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INELASTIC PERFORMANCE OF A RC WALL BUILDING COLLAPSED IN THE 27 FEBRUARY 2010 CHILE EARTHQUAKE

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Abstract

The seismic structural performance of a real case concrete wall building (Building 1) that collapsed during the 27 February 2010 Chile earthquake is evaluated. Similarly, the response of two other concrete frame buildings (Buildings 2 and 3) designed and analyzed according to the current Chilean and Mexican Regulations, respectively, and with a geometric structuring similar to the real case structure is studied. The objectives of this work are: evaluate the seismic performance of the three buildings and compare their elastic and inelastic (nominal and over-strength cases) seismic responses calculated with two accelerograms, CCH-NS (Chile 2010) and SCT-EW (Mexico 1985). Due to the fact that the flexural strength in beams and the flexural-compression strength in columns and walls in the ground floor level were exceeded by the demand induced from the seismic load, plastic hinges were developed at the ends of the structural elements. It was verified that the shear strengths of the selected structural elements were higher than the seismic demands.

Keywords: seismic structural performance, inelastic seismic response, modal spectral dynamic seismic analysis

1. Introduction

In the last decades, severe earthquakes have been generated in the Pacific Ring of Fire, which have induced important human and material loses. Such were the cases of the September 16, 1985 earthquake in Mexico and, more recently, the February 27, 2010 earthquake in Chile. The epicenter of Chile earthquake was located at 43 and 150 km away from the southwest and northeast of the cities of Cobquecura and Concepcion, respectively, with a focal depth of 30.1 km and a seismic moment magnitude of M_w=8.8. On the other hand, the epicenter of Mexico earthquake was located in front of the coastline of Michoacan at a distance of 350 km from Mexico City with a focal depth of 15 km and a seismic moment magnitude of M_w= 8.1.

Although the mentioned earthquakes induced several human and material loses, it was observed that the previously adopted advances in seismic engineering can be significant and favorable as long as the theory, design philosophy and reinforce details in the buildings are properly applied.

The structural performance of three buildings is studied in this paper: Building 1 is a real case of a concrete wall building that collapsed during the 2010 Chile earthquake (Figs. 1 and 2), and Buildings 2 and 3 (Fig. 3) are concrete frame buildings with a geometric structuring similar to Building 1, designed according to the current in-force Construction Regulations in Chile (NCH-430, NCH-433) and Mexico (RCDF-04).

The main objectives of this work are the following: 1) Evaluate the seismic performance of the three buildings; 2) Compare the elastic and inelastic (nominal strength and over-strength cases) seismic responses of the buildings considering two accelerograms (CCH-NS (Chile 2010) and SCT-EW (Mexico 1985)) (Fig. 4) for each model.

Fig. 1 – Building 1 after the February 27, 2010 earthquake in Chile

2. Analysis and design criteria

Building 2 was designed and analyzed in accordance with the guidelines for “Seismic Design of Buildings” (NCH-433) and “Reinforced Concrete – Design and Calculus Requirements” (NCH-430) [1] contained in the Chilean Regulations, which are mainly based on ACI-08 Standards. The corresponding design and analyses of Building 3 were performed according to the “Complementary Technical Norms for Seismic Design” (NTC-Seismic) and “Complementary Technical Norms for Design and Construction of Concrete Structures” (NTC-Concrete) of the “Construction Code for the Federal District” (RCDF-04) [2] of the Mexican Regulations. The design and three-dimensional modal spectral dynamic seismic analyses of the concrete frame buildings were computed with ETABS [3] (see Fig. 5). Gravity loads, P-D effects, and three degrees of freedom for each level (two in translation and one in torsion) were considered.

Sizing of Buildings 2 and 3 was made in order to satisfy the service state limit, which consists in a maximum drift of 0.002, as it is defined in the Chilean Regulation (NCH-433) for any structure, and in Appendix

A of the Mexican Regulation (NTC-Seismic) for ductile concrete frame buildings where there are no structural elements unable to resist significant deformations.

In order to calculate the design inelastic seismic spectra and compute the corresponding analyses, Buildings 1 and 2 were considered to be located in soil site III of the seismic zone III in Chile.

