Behavior of Reinforced Concrete Buildings in Viña del Mar: Lessons of February 27th 2010 Earthquake

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SUMMARY:
Due to the last great Chilean Earthquake, February 27th, 2010 (Mw8.8) the majority of R/C buildings did not have damage or suffered minor damage. However, in a few medium height buildings founded on deep soft soils, severe and brittle damage in walls in the first floor due to shear stress or high flexural-compression stress on boundaries of T and L walls was observed. A registry of observed failure modes in medium height buildings located in Viña del Mar (with and without severe damage) was developed, verification of designs and nonlinear analysis were done using the acceleration record obtained close to the buildings on similar soil conditions. A basic characterization of the elastic dynamic soil properties was done from measured micro tremors using Nakamura´s method. Results of those analyses show that design decisions and push over analysis can explain either damage or non damage in each building: an important lesson of this study is that in all cases a linear and brittle behavior was obtained, so the presence of damage depends mainly on the amount of displacement demands imposed by the earthquake, since the predominant brittle failure occurs at relatively low displacements, a great risk for similar buildings that behaved well in this earthquake exist in the future. Results show that elastic fundamental vibration periods of analyzed buildings are slightly lower than the soil´s vibration period, so damage occurs due to a building resonance response to the ground motion. It can be concluded that the risk can be reduced with a better characterization of the soil response.

Keywords: Compression, displacement, shear, vibration, period, failure.

1. INTRODUCTION

After the March 3rd 1985 Earthquake, a satisfactory behaviour of reinforced concrete buildings was observed. The structures were built with high density of structural walls, with a wall area to the floor area ratio of around 6%.

Due to this good behavior, the improvement of concrete quality and expanding research into the behavior on structural walls, the structural system used in R/C buildings was changed when engineering practices started to decrease the wall thickness, maintaining the amount of walls but increasing the building height. Consequences of this practice became evident after the February 27th, 2010 Chilean Earthquake with a magnitude of Mw8.8. The epicenter was located under the ocean off the coast of Curanipe and Cobquecura, 150 km northwest of Concepcion at a depth of 47.4 km. Many buildings founded on deep soft soils had brittle failures; contrary to what was implicitly incorporated in the design reduction factor were there should be a desired ductile behavior.

Displacement demands were not well evaluated in the Chilean seismic design code, limit to lateral drift obtain from the analysis done with reduced forces, and displacement spectra derived from design elastic acceleration spectra underestimate actual lateral displacement imposed by the design earthquake. Important changes were introduced to seismic code designs after the February 27th, 2010 Chilean Earthquake producing changes in new structural systems.

2. TYPICAL CHILEAN BUILDINGS

Typical Chilean buildings, generally of medium height, have a structural system formed by a great density of walls (see Figure 2.1). In recent years wall thickness was reduced due to the use of modern seismic code designs allowing greater shear forces in walls than those used in the past.

