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ASSESSMENT OF GROUND MOTION AND CORRELATION WITH DAMAGE IN BUILDINGS, 2010 CHILE EARTHQUAKE

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Abstract

Several strong motion records have been obtained during the 2010 Maule Earthquake in Central Chile. Medium rise RC buildings were badly damaged in Concepcion and Viña del Mar cities. New high-rise buildings located on soft soils suffered damage in the lower levels of walls during the 2010 Chile Earthquake, while similar buildings behaved quite well when they were located on firm soils. Buildings with natural periods shorter but close to the soil site predominant period suffered damage in their walls, particularly where the bedrock depth was deeper than 70 meters. Assessments of some damaged buildings have concluded that the observed type of damage would have occurred at approximately one per cent drift, with a very brittle behavior.

The only record obtained in Concepcion city soft soil has been useful to understand the damage in many medium rise R/C buildings constructed in that area. This paper describes recorded surface ground motion in Concepcion city obtained in the 2010 Chile Earthquake. Assessments of soil amplification and frequency content evolution during the earthquake and its correlations with damage in buildings are presented. Apparently building damage in Concepcion city would have occurred after the time of arrival of the seismic pulse from the asperity. Buildings with damage in walls showed to be very brittle and they failed before 20 seconds of initiated the earthquake.

San Pedro record represents areas where the rock under the surface of soft soil is around 25 to 40 meters. When buildings are constructed on this relatively firm soils, lateral drifts would be in order of 0.6 to 0.8 percent of height, a response with low incursions into the nonlinear range with visible cracks, and then it can be classified as immediately occupancy. However, in areas similar to Concepcion Center, where rock depth is greater than 120 meters, large lateral drifts are expected, between 1 to 1.4 percent of height. For well detailed structural walls, one percent drift can be related to moderate damage, but 1.4 % drift could be excessive. Lateral drifts do not depend significantly on yield forces. Precautions must be adopted to limit drifts and supply ductility in these soil conditions.

Keywords: 2010 Earthquake; damage; amplification; frequency content; rock depth.

1. Introduction

The M_w 8.8 Maule earthquake (Chile, 27th of February 2010) generated Mercalli intensities of the order of MM, VII or higher in an area 100 km by 600 km. About eight million people lived in this area in 2010, forty five percent of the population of the country. The central government and a large percentage of the domestic and export industries are located within this area. Direct costs of damage might have exceeded US $25 billion, about 16% of the GDP.

Approximately 9,900 buildings with 3 or more stories were built in this area since 2001 [1], with only 3% of them taller than 10 stories. During the 2010 Maule earthquake, forty buildings were severely damaged in the city of Concepcion and only one 15-stories building experienced collapse. Only ten buildings had severe damage in the coastal city of Viña del Mar, but none of them collapsed. Ten and four of the most severely damaged buildings were demolished in Concepción and Viña del Mar, respectively. Most engineered buildings, designed in accordance with the Chilean practice, might have sustained lateral drifts lower than 0.5%, an indication of a nearly operational response. The historical Chilean practice of using a high density of shear walls as the seismic resistant system, instead of frames, has been considered as the reason of a good global performance of high-rise buildings during the 2010 Chile Earthquake [2]. However, the damage that occurred in this small group of buildings can be mitigated by including some changes to current analysis and design codes.

Four million dwellings were affected by the 2010 Earthquake, and 1,424,809 (36% of the total) had insurance against earthquake damage [3]. Most of the damage occurred in non-engineering housing. According to government statistics, 370,000 dwellings had severe damage (MINVU, 2010).

2. Damage in reinforced concrete (R/C) buildings and soil conditions

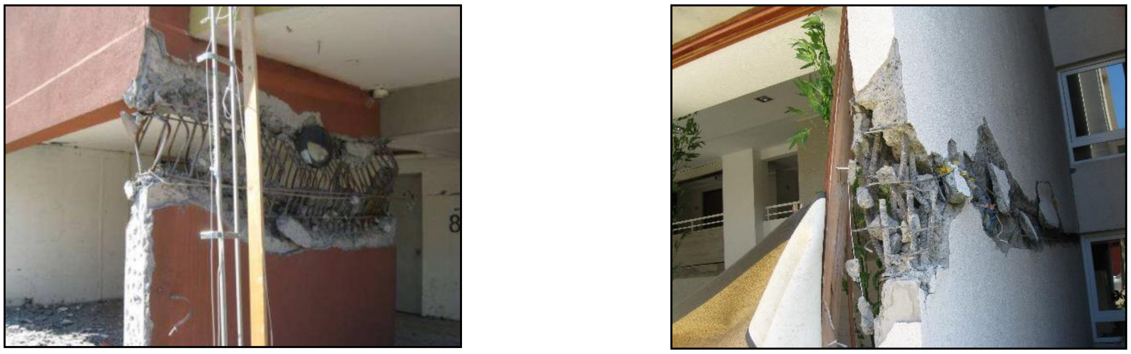
The overall performance of R/C buildings in Chile during the Maule earthquake and during previous seismic events has been considered as acceptable, given the number and severity of the observed earthquakes. However, many newer high-rise buildings constructed on softer soil conditions (i.e. type D or E) suffered damage in the lower levels of their walls, as shown in Fig.1 and Fig.2.

Assessments of buildings designed before the 2010 earthquake have concluded that the typical Chilean tall buildings can withstand lateral drifts between 0.5% and 1% without experiencing visible damage.

(a) Antigona, repaired.

(b) Festival, demolished.