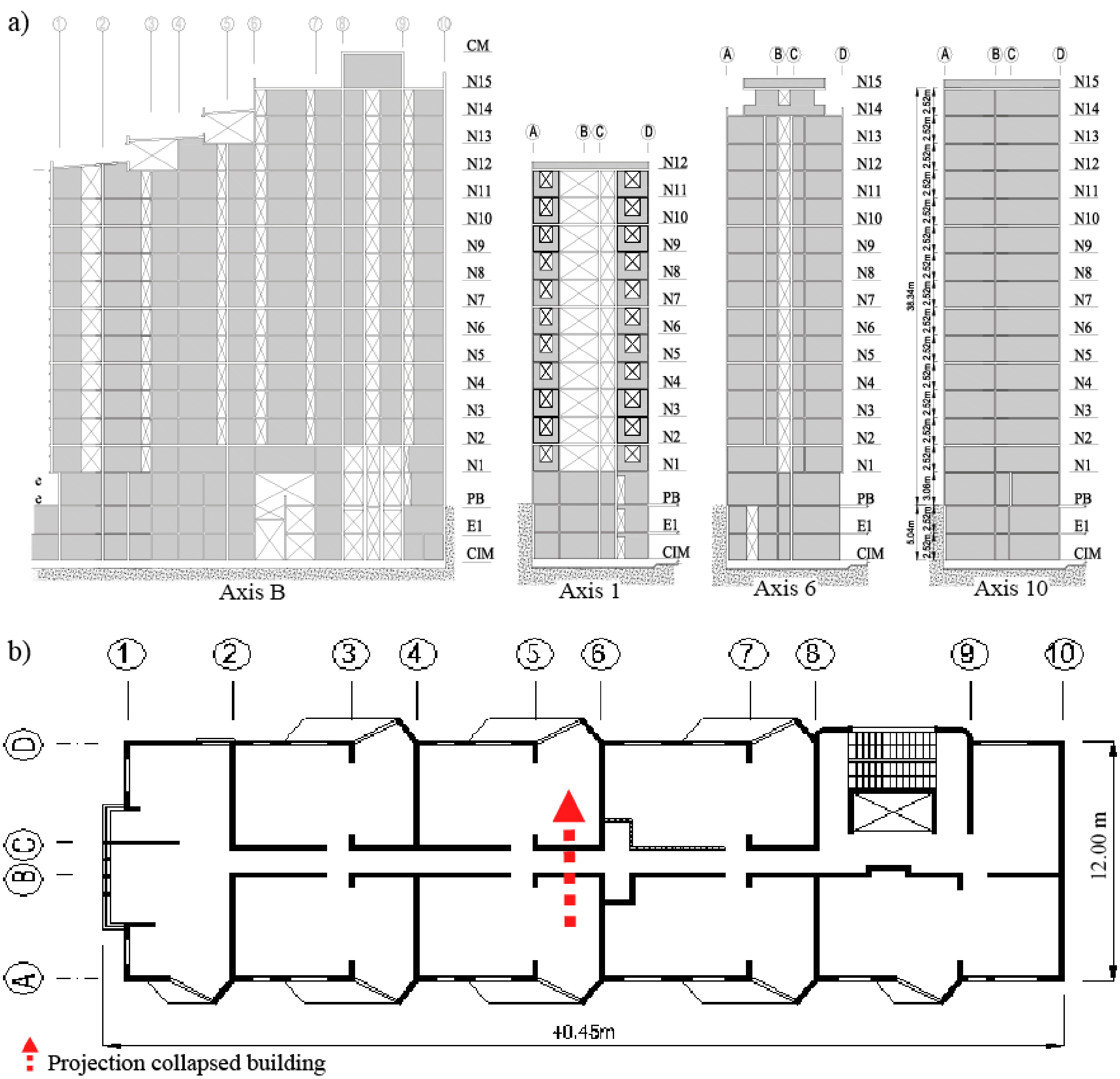

Fig. 2 – a) Elevation and b) plan views of Building 1

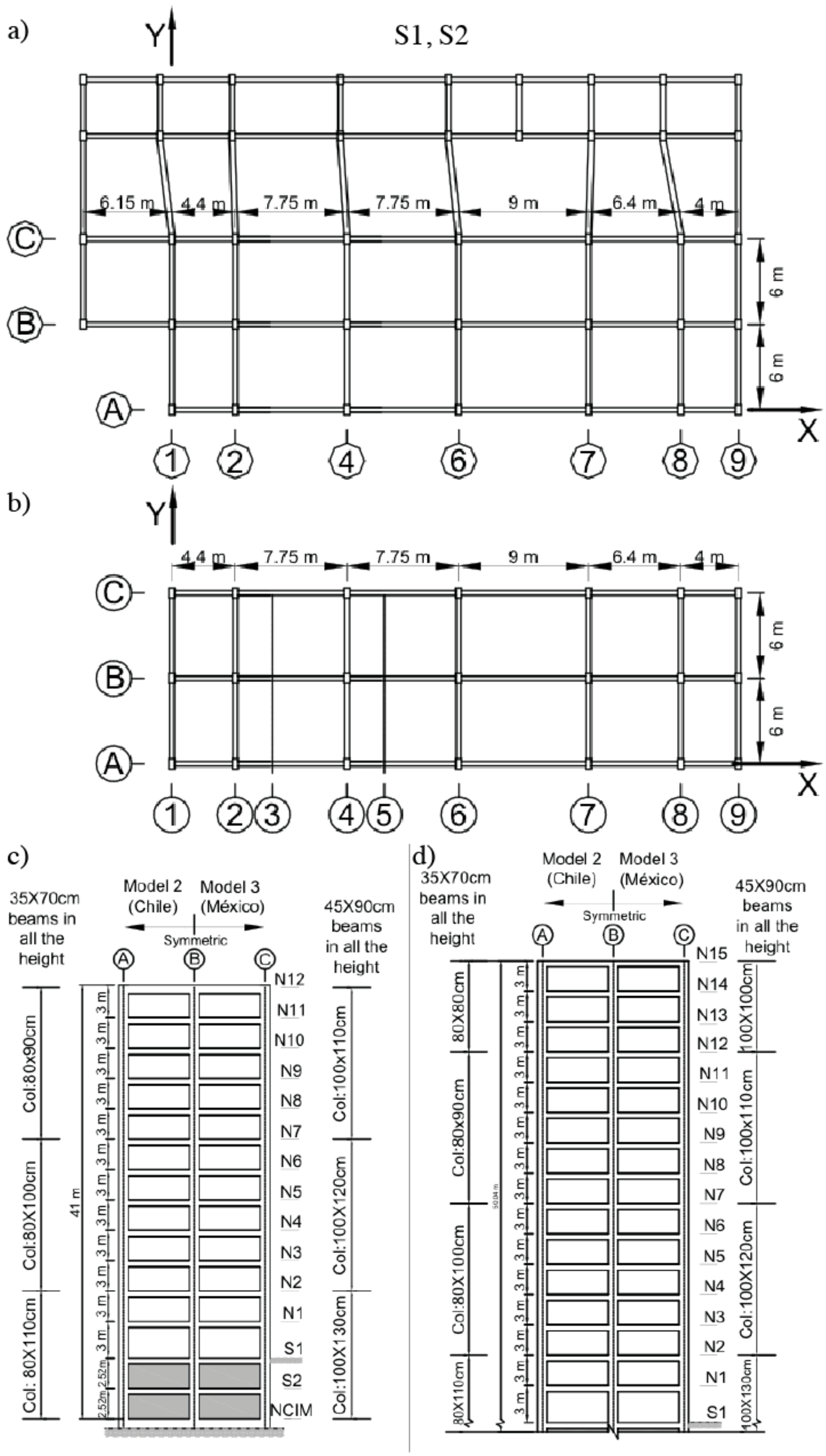

Fig. 3 – Buildings 2 and 3: a) Plan views of parking levels (S1 and S2); b) plan type; c) elevation views of axis 1; and d) elevation views of axes 6 and 9