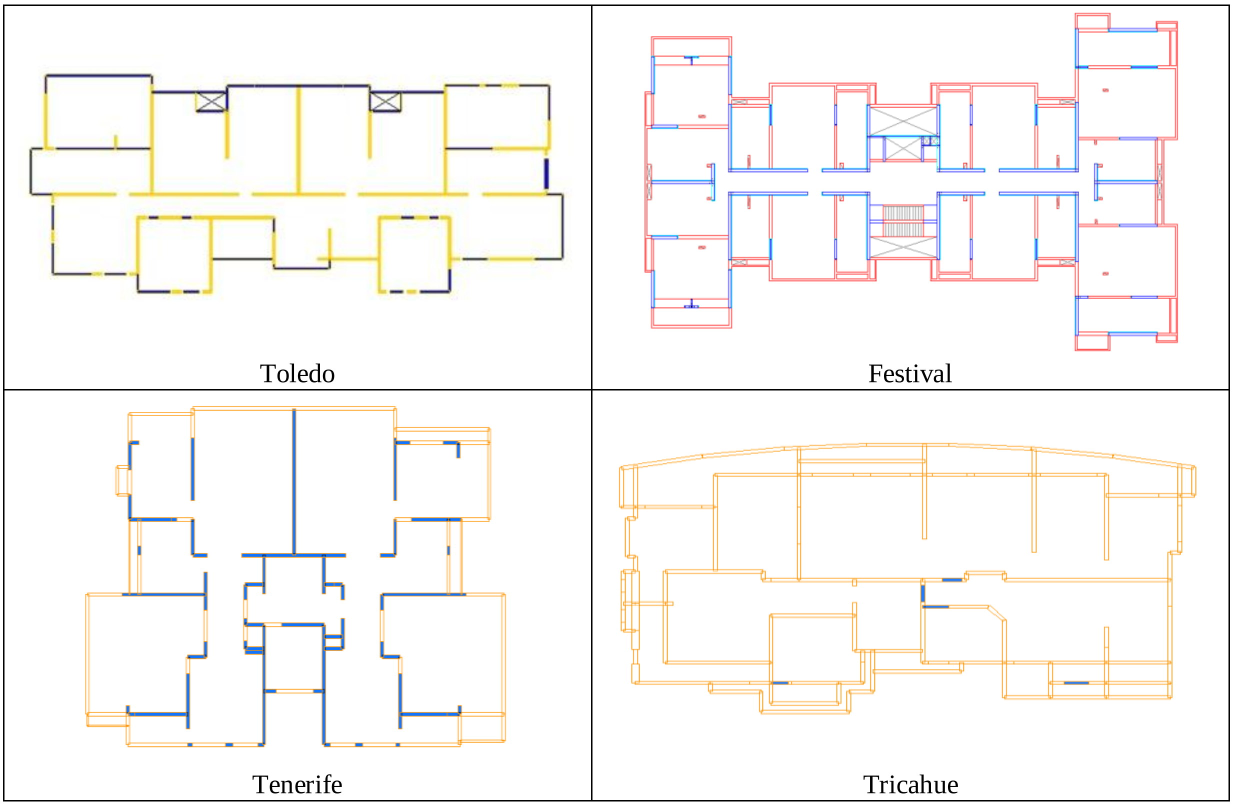


Figure 2.1. Typical building floor plan.

3. DENSITY INDEX

A wall density index, calculated as the wall area in the first floor to the floor area ratio, in each main direction, has been used as an indicator to evaluate the structural characteristics of a reinforced concrete building.

Although this value has remained relatively constant since 1985, when building performance during the earthquake was better than now, the height and weight of the buildings has grown and as a result, an equal density of walls produces higher demands of axial load and shear.

The maximum lateral displacement of the roof depends on the depth of the neutral axis at a critical section in a wall, which is directly related to the axial load, the geometry of the section and the amount of reinforcement of the walls. The walls with T and L sections are especially vulnerable to brittle failures produced by compression in the web due to the steel in tension in the opposite fiber, when large lateral displacements occur.

Nevertheless, it becomes evident that wall design should follow the principles of design by capacity to obtain an individual ductile behavior with the purpose of guarantying a satisfactory global behavior when buildings are subjected to large lateral displacements. Capacity design rules have not been applied, in general, in Chile since ACI318 dispositions have been only partially applied, allowing the generation of walls vulnerable to a shear failure when they are subjected to large lateral displacements, such as it can be observed in the deep soft ground of downtown in Viña del Mar.

4. RESONANCE

According to the measurements of micro tremors of the ground where the tree chosen buildings in this study are, Tenerife, Festival and Coral buildings, the ground has a fundamental period of approximately 1 second, see Figures 4.1 to 4.6. The medium height structures analyzed have a fundamental period of 0.6-0.7 seconds, calculated using gross sections, which increases to nearly 1 second when cracks in concrete appear. This implies that the structure could resonate with the vibrations of the ground, maximizing its response displacement at roof, when the structure respond with a frequency nearly to f=1/T (Hz). The response and the design spectrum (Figure 4.7) show the increase of displacement, which is achieved at the beginning of the resonance between the building and the ground, approximately at a 1 second period.