Fig. 1 – Damage in walls in buildings in Viña del Mar

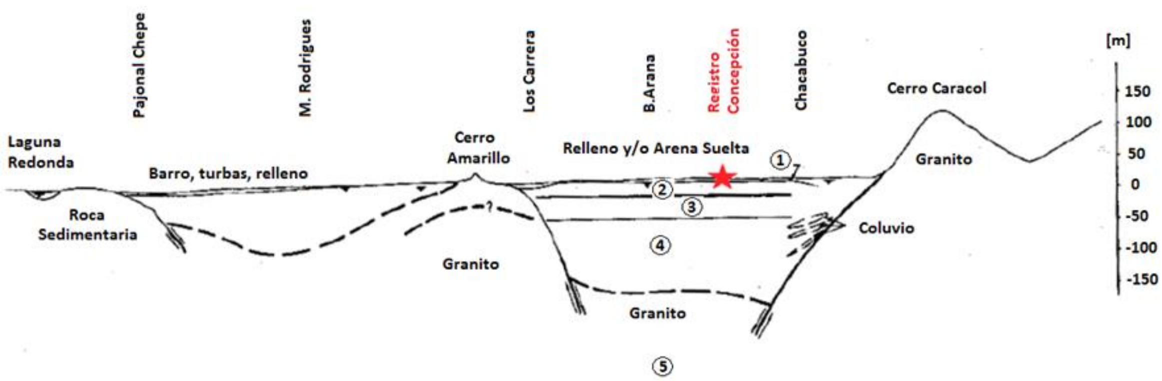


(a) Centro Mayor, demolished. (b) Calle Salas, repaired.

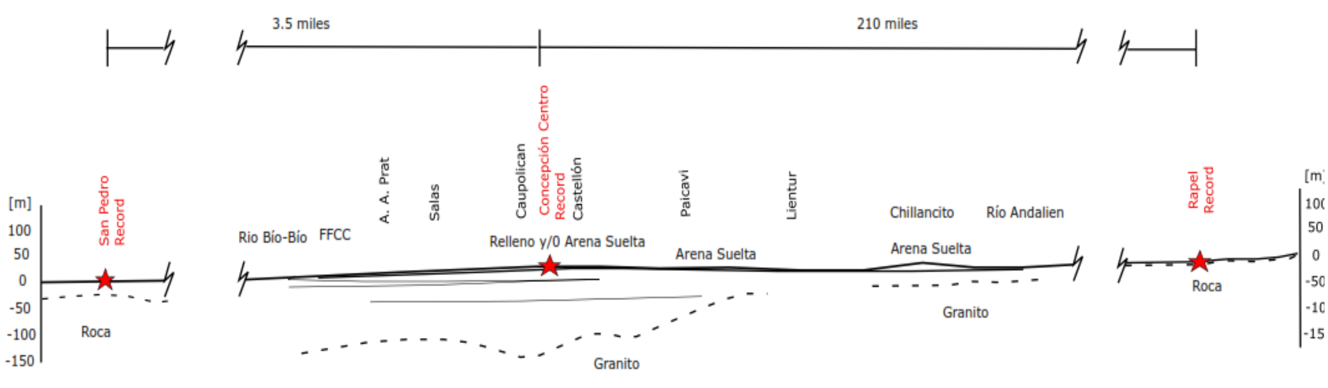
Fig. 2 – Damage in walls in buildings in Concepción.

3. Recorded ground motions close to damaged buildings

Most of the buildings with severe damaged were constructed on top of soft soils where the bed rock was deep. There is only one ground motion recorded under those conditions in Concepción city center [4], where many buildings experienced damage similar to that shown in Fig. 1 and Fig.2. There is another record in San Pedro, de la Paz, shown in Fig.5, 3.5 miles from Concepcion city, obtained where bed rock is located at 25 meters of depth. The closest ground motion recorded on rock was obtained in Rapel (Fig.6), 210 miles north of Concepción. Even though the distance to Concepción is large, Rapel record has a similar distance to the fault surface, so this record is use to assess soil effects in Concepción.



(a) Cross Profile

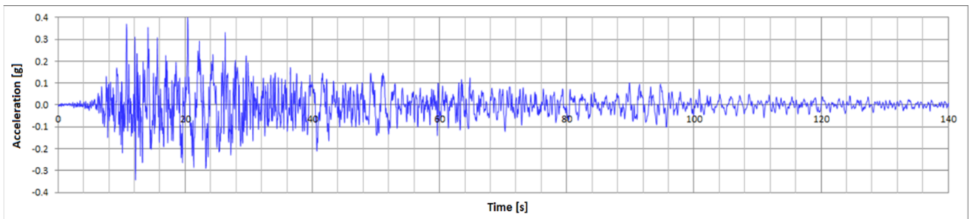


(b) Longitudinal Profile

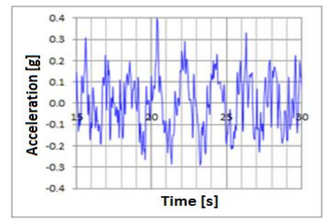
Fig. 3 – Concepción cross and longitudinal profile. Dobry and Poblete [5].

The ground motion obtained in Concepción Centro city was recorded on a class D soil, according to the Chilean standard [6] [7], with average shear wave velocity Vs of 220 m/sec (722 feet/sec) [5]. The duration of the strong part of the ground motion was 85 seconds with more than 10 cycles of high accelerations with amplitudes greater than 0.10g, Fig.4. This characteristic is strongly related with geotechnical conditions of the site. Narrow band-pass filtering of this record at T = 1.9 sec shows 14 sinusoid cycles of increased acceleration [8] result of the response of the deep alluvial soil column where the station is located, as shown in the longitudinal and transversal profiles in Fig.3.