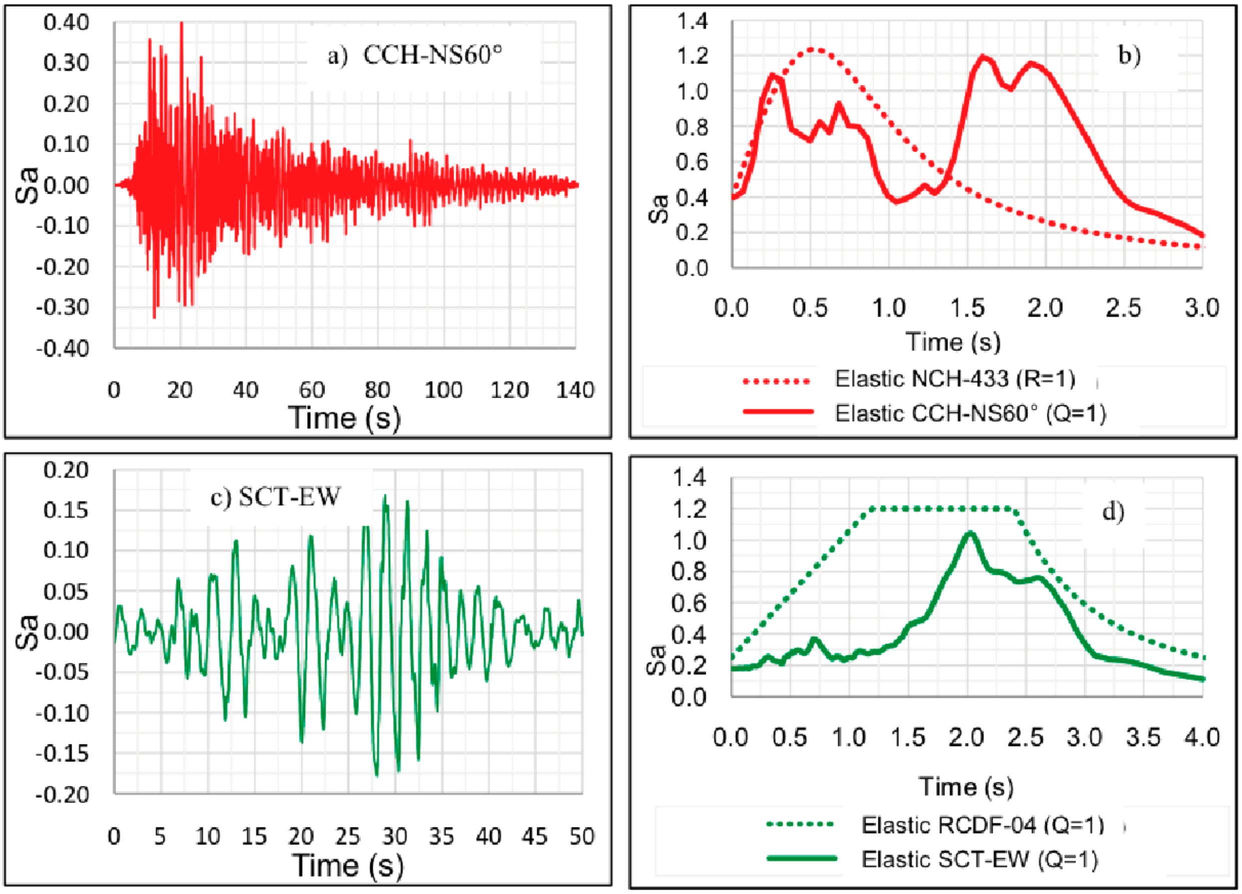

Fig. 4 –February 2010 Chile earthquake (CCH-NS): a) Record of the North-South component; and b) elastic response spectra compared with the elastic design spectra of the seismic zone III of Chile for soil type III. September 1985 Mexico earthquake (SCT-EW): c) Accelerogram of the East-West component; and d) elastic response spectra compared with the RCDF-04 elastic design spectra calculated for soils with dominant vibration period of T_s= 2 seconds

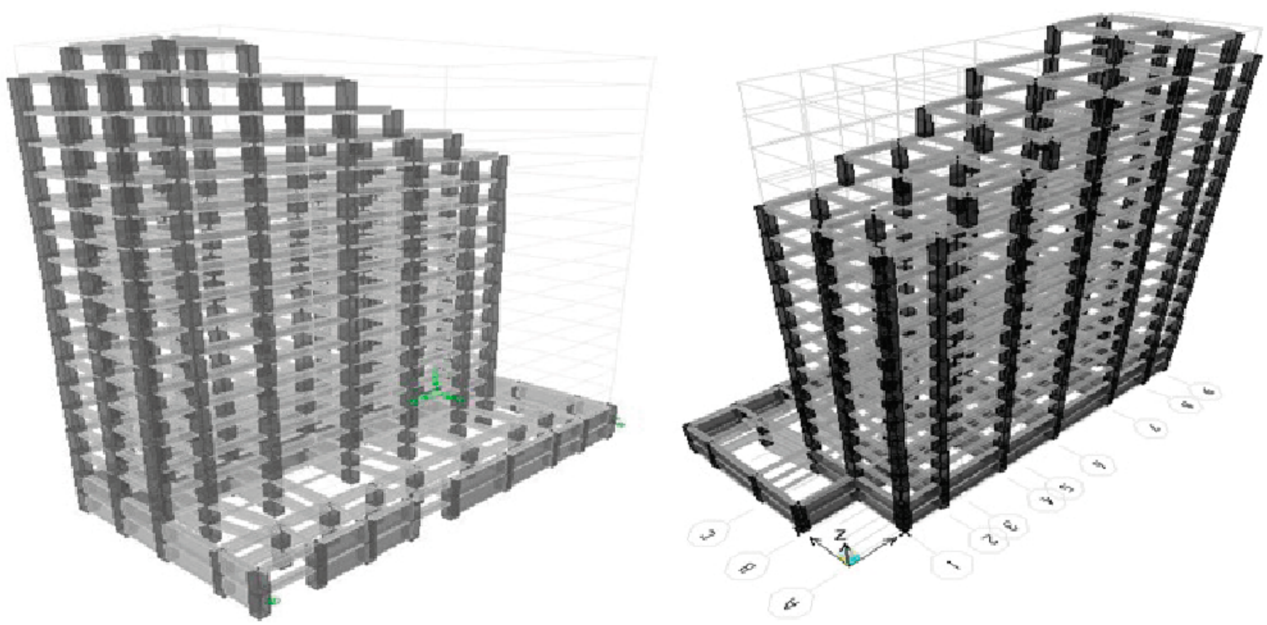

Fig. 5 – Three-dimensional views of Buildings 2 and 3

3. Elastic response and design

Fig. 6 and Table 1 show the modal forms and fundamental vibration periods of Buildings 1, 2 and 3, respectively. Building 1 presents the lowest vibration period because it is a concrete wall structure. On the other hand, Building 2 presents the highest vibration period because its structure is based on frames and its structural elements dimensions are smaller than the ones of Building 3. For the first vibration mode, Building 1 presents a pure translation behavior in its damaged transversal direction.