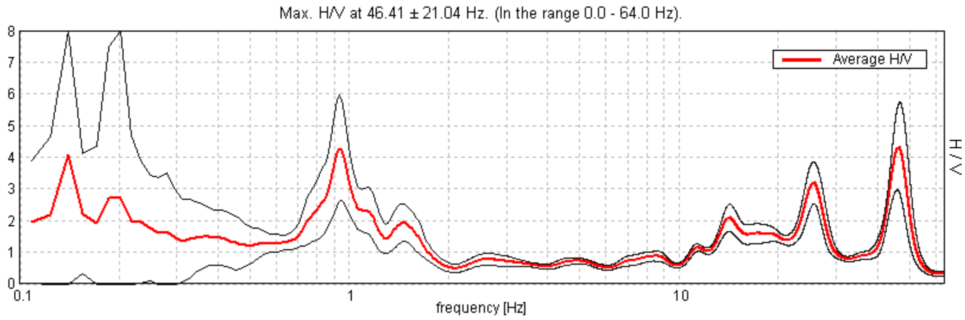


Figure 4.1. Vibration frequency of the Tenerife Building on the first floor.

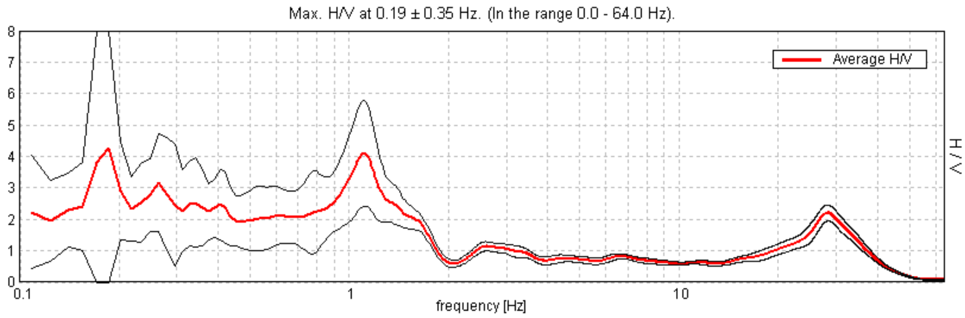


Figure 4.2. Vibration frequency of the ground near the Tenerife Building

The same phenomena occurred with the Festival Building, Figure 4.3 and 4.4.

