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The 2010 GREAT CHILE EARTHQUAKE - CHANGES TO DESIGN CODES

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ABSTRACT: The 27 February 2010 Mw 8.8 Maule in Chile produced a disproportionate concentration of structural damage in newly built buildings, including the collapse of three buildings, and widespread damage to nonstructural elements prompted the government to revise current design practice. Results suggest that structures with post yielding positive hardening must be used in soft deep soils since an instable response could arise. Non linear displacement spectra must be proposed to determine the design displacement, instead of linear ones.

Key Words: 27 February 2010 Mw 8.8 Maule in Chile earthquake, long strong shaking, several cycles of high accelerations.

INTRODUCTION

The 27 February 2010 Mw 8.8 Maule in Chile generated MM VII intensity or higher and PGA = 0.3g or higher on a 100 km wide by 600 km long corridor, as well as a tsunami. About eight million people live in this corridor. The central government, and a large percentage of the domestic and export industries are located within. The direct cost of damage may exceed US $25 billion, or about 16% of GDP.

The earthquake occurred over the known Concepción-Constitución seismic gap in central Chile, where several prior studies had concluded the most likely near future occurrence of an earthquake of the characteristics observed (Boroschek et al. 2006, Ruegg et al. 2009). The rupture models of the main shock processed by the USGS (2010), Delouis et al. (2010) present two or three asperities with high slips and energy release.

Several strong motion records were obtained by the University of Chile accelerograph network during this earthquake. Peak ground accelerations, some of them in the order of 0.9 g are presented in Boroschek et al 2010. Figures 1 and 2 show the ground motions recorded at Viña del Mar Marga-Marga and downtown Concepción stations, at epicentral distances of 340 and 100 km, respectively. These records are long, which typifies this kind of large magnitude subduction earthquakes. For example, the Viña del Mar Marga-Marga record present strong motion duration based on 90% of Arias Intensity of 30 sec and an envelope with three periods of large amplitude vibrations.

For the case of downtown Concepción, Figure 2(a), the record shows a duration of 85 seconds and include more than 10 cycles of high accelerations with amplitudes greater than 0.10 g. This characteristic is strongly related with geotechnical conditions of the site (Class D, average Vs of 220 m/sec). Narrow band-pass filtering of this record at T = 1.9 sec shows 14 sinusoid cycles of increased acceleration resulting from the response of the deep alluvial soil column where the station is located, see Fig 2(b). A more detailed description of the sites can be found in Arango et al, 2011.

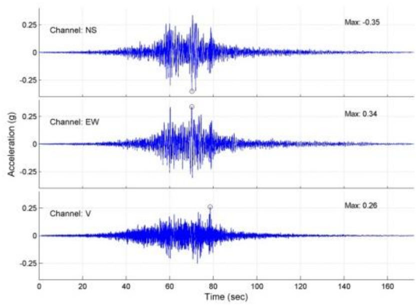
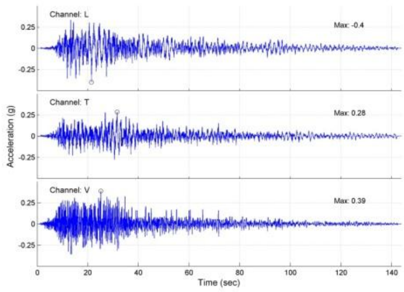
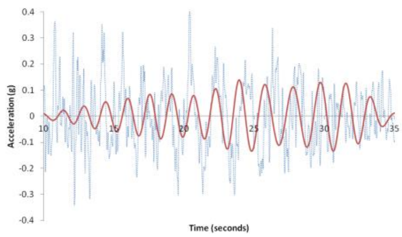


Fig. 1 Corrected acceleration time histories. Station: Viña del Mar-Puente Marga Marga. Filter band [0.05 90] Hz.