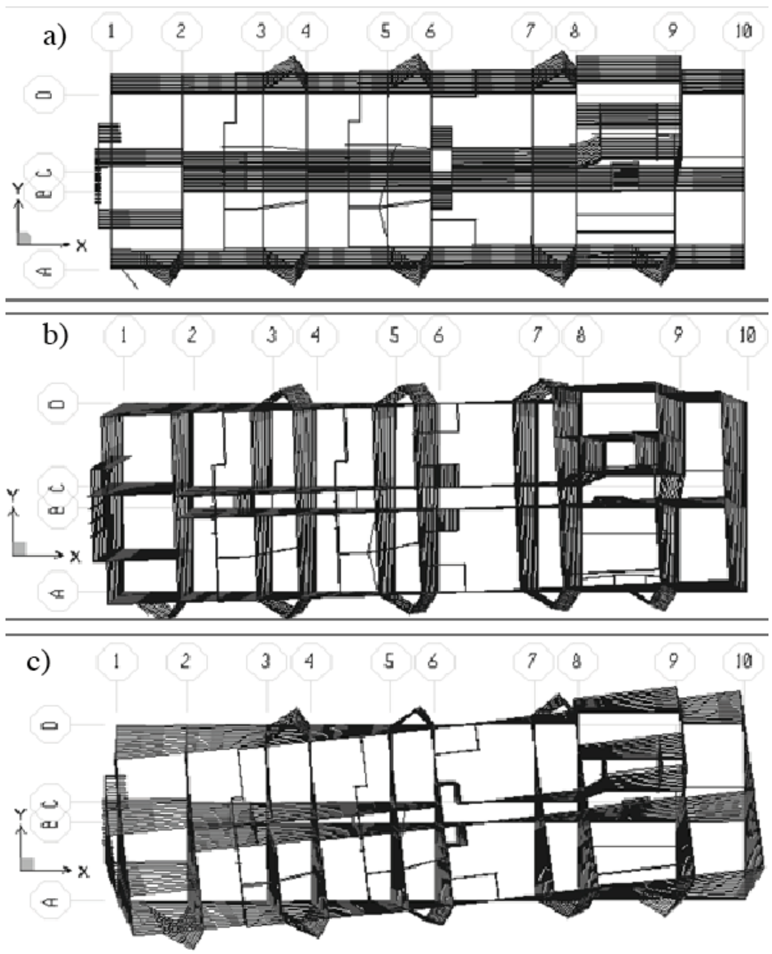

Fig. 5 – Modal deformations plan view for a) first, b) second, and c) third vibration modes of Buildings 1, 2 and 3

Table 1. Fundamental vibration periods (seconds) of Buildings 1, 2 and 3

Building	**T_{1X} (Longitudinal direction)**	**T_{1Y} (Transverse direction)**	**$T_{1\theta}$ (Torsion)**
1	0.476	0.623	0.463
2	1.260	1.335	1.012
3	0.864	0.950	0.710

Figs. 7 and 8 show the seismic responses of maximum story drifts and maximum story shear forces of the three buildings, respectively. The responses correspond to the transversal direction of the buildings and were calculated through spectral modal dynamic analyses.

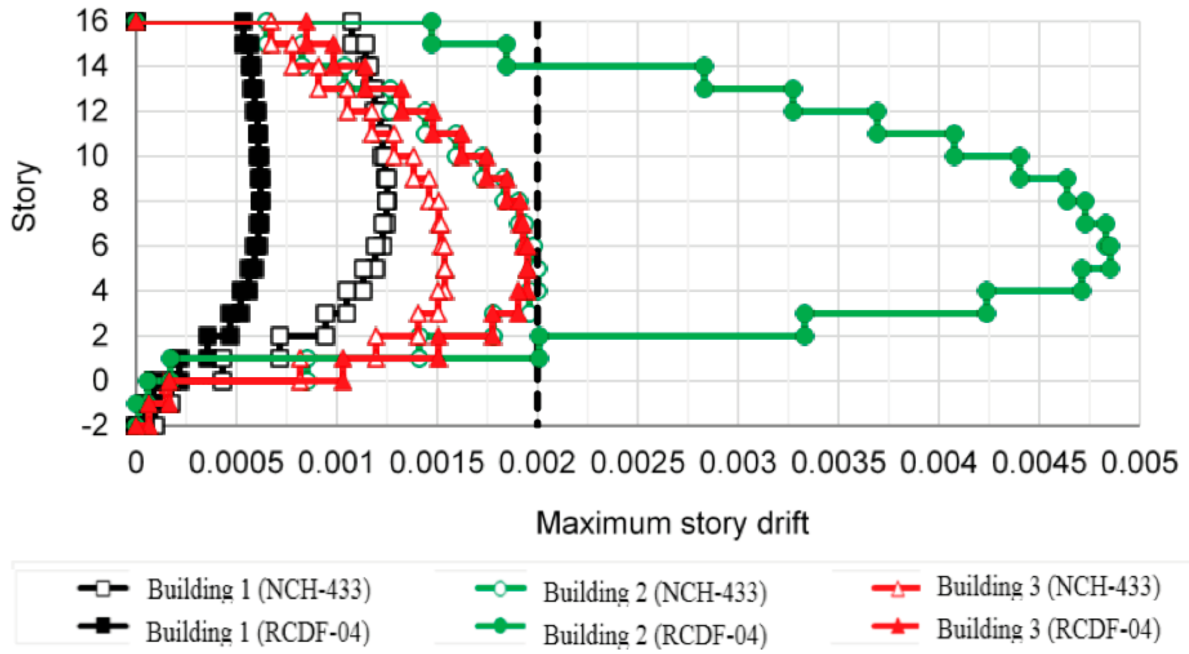

Fig 6. – Maximum story drifts calculated with the inelastic design spectra of the NCH-433 (soil type III) and the Appendix A of the Seismic Regulations of the RCDF-04 (T_s=2 seconds and Q= 3) in the transversal direction of Buildings 1, 2 and 3