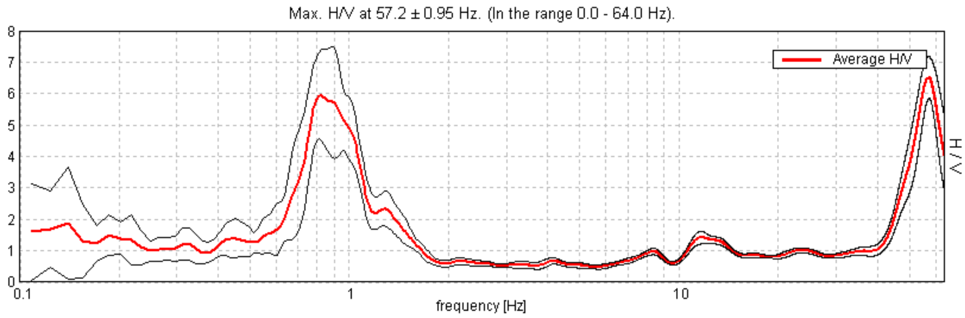


Figure 4.3. Vibration frequency of the Festival Building on the first floor

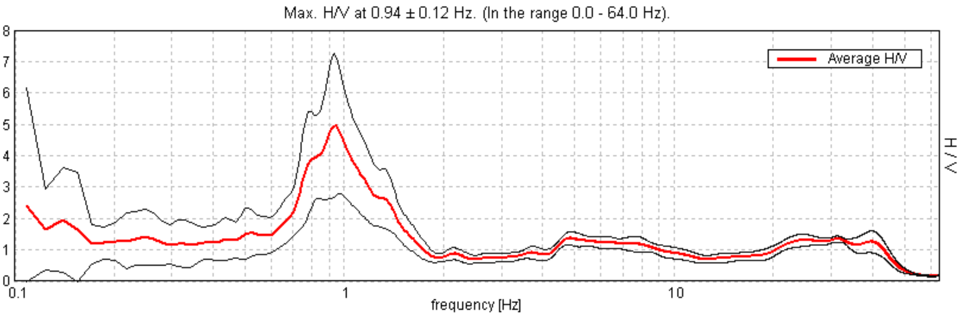


Figure 4.4. Vibration frequency of the ground near the Festival Building

Coral Building, close to the other two buildings analyzed, suffered a similar phenomena.

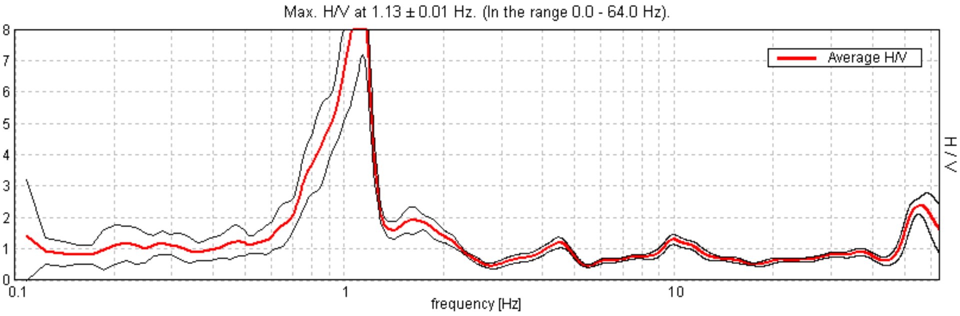


Figure 4.5. Vibration frequency of the Coral Building at the first underground level