(a) (b)

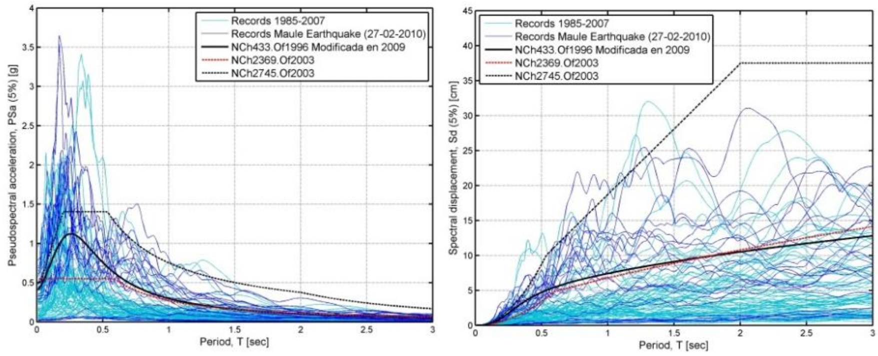
Fig. 2 Corrected acceleration time histories. Station: Concepción-Colegio Inmaculada Concepción. (a) Acceleration Signal Filter: [0.2 80] Hz. (b) Filtered Acceleration Signal - S63^{o}W component.

DESIGN CODES AND DESIGN SPECTRA

In Chile there are several loading and design codes, differentiated by their functionality or structural system. The loading codes are: NCh433 for residential and office buildings; NCh2369 for industrial installations; NCh2745 for base isolated buildings. In general, the seismic demand is represented by seismic zoning, a reference design zero period acceleration (ZPA) and a design response spectrum for elastic response. ZPA and response spectra are not strictly derived from a probabilistic or deterministic hazard study. The shape of the spectra has been calculated from the shape of the average acceleration response spectra normalized by PGA for the database of historical ground motions of Chile. This shape has been incorporated into the NCh433 and NCh2369 loading codes. With such spectra, tall buildings that have become common in Chile since 1985, are subjected to unrealistically small displacement demands as evidenced in the displacement response spectra depicted in Fig. 3.

In contrast, Newmark type acceleration response spectra have been incorporated into the ETG 1015 and NCh2745 loading codes. In NCh2745 the combined effective acceleration and spectral shape is scaled, cut or amplified based on the results of analysis on several typical reference structures. Then design spectrum in any Chilean code does not correspond to elastic seismic demands derived from local recording by large factors. As can be seen in Figure 3, the criteria for the shape and scale of each code design spectra is quite different. The NCh433 was calibrated so it has the average shape of the response spectra real records, but values are scale so residential buildings with period between 0.5 and 1.5 seconds have a base shear coefficient consistent with pre 1985 design standards. In NCh2369 the spectral shape contained in NCh433 is used as reference, but in the industrial code the spectral values are limited by a maximum coefficient generating an extremely low equivalent plateau. This was done so rigid structure, equipment or system, could be design with the traditional values used prior to the development of the code.

Strong emphasis to displacement demands is given in the spectrum incorporated into NCh2745 for buildings with periods greater than 1.5 sec. This is because it was deemed that the design of base isolated buildings should result in large displacement demands. Moreover, the ZPA value in this code does reflect the results of a probabilistic study; and the plateau in the design spectra represents a typical amplification factor for the acceleration control band of the spectra using Chilean records.



(a) (b)

Fig. 3 Response spectra at 5% damping for records obtained in zone 3 and soil type II according to Chilean seismic codes. Elastic demands of NCh433, NCh2369 and NCh2745 are shown. (a) Acceleration Spectra. (b) Displacement Spectra.

Lateral forces obtained from the base response spectrum in each of the loading codes are modified by the importance of the structure, by its inherent ductility and damping. Minimum and maximum base shear limits may control these forces. Limits are also placed to controlled inter story drifts.

All loading codes in Chile only require a linear elastic analysis. A traditional modal response spectrum method is the preferred choice. Nonlinear pushover analyses are not mandatory but such analyses have been performed to support the design of some tall residential buildings as part of the peer review verification process. Nonlinear time history analyses are rarely used, and have been conducted to support the design of some base isolated buildings and of buildings that incorporate supplementary damping devices.

DESIGN LATERAL DISPLACEMENT

A difficulty exists in the calculation of the displacement demands for buildings on soft soil conditions in Chile. The 2010 issue of the Chilean code NCh433-mod2010, calculates the lateral displacement at roof as:

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

where S_{de} is the elastic displacement spectrum ordinate for 5% damping ratio, calculated with the fundamental natural in the analysis direction considering cracked sections.

The problem with this formulation is that, in soft soil, the amplification in the displacement demand of an inelastic building with a period shorter than the predominant period of the soil can be underestimated. For example, the ground motion as recorded in "Concepción Centro", on a deep alluvial soil with a 30 m shear wave velocity of 220 m/s, induces large displacements in non-linear structures whose period is somewhat shorter than the predominant period of the soil. When soil-structure interaction is accounted for, something that is seldom considered in design, the period of the soil-structure moves closer to that of the soil and the structure experiences larger displacement demands. Such demands are completely overlooked in design.