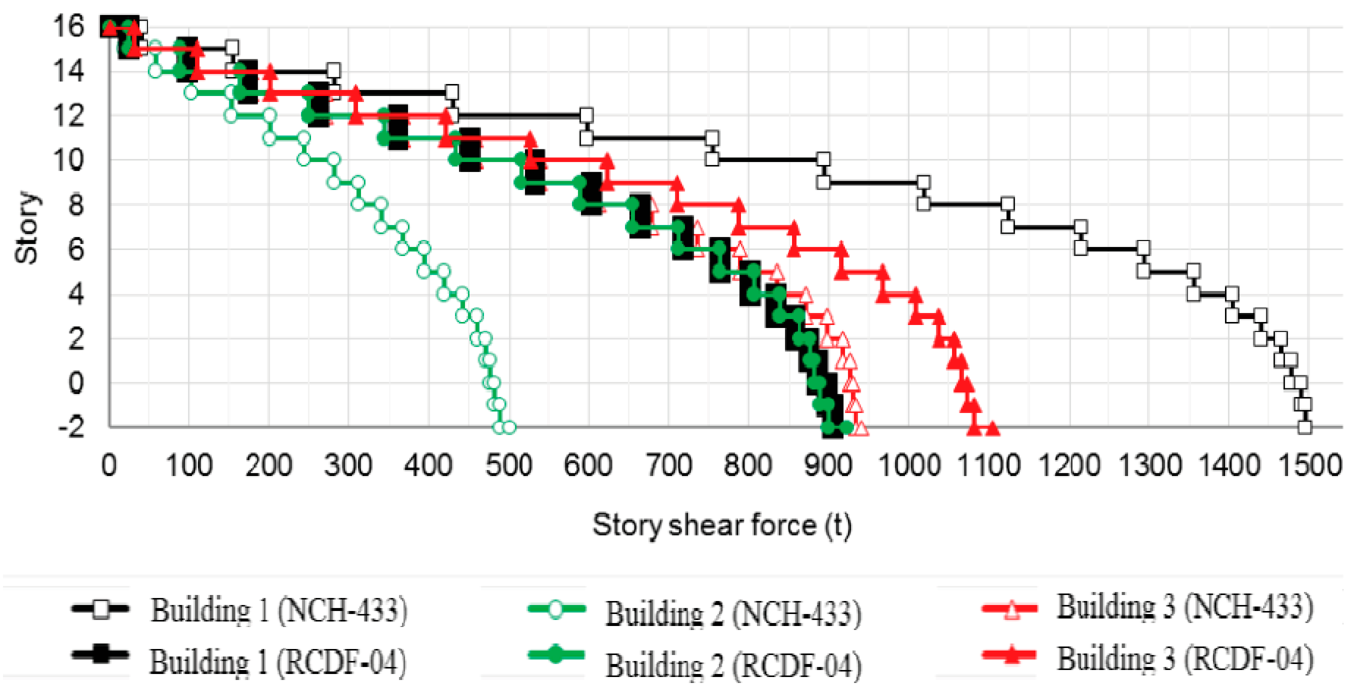

Fig. 7 – Maximum story shear forces calculated through modal spectral dynamic seismic analyses with the inelastic design spectra of the NCH-433 for buildings built in soil type III, and with the Appendix A of the NTC-Seismic (RCDF-04) for a dominant soil period T_s= 2 seconds and seismic behavior factor Q= 3, transversal direction of Buildings 1, 2 and 3

With the aim to compare the structural performance of the buildings, the seismic responses were calculated with the inelastic spectra established in the NCH-433 and NTC-Seismic regulations. According to the values of Q' and R that correspond to the fundamental vibration period of the models, the maximum story displacements and story drifts, service condition, that were calculated with the NTC-Seismic are multiplied by Q'R/7; where Q´ is a factor depending of Q and the natural period, and R is the over-strength factor equal to 2, approximately.

As mentioned above, Building 1 is an existing real case structure that collapsed as a consequence of the 2010 Chile earthquake, and therefore its design was not performed. In the cases of Buildings 2 and 3, axes 1, 6 and 9 were specifically designed for the purposes of this work.

4. Inelastic responses

The three-dimensional elastic models of each structure were taken as a reference in order to calibrate periods, lateral displacements and mechanical elements induced by seismic and gravity loads of the two-dimensional models made based on axes 1, 6 and 9 of each building with DRAIN-2DX [4].

The elastic and inelastic seismic responses of the chosen axes for each building were calculated by means of step-by-step time history analyses. Two accelerograms were considered: 1) 60° North-South component of the February 2010 Chile earthquake, which was registered in Concepcion, Chile (CCH-NS60°), and 2) East-West component of the September 1985 Mexico earthquake, which was registered in Mexico City (SCT-EW 1985). The seismic responses were obtained for the elastic (high strengths) and inelastic (nominal strength-NS and over-strength-OS) cases. A 30% increment in the concrete compression strength (f'_c) and a 25% increment in the steel stress yield (f_y) were considered for the over-strength cases.

Fig. 9 shows the inelastic design spectra with the specific location of the fundamental vibration periods of Buildings 1, 2 and 3, compared with the CCH-NS60° and SCT-EW elastic response spectra.