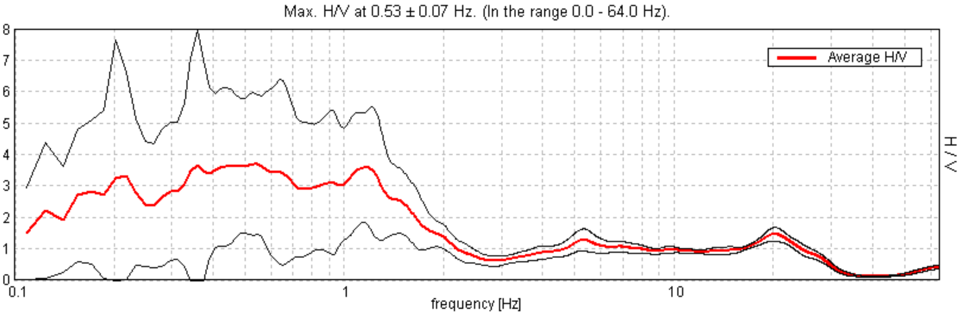


Figure 4 .6. Vibration frequency of the ground near the Coral Building

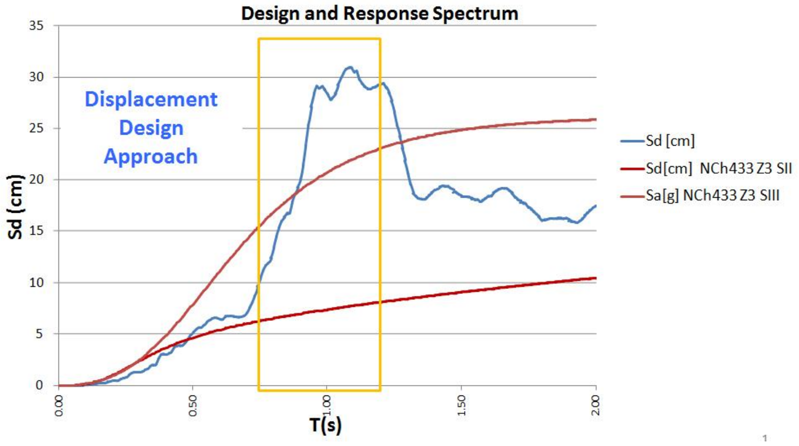


Figure 4.7. Response Spectrum in Viña del Mar for an artificial record.

5. OBSERVED FAILURES

Buildings of medium height located on soft ground suffered great deformations and severe damage. The damage observed in the buildings presented in Figure 2.1, were almost the same, brittle failure in compression and/or buckled bars.

To achieve a desired mechanism of collapse under lateral displacement in the design based on the behavior, under major seismic demands to the maximum considered in the design, a ductile behavior is necessary. If ductility is not achieved it is not possible to design for different levels of damage of different earthquake magnitudes. Without an improved design of R/C structural walls, which prevents brittle failures, the performance based design approach cannot be applied.

In modern buildings, built after the 1985 Chilean Earthquake, with wall thickness less than what was previously used, brittle failures have been observed with vertical reinforcement buckling occurring in walls with T and L transverse sections. Some failures could be explained from the compression strut in the web produced by flexural-compression and shear action (Figure 5.1 and 5.2).

A pushover analysis done on the Toledo Building, a building, which suffered extreme damage, close to the other three buildings, was studied extensively. The building is founded on a soil with similar properties to the others, but instruments were not allowed in that site. Results indicate that the observed failure could have happened at a lateral displacement in the roof equal to 141 mm, with crushing of concrete due to compression and shear. Compressed struts are shown with a red line in Figure 5.1.

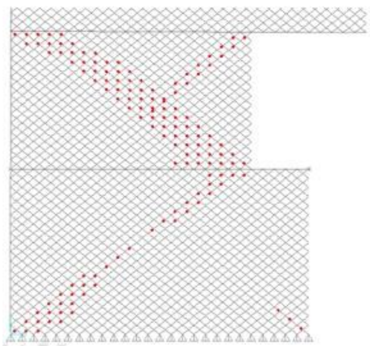


Figure 5.1. Compression struts from a pushover analysis

Figure 5.2. Observed failures in the Toledo Building after the February 27th 2010 Chilean Earthquake.

6. CHANGES TO SEISMIC CHILEAN CODES

After the February 27th 2010 Chilean Earthquake, changes were introduced to NCh433 and NCh430 seismic design codes.

Decree 61, modifies NCh433Of.96mod2009, the action code:

- There is a new classification of soil (A,B,C,D,E and F) I being equivalent to A, II equivalent to B, soil C is added, III is equivalent to ground D, soil IV equals E and soil F is added for special cases.
- The acceleration spectrum is maintained with respect to NCh433 with minor changes.

Table 6.1. Elastic design spectrum according to NCh433.

Parameter	Formula	Commentary
Design Spectrum	$Sa = \frac{I\ A_0\ \alpha}{R^*}$	I: factor of importance A_0: maximum effective acceleration by zone R^*: reduction factor α: amplification factor
Amplification Factor	$\alpha = \frac{1 + 4.5\left(\frac{T_n}{T_0}\right)^p}{1 + \left(\frac{T_n}{T_0}\right)^3}$	T_n: vibrating period of the n mode T_0 y p: soil parameter
Reduction Factor	$R^* = 1 + \frac{T^*}{0.1T_0 + \left(\frac{T^*}{R_o}\right)}$	R_0: structural system parameter R_o= 11 for shear walls T^*: mode period with major mass in the direction of analysis

An elastic displacement spectrum has been added only to calculate the lateral design displacement at the roof to determine special boundary elements in walls when ACI318-08 is applied. Since acceleration design spectra were corrected to obtain rational designs using elastic analysis, displacement spectra derived from acceleration spectra under estimated lateral displacements in buildings, then a factor C_d^* was introduced in equation (1), Figure 6.1:

$$S_{de}(T_n) = \frac{T_n^{\ 2}}{4\pi^2}\alpha A_0 C^*{}_d \qquad (6.1)$$

were C_d^* depends on type of soil.