Figure 4 compares the spectral displacements obtained from the ground motion recorded in downtown Concepcion, on Soil Type III, for elastic response and for inelastic response for a ductility of 2 and 3 with those derived from the spectral ordinates of three of the Chilean codes. It is evident that the spectral displacements calculated with NCh2369 are significantly below those computed from the ground motion for elastic and inelastic response. The spectral ordinates of NCh2745 are somewhat similar to those computed for inelastic response, but below that computed for elastic response. The spectral displacements computed from NCh433 are greater than those computed for the ground motion only for periods below 1.2 sec.

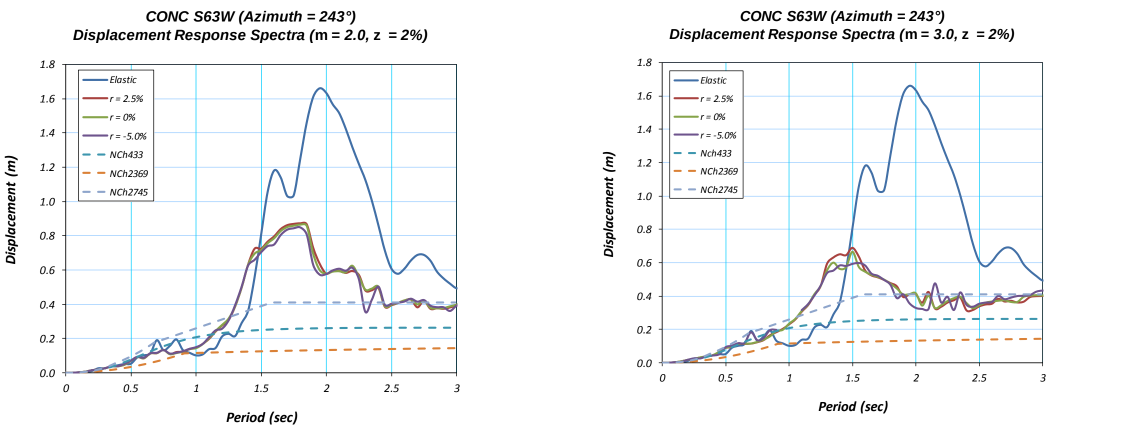


Figure 4. Displacement response spectra, Concepcion Centro, Maule Earthquake and Chilean codes, Zone III, Soil Class II and III.

A reason for the mismatch of the code and ground motion displacement spectral demands is the lack of recognition that in these soil conditions the motion has strong sinusoid components. To demonstrate this, the displacement coefficients, C_μ, have been computed for the ground motion and for a record with 14 sinusoids of increased amplitude. C_μ

coefficient has been defined as the ratio between the maximum displacement of a non-linear single degree of freedom system and the maximum displacement of a linear one with same initial stiffness (Miranda, 1999). Comparisons with those obtained from the semi-synthetic record are illustrated in Figure 5.

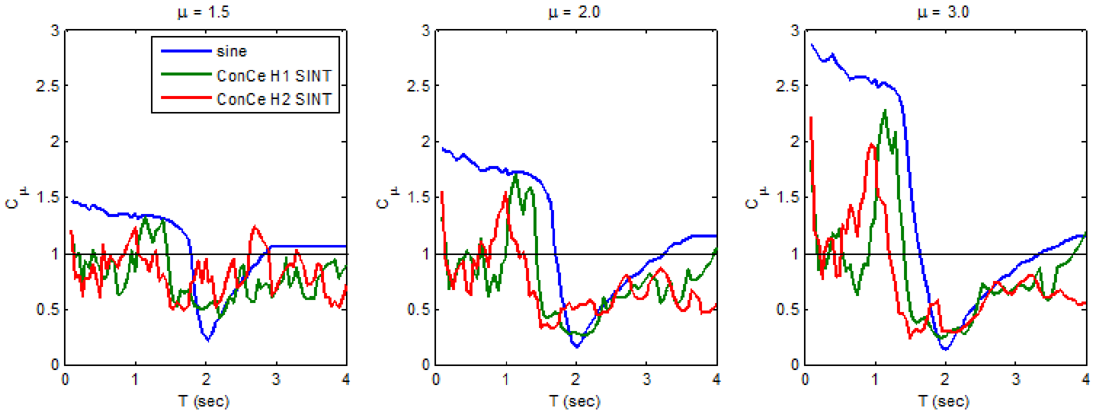


Figure 5. Comparison between C_μ values for harmonic representation and the ground motion.