The record obtained in Concepción Centro "Concepción Colegio Inmaculada Concepción, longitudinal direction", in this paper will be referred as Concepción L from now on. Table 1.



(a) Concepción Centro L Record.



(b) Time lapse with low frequency content.

Fig. 4 – Acceleration time histories. Station: Concepción-Colegio Inmaculada Concepción

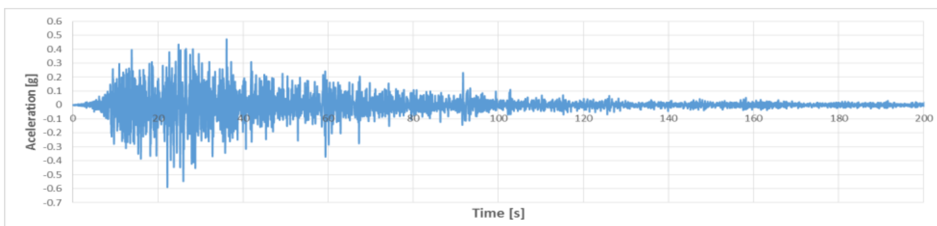


Fig. 5 – San Pedro EW Record.

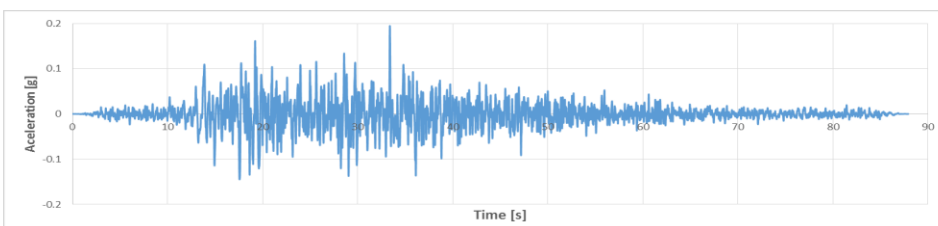


Fig. 6 – Rapel EW Record.

Table 1 – Record characteristics

Station	Direction	Record name	PGA [g]
Concepción Colegio Inmaculada Concepción	Longitudinal	Concepción L	0.4
San Pedro de la Paz	East-West	San Pedro EW	0.59
Rapel	East-West	Rapel EW	0.2

Displacement response spectra have been considered in this paper to correlate damage with the characteristics of the ground motion. Fig.7 shows linear displacement spectra for the main horizontal component of the three records considered. Buildings with important damage in Concepcion and Viña del Mar had fundamental periods around 1.0 second, and at this period the spectral displacements is approximately 300 mm in the EW component of San Pedro which is nearly three times larger than does observed for Concepción city (approximately 100 mm). The spectral displacement computed with Rapel EW record, obtained on rock, are less than 100 millimeters for all periods.

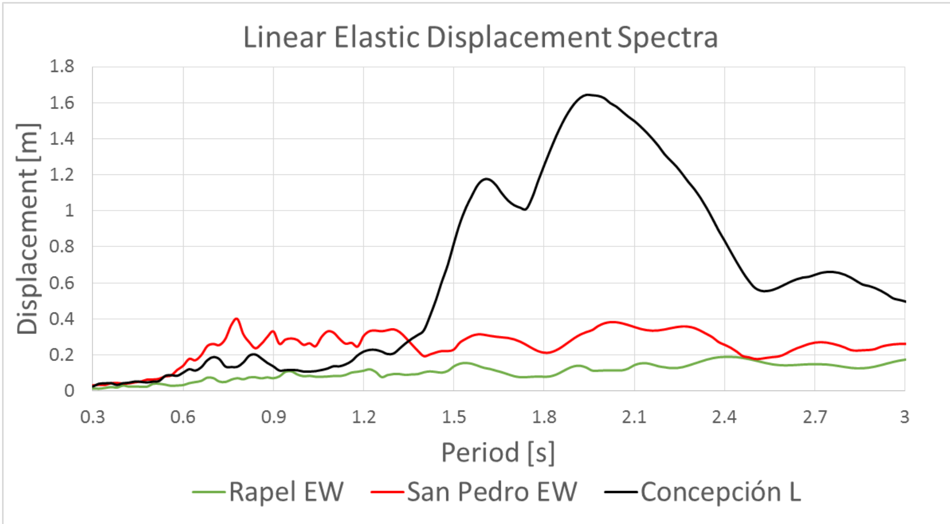


Fig. 7 – Linear elastic displacement spectra for Rapel EW, San Pedro EW and Concepción L Records.

R. Saragoni and S. Ruiz [9] compared the ground displacement histories of the records Concepción L record and San Pedro EW, shown in Fig.8. The ground displacement obtained integrating the recorded ground acceleration, which was processed using a Linear Baseline Correction and applying a Butterworth Bandpass filter (order 4, 0.01- 25 Hz). Two long pulses have been identified [9]. In Fig.8 it can be noted that for both records the first pulse arrival occurs at 10 seconds.

