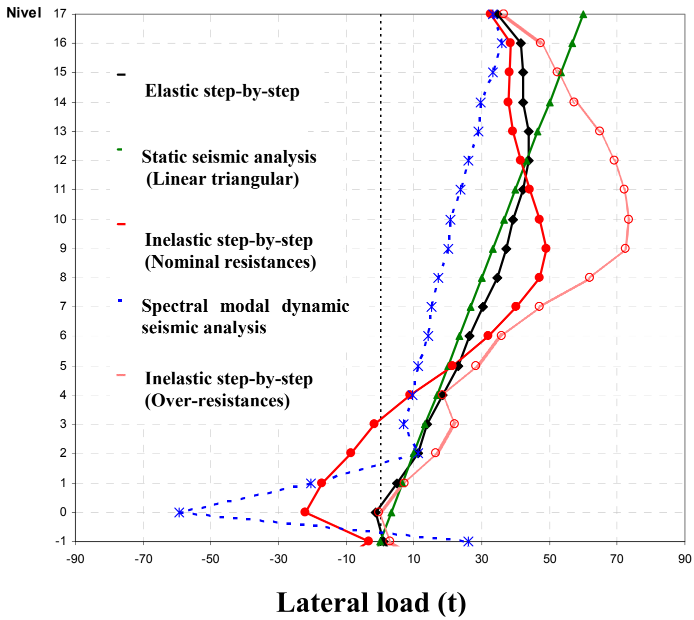


Figure 11: Lateral load distributions for the non-linear static analysis (Push-over), 9 and 17 level buildings, B case.

distribution with forces at the floor level (F_i), determined from the story shear forces (V_i) from spectral modal dynamic analysis, involving all modes of lateral vibration. 2) The elastic step-by-step distribution with forces at floor level (F_i), calculated with the story shear forces (V_i) for a "t_i" time from a step-by-step analysis when the structure is working in the elastic range. 3) The inelastic step-by-step distribution with forces at floor level (F_i), defined with the story shear forces (V_i) for a "t_i" time from a step-by-step analysis when the structure reaches the maximum inelastic roof displacement. 4) The linear triangular distribution from F_i forces as a result of the hypothesis of static seismic analysis (linear). Fig. 11 shows the laterals load distributions for the non-linear static analysis (Push-over) for the 17 level buildings, B case. The results shown are only for the case in which the distributions are obtained from a spectral modal dynamic analysis (SMD), with the participation of all modes of the lateral vibration. The Push-over analyses were made according to the following conditions: a) the maximum demands of the local ductility in the beams are equal to 35, b) the maximum demands of the local ductility in the columns are equal to 20, c) the maximum story drift of the collapse condition is 0.03, according to A Appendix of the CTN-Seismic of RDF-04 Code for structures for ductile concrete frames with Q= 3, d) the mechanism of collapse of the structure is reached.

4.1 Maximum story drifts

The 9 level models with nominal resistance, both cases (A and B), are not limited by the condition of permissible story drift of collapse, but they tend to be governed by the limit condition previously established of the maximum demands of the local ductility of structural elements; the behavior of models with over-resistance is limited by the condition of story drift of collapse of 0.03. In the 17 level models with nominal resistances, A and B cases, the maximum allowed by the demands of local ductility in the columns (20) was reached simultaneously when the allowed maximum story drift is presented.

4.2 Base shear force–roof lateral displacement relationship

Fig. 12 compares the results of the base shear force-roof lateral displacement relationship of the Push-over analyses against relationship of the corresponding inelastic step-by-step analyses, with and without over-resistance effects, for the 9 and 17 level models designed with A Appendix (B case). The results of both types of analysis confirm the lateral stiffness and lateral resistance of each structure, as well as their capacity for energy dissipation when facing the typical seismic effects of Mexico City. In both A and B cases of the 9 level models with nominal resistances, the maximum displacement did not exceed 60 cm due to the regulation of the limit condition previously established for the maximum demands of ductility in columns of 20 in. For 17 level models, the resulting behavior from the Push-over analysis was generally governed by the condition of permissible drift of collapse of 0.03. The influence of over-resistance effects is very important.

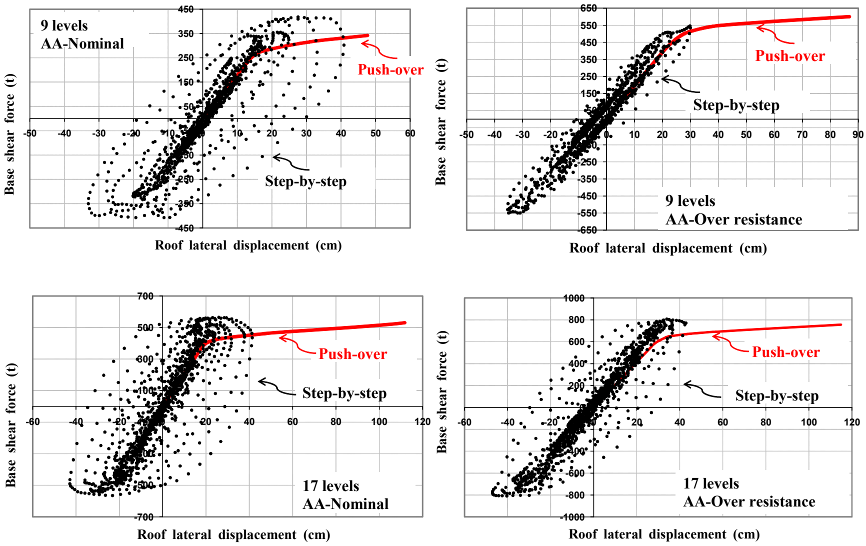


Figure 12: Base shear force–roof lateral displacement relationships comparison, non-linear static analysis (Push-over) and inelastic step-by-step dynamic analysis with nominal resistance and over-resistances, 9 and 17 level structures, B case (A Appendix).

4.3 Global distribution of plastic hinges and maximum demands of local ductility in beams and columns

With the design of A Appendix (B case), the 9 level models present a slightly higher number of plastic hinges and a roof horizontal displacement that is slightly higher. In the case of nominal resistance, the maximum demands of ductility in the column reached the maximum allowed value (20); the maximum demands of local ductility developed in beams are below the predetermined limit of 35. The behavior of the 17 level buildings, A and B cases, are not very different. In a large number of yielded structural elements for the B case and for the A and B cases with nominal resistance, the maximum demands of local ductility are presented in about 20 in columns, whereas beams have maximum values of 25. When the over-resistance effects are considered, the demands of local ductility in beams and columns are not parameters that govern the behavior.

5 Conclusions

Buildings designed with the RDF-04 Code show satisfactory performance, with enough reserve of resistance, to avoid brittle failure. Significant variations between the designs of the A (Main Body) and B (A Appendix) cases are not found. With the help of the over-resistance effects, the maximum responses tended to be lower. The tendency of plastic hinges from step-by-step and non-linear static (Push-over) analyses show, in general, a strong column-weak beam behavior according to the current design philosophy, which ensures a ductile behavior. The analysis results show consistency regarding responses of dynamic analysis in time history.

Reference

[1] Gaceta Oficial del Gobierno del Distrito Federal (2004), “Reglamento de Construcciones de la ciudad de México y Normas Técnicas Complementarias para Diseño por Sismo, y para Diseño y Construcción de Estructuras de Concreto”.