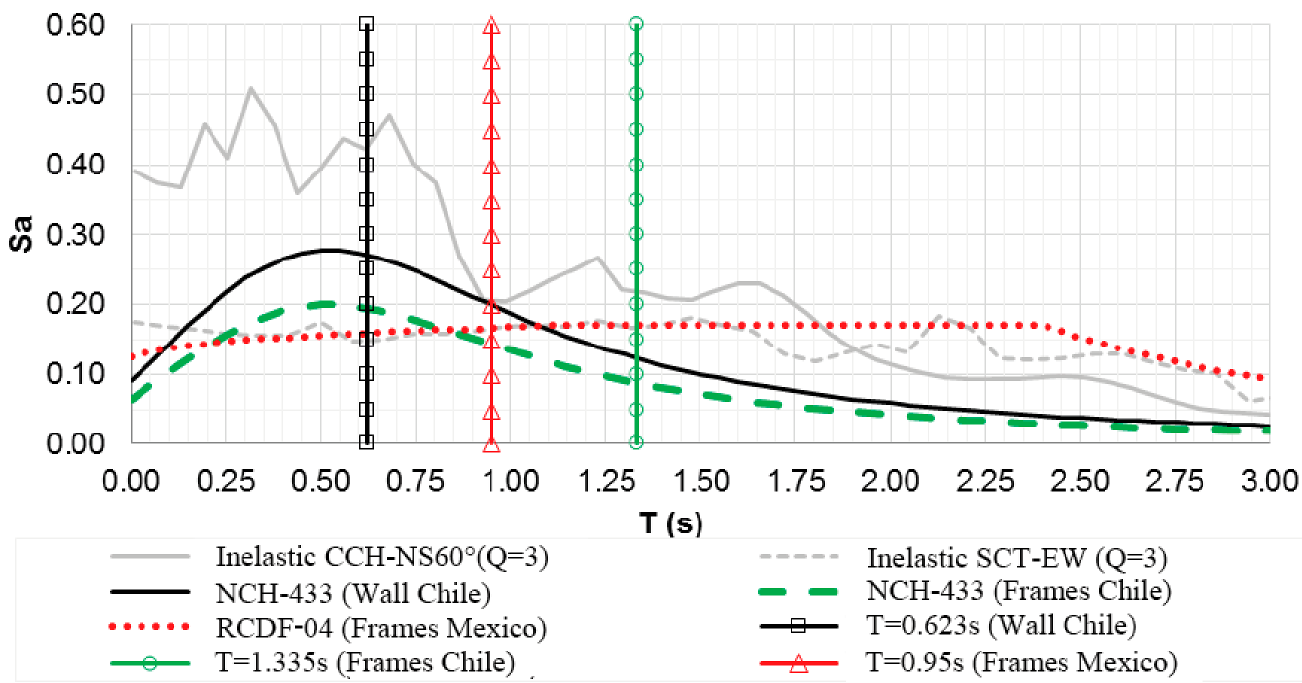

Fig. 8 – Comparison of the elastic response spectra and the inelastic design spectra for the CCH-NS60° and SCT-EW accelerograms, with location of fundamental vibration periods of Buildings 1 (wall-Chile), 2 (frames-Chile) and 3 (frames-Mexico)

Fig. 10 shows the global responses (story drifts) of axis 6 of each building for the elastic and inelastic cases (with nominal and over-strengths), calculated by means of the step-by-step time-history method under the CCH-NS60° and SCT-EW accelerograms. The story shear forces were calculated and it was verified that the shear strength is higher than the demands in each element (walls, beams and columns).

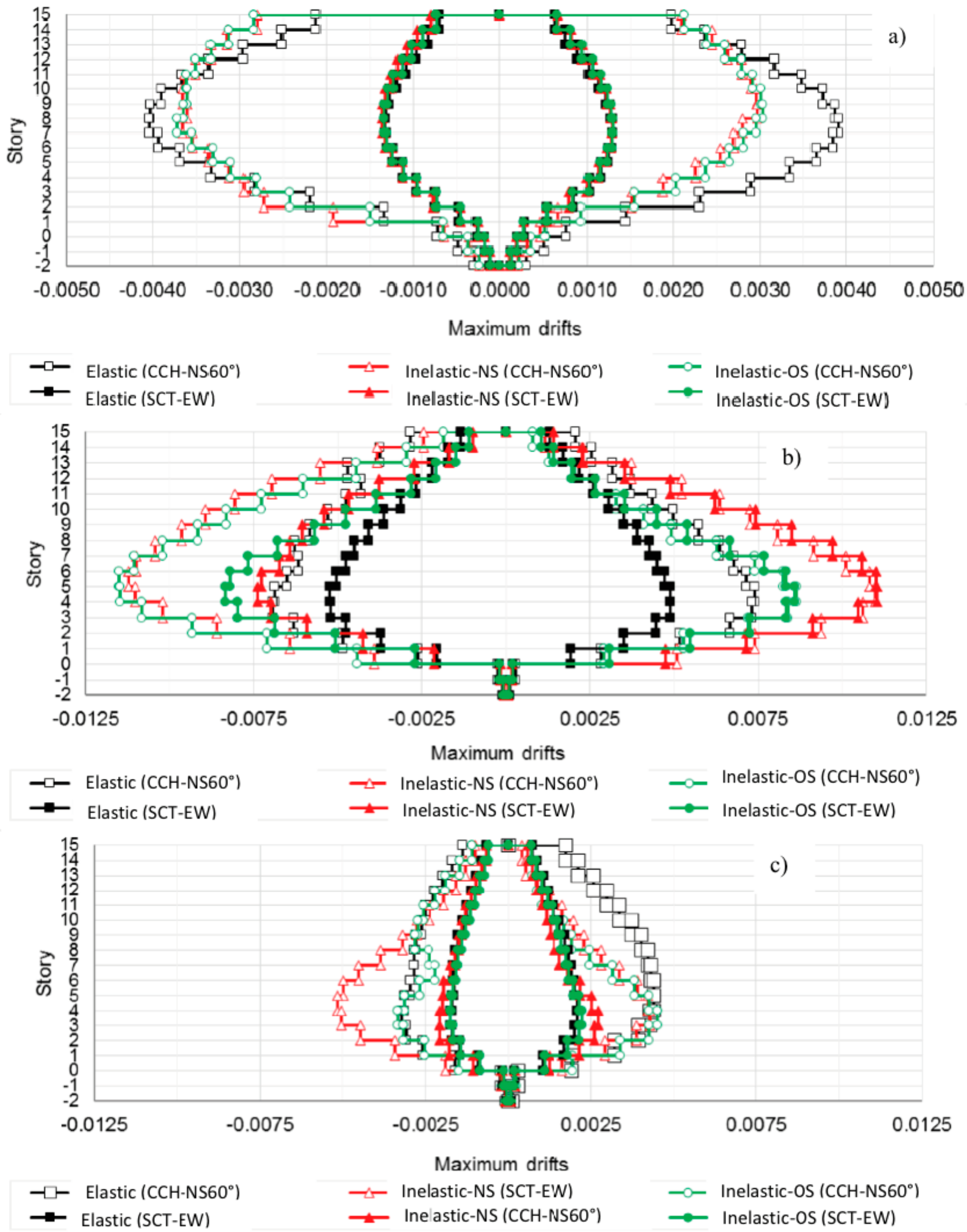

Fig. 9 – Axis 6 drifts for the elastic and inelastic (with nominal and over-strengths) cases for buildings a) 1, b) 2, and c) 3, by means of the step-by-step time-history analyses under the CCH-NS 60° and SCT-EW accelerograms