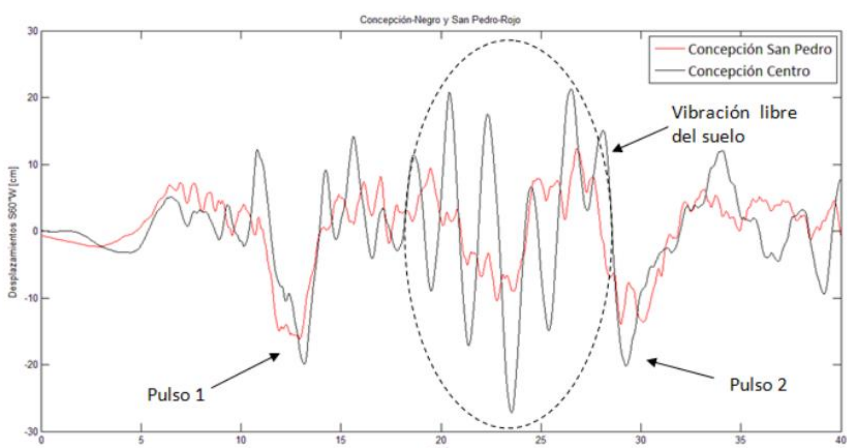


Fig. 8 – Identification of two seismic pulses and free vibration of ground displacement record S60 ° W Concepción Centro and San Pedro EW. Saragoni and Ruiz [9]

Two predominant frequencies, (0.6 and 1.6 Hz) can be identified in the Concepción L record using a Short Time Fourier Transform. These frequencies can be associated with soil predominant periods as shown in Fig.9. Also it can be observed that there is a visible significant change in the predominant frequency during the ground motion as a function of amplitude varying from 1.7 to 2.0 seconds.

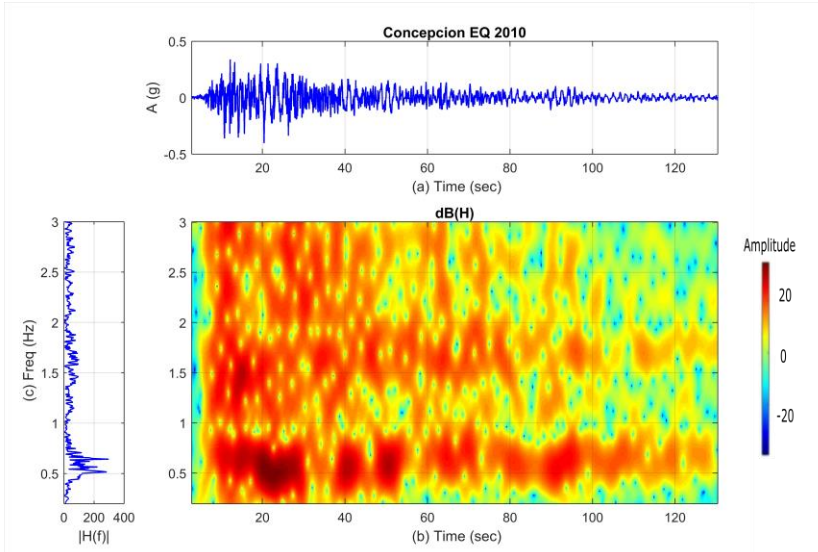


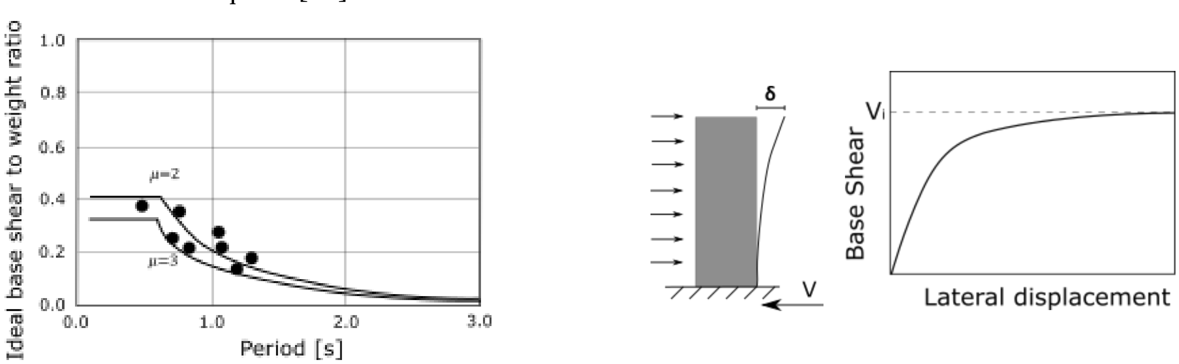
Fig. 9 – Short time fourier transform amplitudes, Concepción Centro L Record.

4. Nonlinear Response of Structures

Actual buildings normally respond within the nonlinear range under major seismic events. As such, nonlinear spectra shall be considered for assessing their damage. To evaluate the effect of the strength of the structure on the response, nonlinear displacement spectra can be constructed as a function of the yield force. For a single degree of freedom system the yield force to weight ratio has been defined as C_y, Eq. (1).

$$C_y = \frac{F_y}{Weight} = \frac{F_y}{m\,g}; \qquad F_y: Yield\ Force, \qquad m: Mass, \qquad g: Gravitational\ acceleration \tag{1}$$

Actual buildings have ideal base shear to weight ratios between 30 and 40 percent for short periods, and between 15 and 20 percent for natural periods around 1.0 seconds. Fig.10 shows assessments for actual buildings after the 1985 Chile Earthquake [10].



(a) Ideal base shear at buildings of Viña del Mar built before 1985. [10]