COUPLED WALL RESPONSE

The two reinforced concrete bearing walls coupled by the slabs shown in Figure 6, has been chosen to evaluate the response to a ground motion on a soft soil such as that registered in Concepcion. These walls are very similar to those employed in 16 story building that collapsed during the 2010 Chile Earthquake in Concepcion Centro.

Lateral displacement at roof computed for $r = 0$ and $r = -0.025$, are shown in Figure 6.

A semi synthetic record representing the ground motion registered in Concepcion, during the Chile 2010 earthquake, in a soil with similar characteristics to the soil where the building was founded has been considered. For $r = 0$, the maximum lateral roof displacement was close to 40 centimeters, with a stable response. Analysis was done using Ruaumoko (Carr). When a decay of resistance is considered, for $r = -0.025$, a non linear resonance phenomena occurred, being the response instable. Lateral displacement increases until collapse. Post yield hardening is represented by factor r computed from the moment curvature relationship. Slab coupling effect was taken into account through a bar element assuming an effective web.

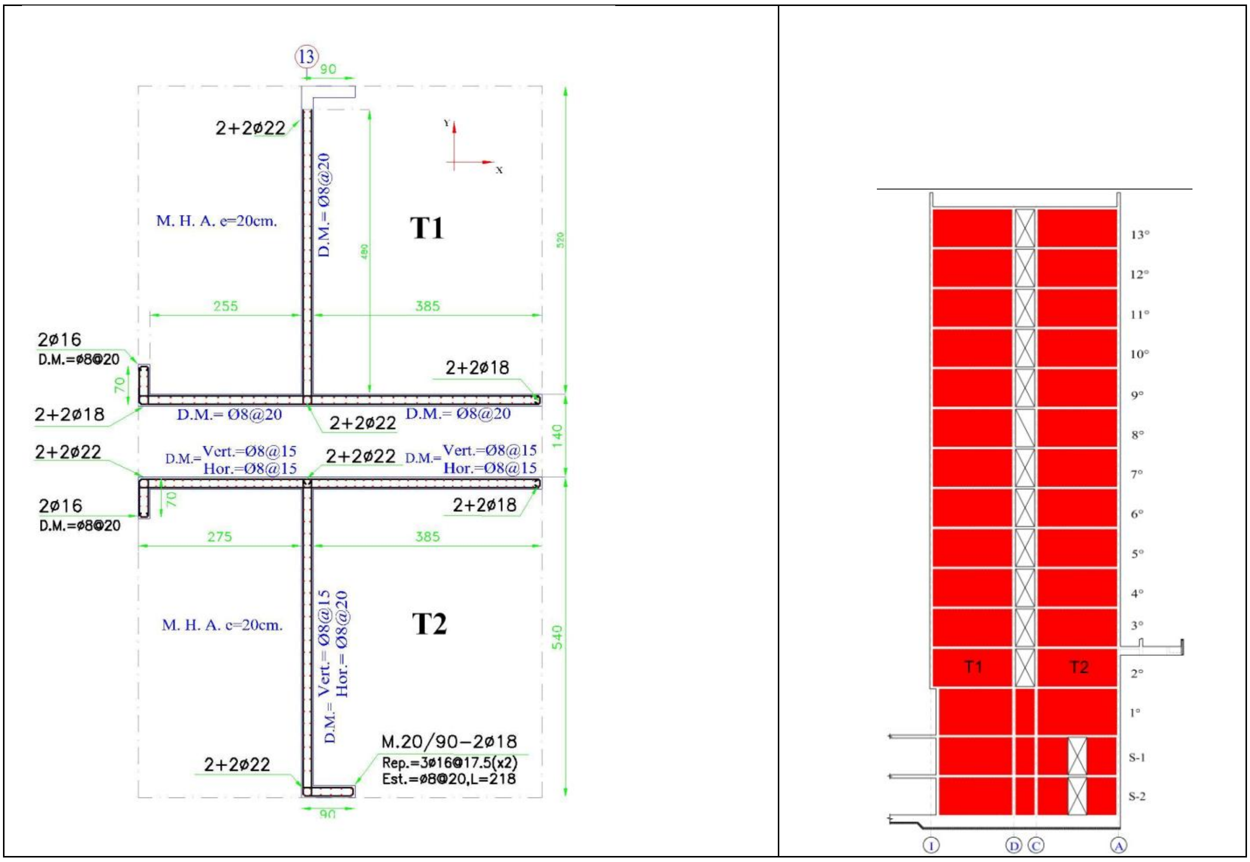
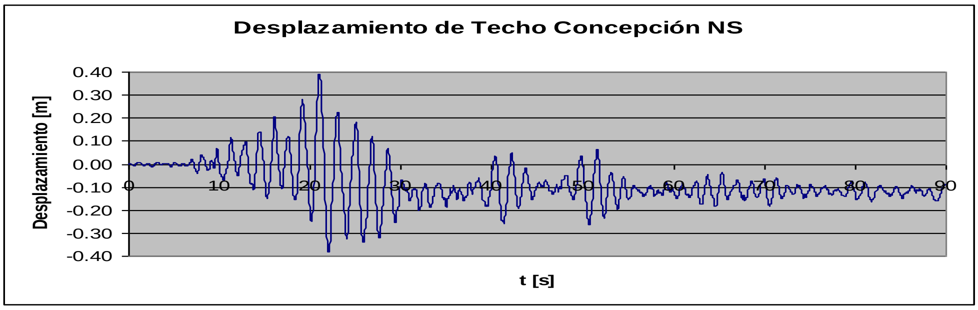
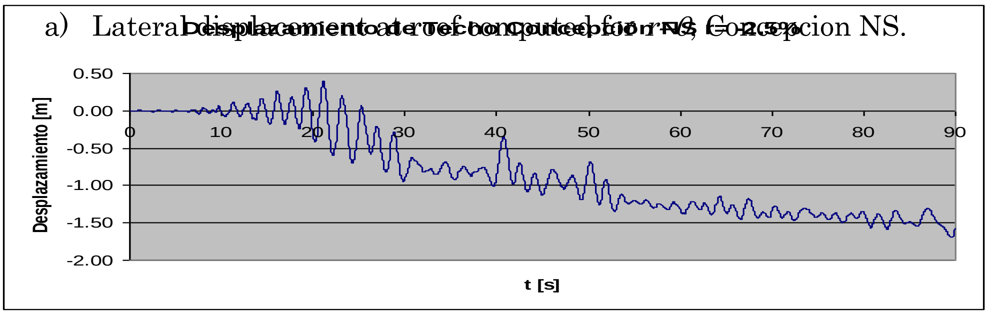