The lateral design displacement at roof, δ_u, is calculated with Eqn. 6.2.

$$\delta_u = 1.3\ S_{de}(T_{ag}) \qquad (6.2)$$

where T_{ag} is the fundamental period of the structure calculated using cracked section properties, which can be approximated at 1.5 times the fundamental period when gross sections have been considered.

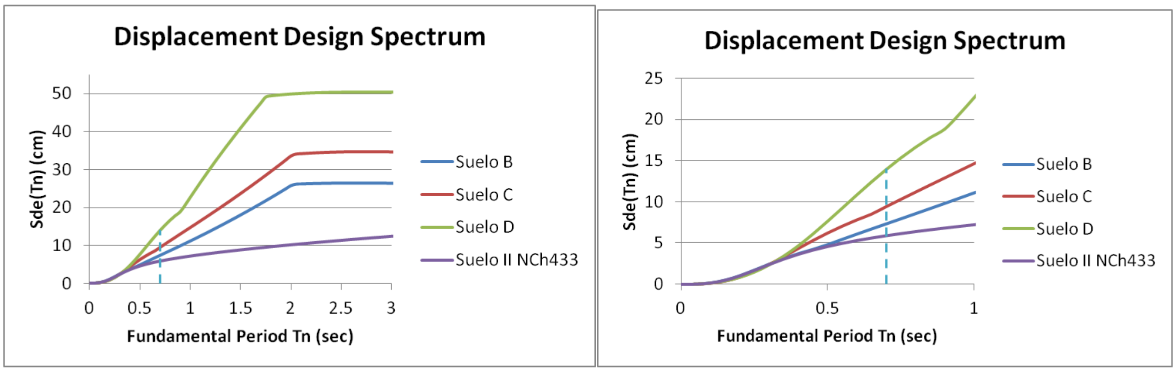


Figure 6.1. Elastic Displacement Spectrum for soil type B, C y D.

Decree 60, modifies NCh430, which makes ACI318-08 be applied in Chile.

- Limit the concrete strain ε_c to 0.008 (8‰) in extreme fiber in special boundary elements in walls.
- Limit the axial load: $P_u \leq 0.35f'_c A_g$

7. DEFORMATION CAPACITY

The data obtained in the type T wall, of the Toledo building, is shown with characteristics specified later, which is situated in zone 3 and soil type II.

Type of wall analysis: wall T, 200 mm thickness, height 26.51 meters plus 3.55 meters of underground, Reinforced Concrete f_c'=22,5 MPa, Steel f_y=420MPa.

$$\Delta_r = \Delta_e + \Delta_p \tag{7.1}$$

$$\Delta_e = \frac{H_W{}^2}{3} * \varphi_y \tag{7.2}$$

$$\Delta_p = \left(\varphi_u - \varphi_y\right) * L_p * (H_w - \frac{L_p}{2}) \tag{7.3}$$

$$\varphi_y = \frac{2\varepsilon_y}{L_w} \tag{7.4}$$

Where Δ_r, Δ_e and Δ_p are: total, elastic and plastic displacement at roof, respectively. H_w is the height of the wall between critical section and roof, φ_u and φ_y are the ultimate and yield rotation at the critical section. L_w is the length of the wall and L_p the length of the plastic hinge.