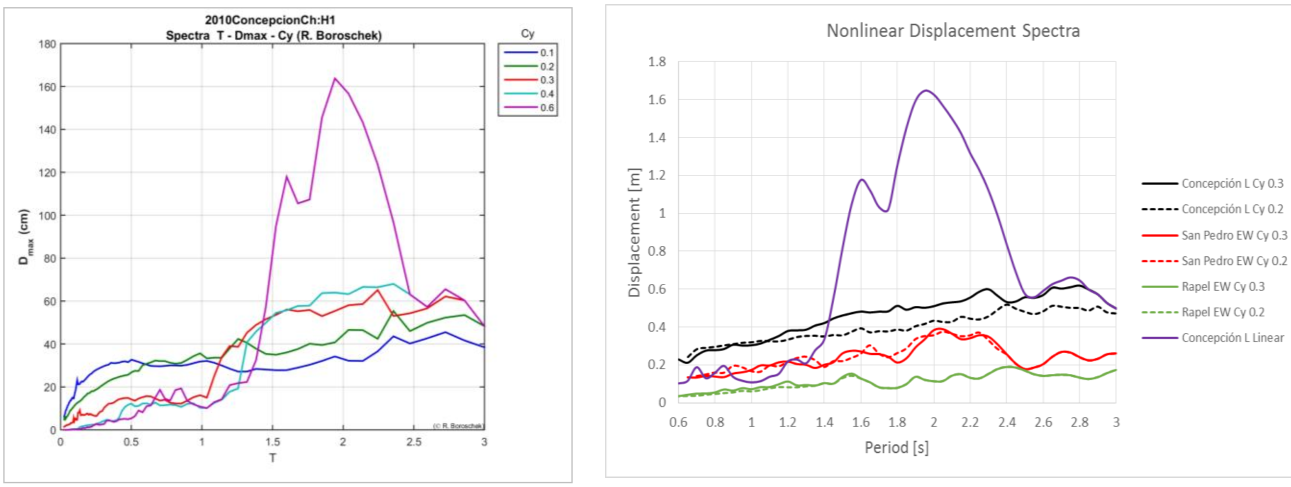
(b) V_i = Ideal base shear definition.

Fig. 10 – Lateral strength at actual buildings assessment for Viña del Mar buildings after 1985 Chile Earthquake.

4.1 Lateral strength effect on displacements.

Fig.11(a) shows nonlinear spectra obtained for Concepción Centro L record and yield strength to seismic weight ratios of 0.1, 0.2, 0.3 and 0.4, using Clough hysteresis rule [11] without post yield hardening (i.e. r = 0, where r is the post yielding stiffness ratio) and a damping ratio of 2%. It can be observed that between 0.5 and 1.2 seconds the spectral displacement strongly depends on the yielding force level, with values in order of 350 mm for C_y = 0.2, and 15 centimeters for C_y=0.3.

Nonlinear displacement spectra computed for Concepción Centro L, San Pedro EW and Rapel EW records for yield strength equal to 20 and 30 percent of the weight and post yield hardening ratio r=0.05 are shown in Fig.11(b). The ordinates of the nonlinear spectra depend on the hysteretic rule and the parameters considered. The soil amplification effect is important in Concepción record, with displacements as large as 600 mm for periods between 1.6 and 2.5 seconds, whereas in San Pedro displacements are not greater than 400 mm. For periods larger than 1.5 seconds, the greater the yield strength, the larger the spectral displacements. In this range a resonance response will be expected due to Concepcion soil properties.



(a) Displacement Spectra for Concepción. r=0.

(b) Envelope of Displacement Spectra for Concepción EW, San Pedro EW and Rapel EW. r=0.05.

Fig. 10 – Inelastic displacement spectra for Rapel EW, San Pedro and Concepcion Centro records for C_y = 0.20 and 0.30. ξ = 2% and Clough hysteretic model.

Most of the damaged buildings in Concepción and Viña del Mar had a natural period near to one second [12]. For C_y = 0.3 and r=0 displacement spectra ordinates are around 200 mm, then if the maximum lateral displacements at roof should be approximately 300 mm, considering $\delta_u = 1.5\,Sd$ [13].

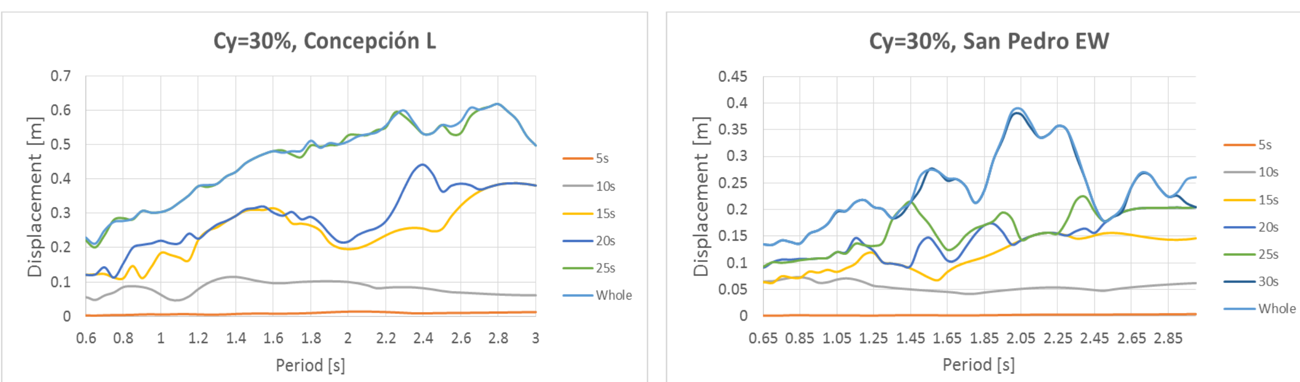
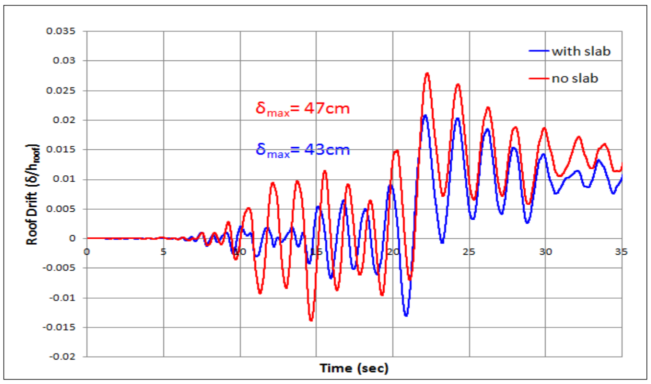