The combined flexural moment–axial load (M-P) local responses of the inferior end of the wall of axis 1 (Fig. 11.a), 6 (Fig. 11.b) and 9 (Fig. 11.c) of Building 1, were calculated through the step-by-step time-history seismic analyses and the CCH-NS 60° accelerogram for the inelastic case with nominal strength.

Fig. 12 shows the relations of the base shear force– lateral roof displacement of axis 6 of Building 1 for the elastic and inelastic (nominal and over-strength) cases obtained with the step-by-step dynamic seismic analyses under the CCH-NS60° and SCT-EW records.

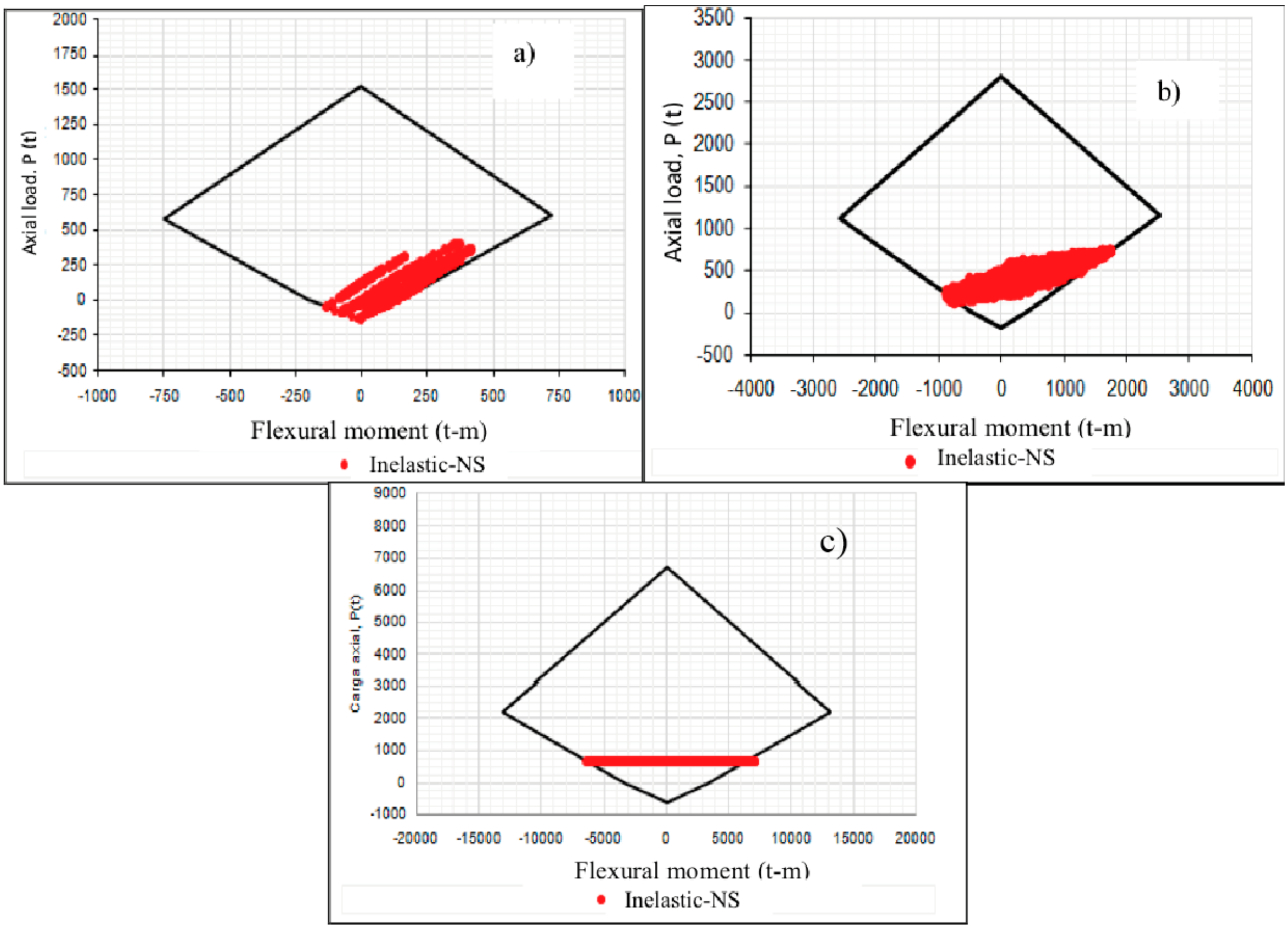

Fig. 10 – Combined flexural moment–axial load (M-P) response of the inferior end of the wall of axis 1 (a), 6 (b) and 9 (c) of Building 1, calculated through step-by-step time-history seismic analyses under the CCH-NS 60° accelerogram for the inelastic cases with nominal strengths (NS)