Figure 6. Coupled walls



a) Lateral displacement at roof computed for $r = 0$, Concepcion NS.



b) Lateral displacement at roof computed for $r = -0.025$, Concepcion NS.

Figure 7. Lateral displacement at roof computed for $r = 0$ and $r = -0.025$, Chile 2010, Concepcion NS record, Ruaumoko.

PROPOSED MODIFICATIONS TO ACI318-08 TO DESIGN R/C SPECIAL WALLS IN CHILE

The overall performance of buildings in Chile to the Maule earthquake was rather acceptable, in terms of the amount of damage observed. However, it is evident that newer high-rise buildings suffered a disproportionate percentage of the damage observed. Concentration of damage in high-rise buildings was observed in softer soils conditions. The ground motion obtained from Concepción has shown sensitivity to strength degradation in the period band corresponding to the fundamental period of damaged high-rise buildings built in the area.
Performance based design requires a ductile behavior to permit the structures to reach a desire collapse mechanism under lateral displacements, greater than demands of the maximum considered earthquake. If not, it is not possible to design for different damage levels for different intensity earthquakes. Unless changes in reinforced concrete wall design avoid the brittle nature of unexpected observed damages in the 2010 Chile Earthquake, performance based design in this type of structures could not be applied.
Plants of damage buildings are similar to that one shown at Figure 8(a). Typical damage in R/C walls are shown in Figures 8(b). Several R/C buildings had damage in wall boundaries as shown in Figure 8(c).

(a) (b) (c)

Fig. 8. (a) Typical plant of R/C wall building in Chile. (b) Typical failures in R/C walls, localization caused by buckling of vertical reinforcement. (c) Buckling of Bars in Reinforced Boundary Element.