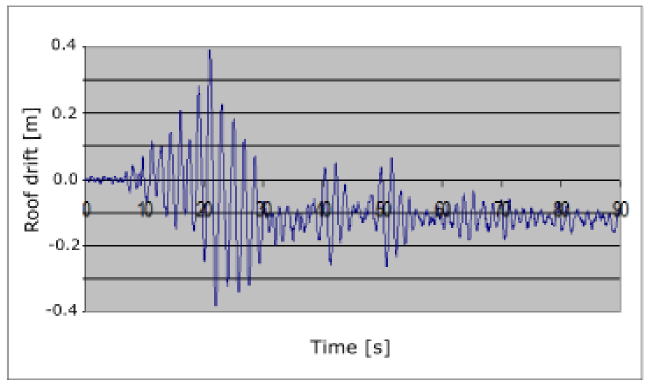
Fig. 11 –Inelastic displacement spectra for different earthquake durations in Concepcion Centro L and San Pedro EW records for a yield strength equal to 30% of weight .ξ = 2%. r=0.05.

Fig.12 shows displacement spectra for the Concepción L record computed for different duration of ground movement and for C_y = 0.3. The set of motions include the first 5, 10, 15, 20 and 25 seconds of the record, as well as the entire ground motion. According to the results, a single degree of freedom (SDOF) system needs at least 25 seconds from initiated the earthquake to reach the maximum lateral displacement. This result shows that the maximum response would occur after the time of arrival of the seismic pulse from the asperity, Fig. 8. For large structural periods the maximum displacement is reached between 20 and 25 seconds after initiated the earthquake. The maximum response with San Pedro is obtained after 30 seconds of the ground motion.

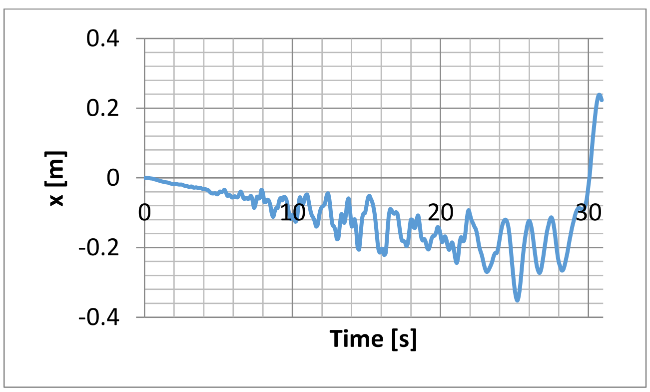
Several authors have studied the only building that collapsed in Concepción, Alto Rio. Fig.13 shows the lateral roof displacement of the segment of the building. Wallace using Perform 3D [14] Alarcon [15] using Ruaumoko and Wilford using LS Dyna [14], obtained similar results concluding that lateral displacement at the roof level might have been of the order of 400 mm, reached before 20 seconds after initiated the earthquake. For a natural period of one second, the spectral displacement is only 200 mm, which means that the amplification factor in these case should be closer to 2.0 instead of 1.5.



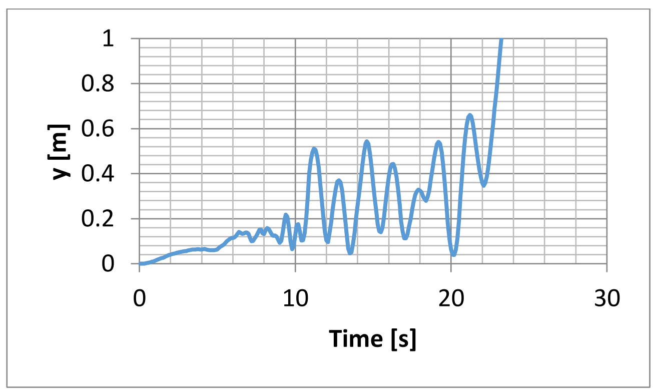
(a) Wallace [14]



(b) Alarcón [15].



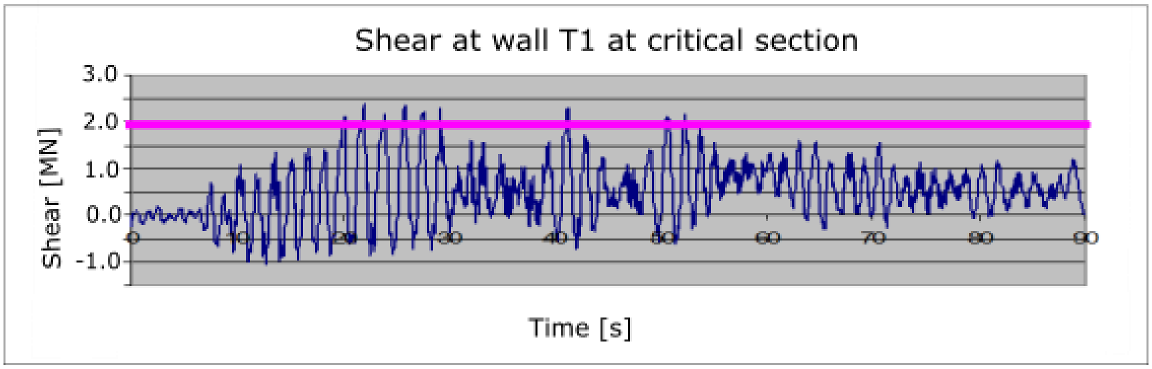
(c) Longitudinal displacement, Willford [14]