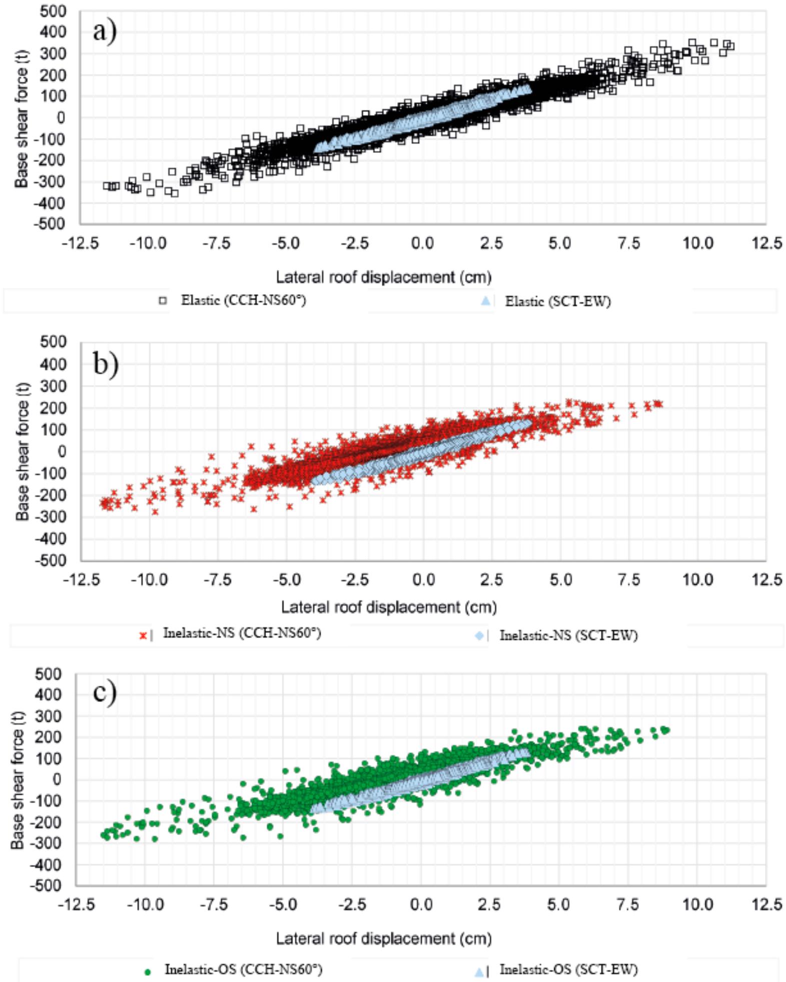

Fig. 11 – Relations of the base shear force- lateral roof displacement of axis 6 of Building 1 calculated with the accelerograms CCH-NS 60° and SCT-EW for the a) elastic and inelastic cases with b) nominal strengths and c) over-strengths effects

5. Conclusions

In general, for Building 1, a good consistency exists between the calculated behavior and the observed damage level after the earthquake. The matching between the fundamental vibration period of the structures and the dominant period of the ground in which the structures are built, like in the case of Building 1, should be avoided as far as possible. The short direction and levels with maximum damages match with the direction and stories with maximum deformations obtained from the analysis. The flexural-compression strength of the walls was low and the shear strength was bigger than the demands induced by the CCH-NS60° record. The ground floor boundary walls of axes 1, 6 and 9 were the most demanded zones because of their high flexural compression and flexural tension effects. In the cases of over-strengths, it can be concluded that it had a minor incursion in the inelastic range performance when compared with the inelastic performance presented in the cases of nominal strengths.

From the results observed after analyzing the ground floor walls from the three studied axis of Building 1, the following conclusions were obtained: a) the flexural-compression strength of the walls was low due to the development of plastic hinges when the two-dimensional model was analyzed with the accelerogram CCH-NS60°; b) the shear strength was higher than the demands induced with the CCH-NS60° accelerogram; 3) the ground floor external boundary wall of axes 1, 6 and 9 were the most demanded zones of each axis, because they had high flexural compression and flexural tension loads induced by the CCH-NS60° accelerogram. The values of the local ductility calculated for the ground floor walls of axes 1, 6 and 9 of Building 1 are acceptable for columns, however, a wall can only be capable to develop those values of local ductility if the transversal reinforcement detail in the boundary walls is adequate, like those described in the NTC-Concrete. Axis 9 of Building 1 resulted to be the axis that presented the highest stresses when it was analyzed with the step-by-step time-history direct integration method under the CCH-NS60° record.

Studying Buildings 2 and 3, the following conclusions can be are drawn: a) the transversal and longitudinal steel reinforcement details established in the NTC-Concrete provide an adequate seismic performance to the buildings; b) the fault mechanism that dominated was strong column–weak beam; c) the analyzed model presented plastic hinge rotations first in the majority of ends of beams and at last in the base of the columns of the ground floor; d) the transversal steel reinforcement provided enough shear strength to avoid a fragile fault by allowing the development of flexural faults; e) the design dimensions and strengths of Building 2 according to the NCH-433 Chilean regulation was adequate, however, it must be noticed that the seismic demand was low since the fundamental period of the building is located between the dominant peaks of the response spectra. In contrast, if the fundamental period of the structure was located at the second peak of the response spectra, like in the case of the axis 9, the pseudo acceleration design spectra would be under the real accelerations, which would be submitted to the structure causing severe damage. The dimensions of the design structural elements of Building 3 resulted to be large due to the need of an important lateral stiffness in order to satisfy the service limit state of a maximum drift of 0.002, as it is established in the Appendix A of the NTC-Seismic. However, it can be concluded that a concrete wall structure would be more efficient.

After studying the performance of Building 1 when submitted to the SCT-EW record, it can be concluded that: a) the structure composed of concrete structural walls provide enough lateral stiffness to easily accomplish the service and collapse limit states; b) the short period of Building 1 is located in the ascendant part of the design spectra of Appendix A of the NTC-Seismic and in the same part of the response spectra, which means that the fundamental vibration period is far of the interval of dominant ground periods.

In the cases of design of concrete structures in zones of high seismic intensity, the steel reinforcement detail of the structural elements is extremely important. The longitudinal steel reinforcement has to be anchored in order to allow that the steel develops its yield stress without sliding, and the transversal reinforcement steel should be the necessary to adequately confine the concrete core of the structural elements in order to avoid undesirable faults like lateral buckling of the longitudinal steel reinforcement in the presence of high compression axial loads.

In walls governed by flexural behavior, it is expected that plastic hinges develop in the base of the wall when the inelastic behavior is reached, like in the results obtained in this paper. Nevertheless, in the design of walls, the longitudinal and transversal steel detail of the borders of the walls has to be considered with special attention, as suggested in the NTC-Concrete.

6. References

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