Table 7.1. Data obtained

ϕ_y	(1/m)	0.001
ϕ_u	(1/m)	0.001
Δ_r	(cm)	17
Drift	(%)	0.7

With:

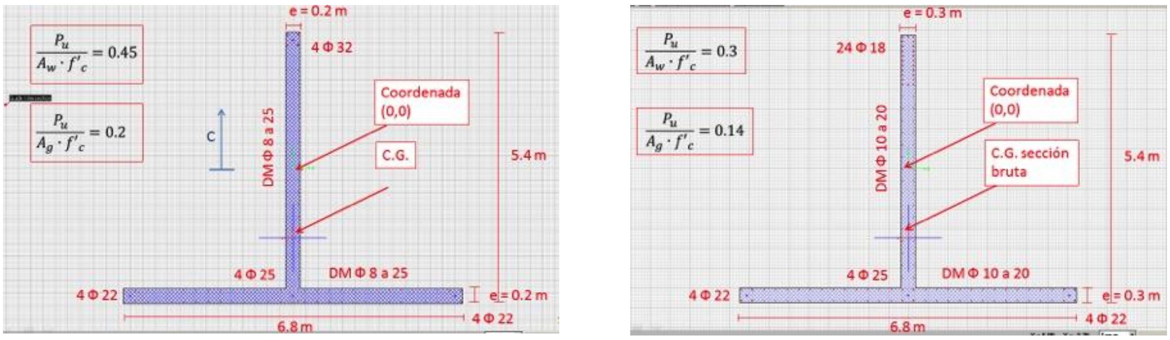
$$\Delta_p \cong 0 \tag{7.5}$$

$$\Delta_r \cong \Delta_e \tag{7.6}$$

$$\mu \cong 1 \tag{7.7}$$

Results show that there is not deformation capacity after reaching the yield displacement. Using the records obtained from the Feb. 27th 2010 Chilean Earthquake, a lateral displacement in the roof equalled to 320 mm could have occurred as a result of that earthquake, however the building would have failed at 170 mm.

Design of a wall with a T section has been assessed. The wall has been designed with dispositions previous to the 2010 Chilean Earthquake, Figure 7.1. a), and then with current regulations modified after the earthquake, Figure 7.1. b). Design of the wall using new dispositions of DS60, requires transverse reinforcement to confine concrete, improving ductility considerably. Deformations obtained from analysis according to the expressions Eqn. 7.2 and 7.3 are shown in Figure 7.3.



a) b)

Figure 7.1. a) T wall with 200 mm wall thickness. **b)** T wall with 300 mm wall thickness

Using design by capacity rules can be seen that the wall can reach its flexural capacity avoiding fails in shear. Over-strength in bending was estimated as $\Omega_0 = 2.3$ (Figure 7.2), but the wall did not reach a non linear behavior since failed before in a brittle manner, then a dynamic amplification factor should be taken as one because the higher mode effect after critical sections yield did not occur. Then, ultimate shear force can be calculated as $V_u=2.3*V_e=1550$ kN, smaller than the nominal capacity of the wall, $V_n=1910$ kN, explaining why typical diagonal shear failure did not occur.

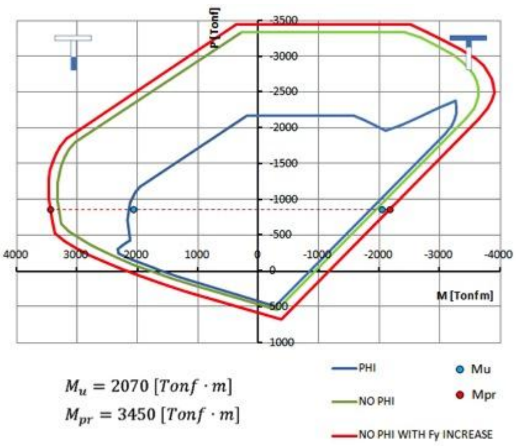


Figure 7.2. Interaction diagram for a T wall. Right: Flange in compression, Left: Web in compression.

