PERFORMANCE OF STEEL STRUCTURES DURING THE FEBRUARY 27, 2010, CHILE EARTHQUAKE

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SUMMARY:
The earthquake of February 27, 2010 was the latest of a number of large seismic events that have affected Chile throughout its history. The damage on structures and the related life and economic losses were limited. In general, steel structures performance was superior to the expected performance stated by modern seismic resistant design codes, despite the fact that the current Chilean seismic codes do not fully consider the latest steel seismic provisions available. This paper presents an overview of field observations after the earthquake, with a brief description of the type of damage observed and the possible causes. Based on these observations, it is postulated that the adequate performance of steel structures is related to the significant overstrength of the structures rather than a large ductility capacity, which is a consequence of the application of the current seismic design codes.

Keywords: Ductility, Seismic performance, Response modification factors.

1. INTRODUCTION

Several major earthquakes magnitude 8.0 and above have affected different regions of Chile over the last 100 years. The giant earthquake of 1960 in the south of the country, the 1985 earthquake in the central region, and the February 27, 2010, Maule earthquake affected populated and industrialized areas. However, the latter is the first large earthquake that has put to the test different types of structures designed according to modern seismic design codes. Moreover, the large affected area, the concentration of urban and industrial constructions in this area, and the fact that most design provisions used for these structures are based on worldwide accepted specifications, make the observations of the performance of structures affected by the earthquake relevant to researchers and code writers everywhere.

The observed damage is similar to that reported after the earthquakes of 1960 by Steinbrugge and Flores (1963), and was mainly due to improper anchorage of equipment, differential movements between adjacent supports of piping and equipment, and foundation movements. There was little, if any, evidence of large ductility demands and damage to most of the structures supporting equipment; but significant damage was sustained by non-structural elements and equipment.

This paper summarizes the observations of the authors collected during several visits to affected regions, shortly after the earthquake. Access to many of the structures was difficult, especially those part of industrial facilities, but the sample is large enough to draw some general conclusions regarding the performance of steel structures during the Maule earthquake of 2010.

2. THE 2010 MAULE EARTHQUAKE

On February 27, 2010 at 3:34 a.m. local time, a magnitude 8.8 (Mw) earthquake struck the central part of Chile. The rupture occurred in the contact between the Nazca and the South American plates and had an approximate extension of 450 km in the North-South direction. This subduction-type earthquake affected an area of approximately 160,000 km2, which houses approximately 75% of the population of Chile. Fig. 1 shows a map of the affected area, the approximated rupture zone, and some of the recording stations closest to the epicenter of the National Network of Accelerographs of the Department of Civil Engineering of the University of Chile (Boroschek et al 2010). This Figure also shows the intensities estimated by Astroza et al (2010) using information collected from several field trips to the affected area, regarding damage to adobe, clay, and concrete masonry structures. This figure illustrates the extent of the area that suffered significant damage due to the earthquake and gives an insight into the performance that should be expected for structures in this area.

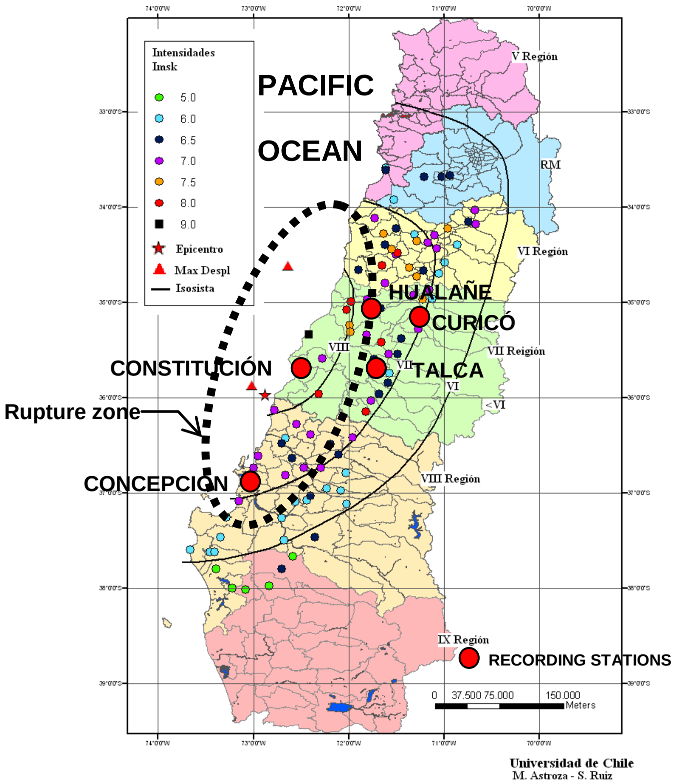

Figure 1. Area affected by the Maule earthquake, intensities, and curves of equal intensity determined by Astroza et al. (2010)

3. CODE PROVISIONS

There are currently three seismic codes in effect in Chile: NCh433 Seismic resistant design of buildings (INN, 2009), NCh2369 Seismic resistant design of industrial facilities (INN, 2003), and NCh2745 Seismic resistant design of base isolated structures (INN, 2006).