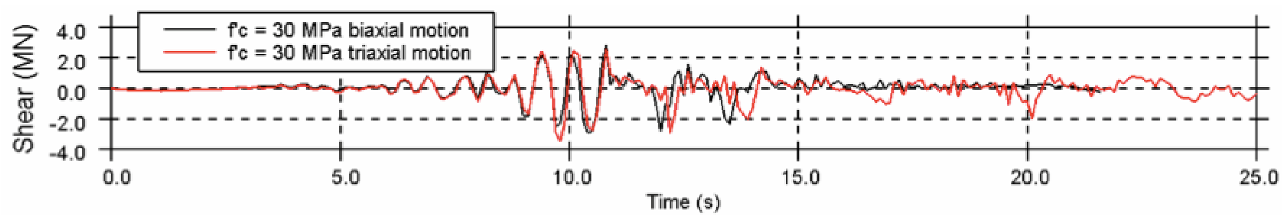
(d) Transversal displacement, Willford [14]

Fig. 12 – Lateral roof displacement for Alto Rio building (Concepción).

Buildings with damage in walls showed to be very brittle behavior. Alarcon concluded that the failure in walls was initiated by a shear failure due to high diagonal compression, and that would it have occurred after 20 seconds from initiated the ground motion, Fig.14(a). Results shown in Fig.13(b) and Fig.15(a) are valid for a ductile response, since that analysis can not consider the response after a brittle failure. Analsys with LS Dyna performed by Wilford reproduced the instant of failure, Fig.13(c), Fig.13(d) and Fig.14(b).



(a) Alarcón, Ruaumoko



(b) Wilford, LS Dyna

Fig. 13 – Shear in a wall of Alto Rio building (Concepción). [15]

4. Drift demands

Assuming that the natural period (***T***) for a wall buildings in Chile is about 1/15 the number of floors (***n***) [16] and, using an interstory height ($\boldsymbol{h_i}$) of 2.6 meters, the total height of the building (***H***) can be estimated as a function of the natural period Eq. (3).

$$T = \frac{n}{15}\,[seconds]; \qquad H = h_i\, n\,[meters]; \tag{2}$$

$$H = 15\, h_i\, T; \tag{3}$$

According to FEMA 273 [14], the lateral drift at the top of a building can be estimated as 1.5 times the spectral displacement for the natural period of the building, Eq. (4).

$$\delta = 1.5\ S_d; \tag{4}$$

Then the global drift**,** $\boldsymbol{\delta/H}$ is then:

$$\frac{\delta}{H} = \frac{1.5\, S_d}{15\, h_i\, T} = 0.1\frac{S_d}{h_i\, T}; \tag{5}$$

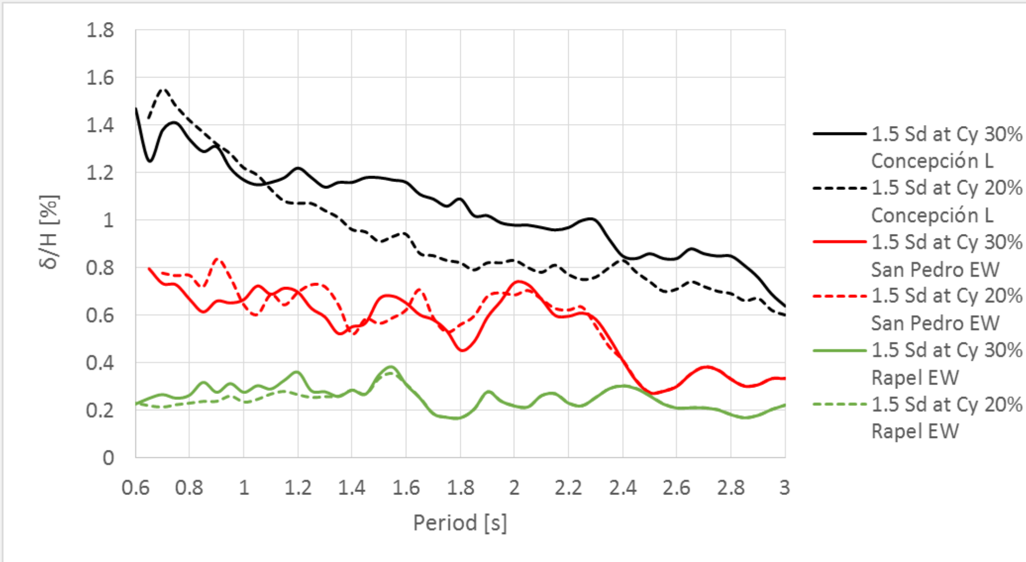


Fig. 14 – Lateral drift at roof level for C_y = 0.2 and 0.3 for Concepción L, San Pedro EW and Rapel EW records. r=0.05.

In Fig.15, the maximum lateral displacement at the roof level divided by the building height is presented as a function of the structural period. Results are similar for different lateral strength of 20 and 30 percent of weight, the order where the ideal base shear is in actual buildings in Chile when walls are used as main structure. When buildings are constructed on firm soils, a response demands similar to those generated by the Rapel record could be expected. San Pedro record represents areas where the rock under the surface of soft soil is around 25 to 40 meters. Lateral drifts would be in this case in order of 0.6 to 0.8 percent of height, a response with low incursions into the nonlinear range with visible cracks, and then it can be classified as immediately occupancy. However, in areas similar to Concepcion Center, where rock depth is greater than 120 meters, large lateral drifts are expected, between 1 to 1.4 percent. For a wall, one percent drift can be related to moderate damage, but 1.4% could be excessive. Precautions must be adopted to limit drifts and supply ductility in these soil conditions.