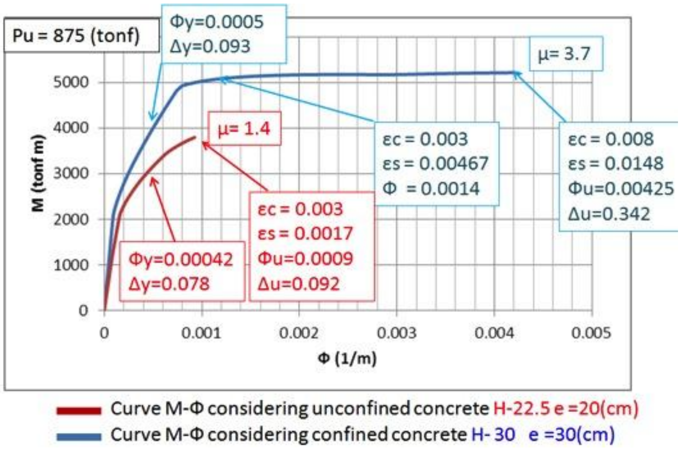


Figure 7.3. Comparison between T wall designed with NCh433 and DS60

It is also important to observe what happened to buildings near Toledo. For example, Magnolio building has similar characteristics to the Toledo building, period T_g=0.65 seg but its height is 31.8 m, 5.3 m higher than the Toledo. This structure did not suffer any damage; the building behavior was almost elastic, it was very hard to find any cracks.

The deformation capacity of one of the most critical walls is approximately 200 mm, the elastic deformation being 180 mm, and plastic 20 mm.

Difference between the deformation capacity of Toledo and Magnolio walls is very small and both of them cannot produce ductile behavior (Figure 7.5).

Figure 7.4. Magnolio and Toledo. Magnolio floor type, based on walls. The analyzed wall is marked.

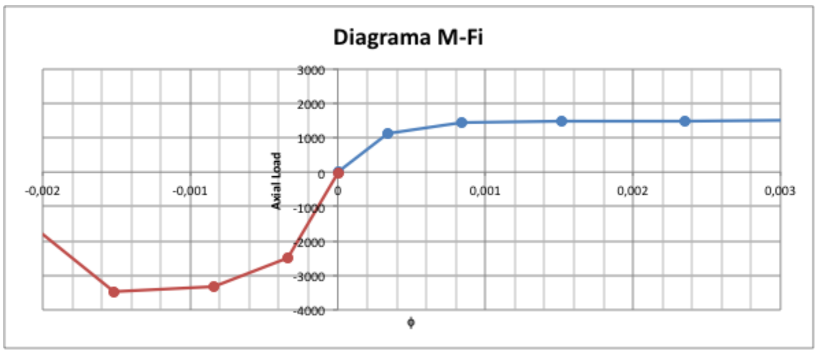


Figure 7.5. Moment-curvature diagram reaching the example wall.

8. CONCLUSIONS

Special boundary elements and transverse reinforcement according to ACI318, chapter 21, had not been considered in Chile previous to the February 27th, 2010 Earthquake. Some buildings founded on deep soft soils failed in a brittle manner without damage propagation in critical wall sections. Computed deformation capacity considering non-confined concrete was higher than demands obtained from non-linear analysis, in several cases of studied.

Diagonal compression failures could have produced observed damages when combined axial load, bending moment and shear are considered to evaluate the wall section behaviour under large displacements.

The displacement capacity of walls was not verify in the old Chilean standards, it is now estimated with a design displacement spectrum using the crack period of the building, and the soil classification now is more conservative than before.

Since ACI318 has been in effect, design lateral displacement in order to add special boundary elements in walls were obtained from linear elastic analysis, without reduction. However, elastic design spectra under estimated lateral displacement demands, especially in long period structures founded on soft soils. Changes in Seismic Codes were introduced after the 2010 Chilean Earthquake resulting in more ductile structures. Capacity design rules are an important issue to be considered in the future since walls are still designed for shear according to the current ACI318-08 version.

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