Urgent modifications were proposed to design especial R/C walls in Chile, after the February 27th, 2010, Earthquake. A law has been promulgated on February 14th, 2011 adopting ACI 318S-08 to R/C building design, but complemented by following requirements presented in this paper. Complementary requirements shall be applied only to R/C special structural walls, defined in section 21.1 in ACI 318-08.
Most of observed failures occurred in thin walls, used in Chile since the eighties. Some old buildings that had had damages in the 1985 Chile Earthquake in Viña del Mar, had several damages again. To avoid brittle failures under bending and axial forces, limitations to axial force have been proposed. A conventional strain in concrete equal to 0.003 has been adopted by ACI318-08 to define a nominal strength under axial load and bending. Under large deformations, shortening strain in concrete is usually greater than this conventional value at critical zones. Tensile strain in steel when shortening strain in concrete is equal to 0.004 in the opposite fiber is used as an index to identify the type of

failure. Failures controlled by compression, as defined currently in ACI-318 for members in flexure see Fig. 9, were not permitted in walls. Sections controlled by tension are recommended in walls, with transverse reinforcement to avoid premature buckling of longitudinal bars in extreme fibers or in boundary elements and adequate confined be given to concrete. Analogy to dispositions for beams in §10.3.5 in ACI318-08, tensile strain in steel ε_t at nominal strength in special walls must be greater or equal to 0.004. Large lateral displacement must not be reached with large shortening strains at confined concrete, longitudinal steal in opposite fiber must yield before the ultimate strength capacity is reached in concrete under compression. For effective confinement considerable thickness are needed in walls, tests done in columns simulating boundary elements similar to actual boundary elements in walls failed under compression during the February 27th, 2010 Chilean Earthquake, accomplishing ACI requirements, have shown how brittle failures can be.
Then, maximum factored axial load permitted in walls must be $\boldsymbol{P_4}$, associated to a tensile strain in steel equal to 0.004 when shortening strain in concrete in opposite fiber is 0.003.
Then, to avoid compression failures in structural walls a special disposition was added to ACI318-08: “The net tensile strain in the extreme tension steel, ε_t, must be equal or greater than 0.004 when the concrete in compression reaches its assumed strain limit of 0.003.”

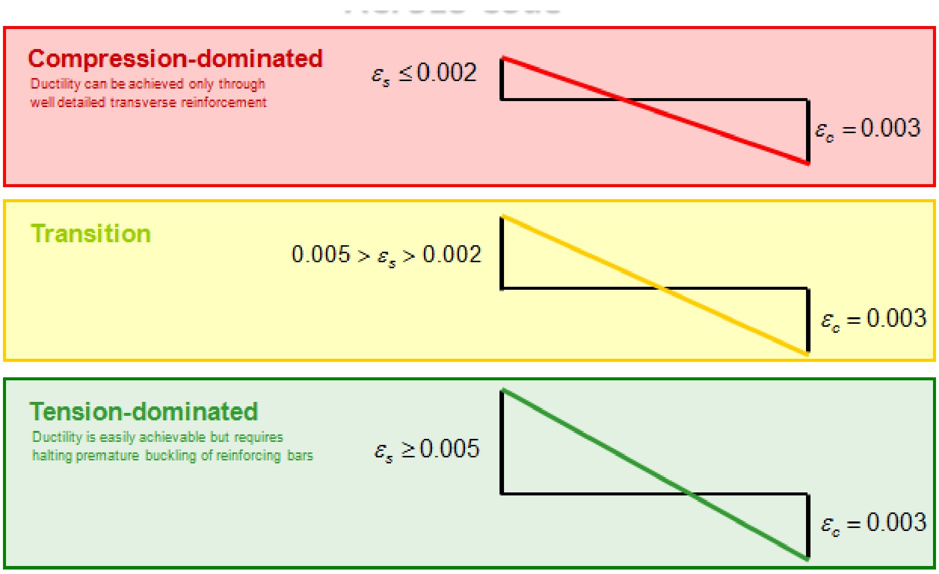


Fig. 9 Combined Flexure Axial Design Approach ACI318-08 Code.

Figure 10 shows one of the walls of the building shown in Figure 8(b), and Figure 11 shows the moment curvature relationships for one of the walls. It is evident that the wall is brittle when the flange is in tension.