5. Conclusions

Buildings with significant damage in Concepcion and Viña del Mar had a fundamental period around 1.0 second. In this range of periods, nonlinear spectral displacement ordinates are approximately 350 mm in Concepción L, twice those computed with San Pedro EW record (approximately 200 mm). For Rapel EW record, they are less than 100 millimeters at this range of periods.

The soil amplification at Concepción station was important, with nonlinear displacements demands for typical strength of Chilean buildings as large as 600 mm for periods between 1.6 and 2.5 seconds, whereas for San Pedro record displacements are not greater than 400 mm. In areas similar to San Pedro, lateral drifts in order of 0.6 to 0.8 percent are expected, related to a response with low incursions into the nonlinear range with visible cracks, and then it can be classified as immediately occupancy. However, in areas similar to Concepcion Center where rock depth is greater than 120 meters, large lateral drifts, between 1 and 1.4 percent, are expected. For a wall, one percent drift can be related to moderate damage, but 1.4 could be excessive. Lateral drifts do not depend significantly on yield forces. Precautions must be adopted to limit drifts and supply ductility in these soil conditions.

Buildings with damage in walls showed to be very brittle in an areas where a ductile response was expected. Precautions to control lateral drifts and supply ductility must be adopted in soft soil conditions.

6. Acknowledgements

Rodrigo Thiers, and Dr. Patricio Quintana have made valuable observations. Partial results shown on this paper are part of a thesis for Master degree at Universidad Técnica Federico Santa María Valparaíso of co-author K. Bonilla.

7. References

[1] Comité Inmobiliario (Real State Committee) CChC (2010): Communication based on INE data, Instituto Nacional de Estadisticas (National Institute of Statistics), http://www.ine.cl/.

[2] Lagos R, Kupfer M, Lindenberg J, Bonelli P, Saragoni R, Guendelman T, Massone L, Boroschek R, Yañez F (2012): Seismic performance of high-rise concrete buildings in Chile. *International Journal of High-Rise Buildings*, **1°** (3), 181-194.

[3] Brain I, Mora P (2013): Emergencia y reconstrucción: el antes y después del terremoto y tsunami del 27F en Chile - Aprendizajes en materia habitacional, urbana y de seguros. *Centro de Políticas Públicas UC., Fundación MAPFRE*.

[4] Boroschek R., Contreras V., Kwak, DY, Stewart JP (2012): Strong Ground Motion Attributes of the 2010 Mw 8.8 Maule Chile Earthquake. *Earthquake Spectra*. **28°** (S1), S19-S38.

[5] Poblete M; Dobry R (1968). Modelo dinámico del subsuelo de Concepción. *Revista del IDIEM* **7°** (3), 111-136.

[6] Instituto Nacional de Normalización (INN) (2009): Diseño Sísmico de Edificios. *NCh433.Of.96 Modificada en 2009*, Santiago de Chile.

[7] MINVU (2011): "Requisitos de Diseño y Cálculo para el Hormigón Armado". *Diario Oficial No. 40133, Decreto Supremo Nº60*. Ministerio de Vivienda y Urbanismo, Chile.

[8] Boroschek R, Bonelli P, Restrepo J, Retamales R, Contreras V (2014) Chapter 11: Lessons from the 2010 Chile earthquake for performance based design and code development. *Performanced-Based Seismic Engineering: Vision for an Earthquake Resilient Society*, Matej Fischinger Editor, Geotechnical, Geological and Earthquake Engineering. **32°** (1), 149-157.

[9] Saragoni R., Ruiz S (2012): Capítulo 6: Implicaciones y Nuevos Desafíos de Diseño Sísmico de los Acelerogramas del Terremoto del 2010. *Mw = 8,8 Terremoto en Chile, 27 de Febrero 2010*. Departamento de Ingeniería Civil, Facultad de Ciencias Físicas y Matemáticas, Universidad de Chile.

[10] Bonelli P (1989): Control de desplazamiento lateral para mitigar los daños de terremoto. Libro Anales de la Universidad de Chile, Estudios en honor a Rodrigo Flores Alvarez. **5°** (21), 375-409.

[11] Clough RW (1966): Effects of stiffness degradation on earthquake ductility requirement. *Report No. 6614,* University of California, Berkeley, CA.

[12] Bonelli P (2014): Surface ground motions, structural damage and emerging technologies in R/C buildings during and after the 2010 Mw 8.8 Maule Earthquake, Chile. *RC Structures Workshop*, Structural Engineering Research Center, Tokyo Institute of Technology.

[13] FEMA 273, (1997), NEHRP Guidelines for the Seismic Rehabilitation of Buildings, prepared by the Building Seismic Safety Council for the Federal Emergency Management Agency, 1997, Washington, D.C. (*FEMA Publication No. 273*).

[14] NIST (2014) Recommendations for Seismic Design of Reinforced Concrete Wall Buildings Based on Studies of the 2010 Maule, Chile Earthquake, *NIST GCR 14-917-25*, prepared by the NEHRP Consultants Joint Venture, a partnership of the Applied Technology Council and the Consortium for Universities for Research in Earthquake Engineering, for the National Institute of Standards and Technology, Gaithersburg, Maryland.

[15] Alarcón G (2012): Comportamiento ante cargas sísmicas de un sistema estructural conformado por muros de hormigón armado de sección T, acoplados con losas. *Memoria para optar al título de Ingeniero Civil. Universidad Técnica Federico Santa María.*

[16] Boroschek R, Yáñez, F (1996): Correlation of Code Design Recommendation for Shear Wall Buildings Base on Observation of Actual Earthquake Response Records. *XI World Conference on Earthquake Engineering, Acapulco, México.*