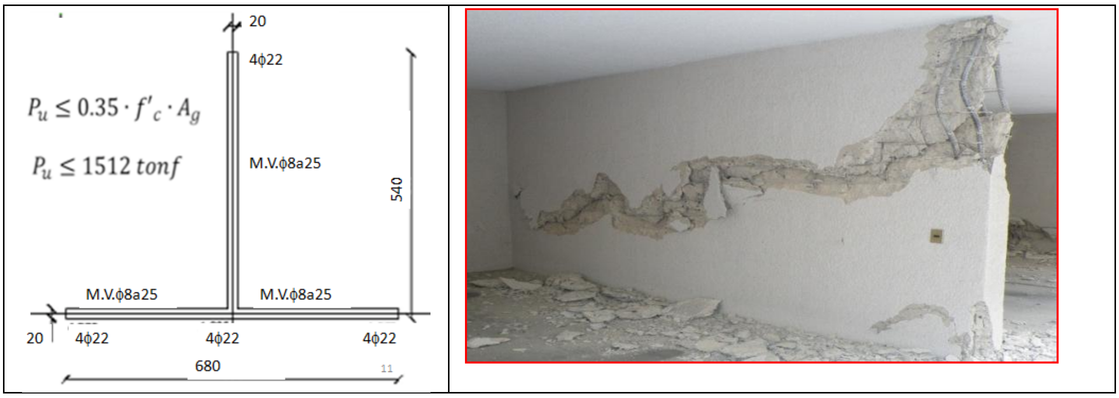


Fig. 10. Walls as in official drawings and state after the 2010 Chile Earthquake.

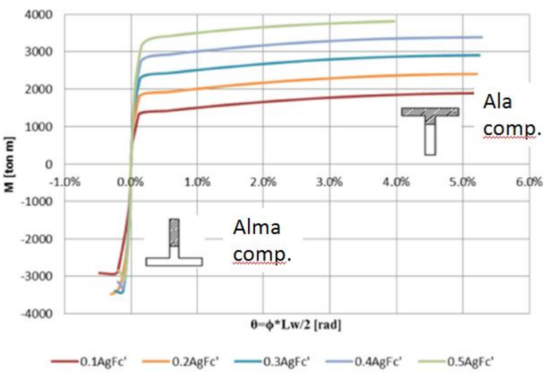


Fig. 11. Moment curvature diagrams for one of the walls

Curvature at yield can be computed according to different criteria resulting in different results. If curvature at yield is estimated as $\phi_y = \frac{2\varepsilon_y}{L_w}$, then

$$\phi_y = \frac{0.004}{5.4} = 0.00074\ [\frac{1}{m}]$$

As $\delta_y = \frac{11}{40} * \phi_y * H^2$ then $\delta_y = \frac{11}{40} * 0.00074 * 26^2 \left[\frac{1}{m}\right] = 0.14 \left[\frac{1}{m}\right]$

The design lateral displacement according to the current Chilean code is 290 mms.

When the building was designed the displacement capacity did not have to be evaluated, according the codes.

A new law approved in Chile in December 13, 2011, permits to consider the elastic deformations separately from the plastic deformation.

$$\emptyset_u = \frac{\delta_u - \delta_e}{L_p(H - \frac{L_p}{2})}$$

But in this transverse section, $\phi_y = \phi_e = \frac{2\varepsilon_y}{L_w} = 0.00074 \left[\frac{1}{m}\right]$

For the considered wall, strain in steel is $\varepsilon_s=0.0017$ when shortening in concrete is $\varepsilon_c=0.003$, then the wall is controlled by compression.

According to the new Chilean law, if transverse reinforcement is necessary to confine special boundary elements, then a minimum thickness of 300 mm is required. When thickness is increased $\varepsilon_s=0.0038$ when shortening in concrete is $\varepsilon_c=0.003$, then the wall is in the transition zone. A maximum shortening in confined concrete in boundary elements is limited to $\varepsilon_c=0.008$, for ultimate limit state. Then, at ultimate, the wall could reach a lateral displacement as large as 340 mm, 1.3% of height, with a steel strain equal to $\varepsilon_s=0.015$. Anyway, T section walls can be very brittle.

CONCLUSIONS

Results suggest that structures with post yielding positive hardening must be used in soft deep soils since an instable response could arise. Non linear displacement spectra must be proposed to determine the design displacement, instead of linear ones.

ACKNOWLEDGMENTS

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