The seismic design of industrial facilities is governed by the Chilean code NCh2369 (INN 2003). This specification aims to achieve two performance levels, Life Safety and Continuity of Operations, for the most severe earthquake expected in a region. Chile is divided into three seismic regions, where the earthquake hazard decreases from the Pacific coast to the Andes Mountains. The seismic demand is characterized by a design acceleration value or a seismic coefficient according to the seismic region, which is modified considering the soil conditions and the inherent equivalent viscous damping, deformation capacity, and overstrength of the structure. In the case of steel structures, given the lack of an official national design code, NCh2369 (INN 2003) contains design considerations largely based on the AISC LRFD Specification (AISC 1999) and the AISC Seismic Provisions (AISC 2002), complemented with recommendations based on the experience of Chilean structural design offices.

Fig. 2 shows the response spectra of the horizontal components of ground acceleration, for 2% equivalent damping ratio, recorded at five sites shown in Fig. 2 within the affected area, namely: Hualañé, Curicó, Constitución, Talca, and Concepción. These spectra are compared to the design spectrum established in NCh2369 (INN, 2003), considering no modification of the response (R = 1) and the same equivalent damping ratio. It can be seen that, with the only exception of the record from Concepción, the recorded ground motion response spectra for periods larger than one second are comparable to the design level earthquake considered by NCh2369; therefore, continuity of operation should be expected for industrial structures in this area. The particular shape of the record from Concepción is related to the unfavourable soil conditions in this city, which sits on fluvial deposits of the Bio-Bio river delta. This record is particularly damaging for flexible structures, imposing a non-decreasing displacement demand for structures with longer natural periods.

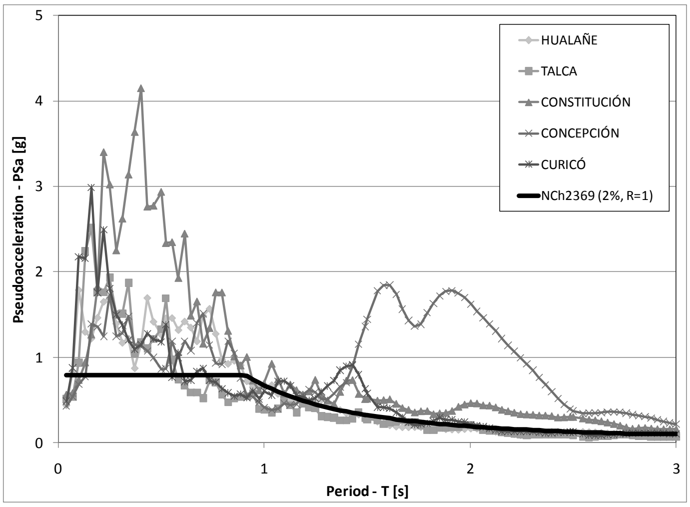

Figure 2. Response spectra of recorded ground motions and design spectrum from NCh2369.Of2003, for 2% equivalent viscous damping

4. STEEL STRUCTURES AND THE FEBRUARY 27, 2010, EARTHQUAKE

The vast majority of steel structures in the country are industrial facilities. Therefore, the discussion will be centered on the performance of this type of installations.

4.1. Description of the industrial facilities in the affected area

The affected area not only houses nearly 75% of the country's population, but also a significant part of the non-mining related industrial activities. According to a report by the Economic Commission for Latin America and the Caribbean (CEPAL for its initials in Spanish), the most affected regions concentrate 52,5% of the Agriculture and Forestry production, 22,7% of the Fishing industry, 27,8% of the manufacturing, and 38,4% of Electricity, Gas, and Water industries, as shown in Table 4.1.

The industrial stock of the Valparaíso, Metropolitan of Santiago, O'Higgins, Maule, and Bío Bío regions includes: six cellulose pulp mills; tree paper mills; two steel mills; two cement plants; fish flour production facilities, two glass factories; fuel refineries and storage facilities; coal-fired electric plants; freight ports; mineral processing plants; wine production facilities; food production plants; etc; i.e., a representative sample of modern and not-so-modern industrial facilities

Most industrial buildings are structured with concentrically braced frames (CBF), moment resisting frames (MRF), or a combination of both systems along perpendicular directions. CBFs can be classified between intermediate and ordinary, according to the AISC Seismic Provisions (AISC, 2005), while MRFs are closer to the intermediate category, according to the same provisions. Therefore, the available ductility of these systems is limited, but this weakness is compensated by a significant level of overstrength stemming from the application of NCh2369 (INN, 2003).

Other structural systems, part of the industrial stock in the area, are tanks, silos y storage racks.

Table 4.1. Industrial production by sector in the affected regions (CEPAL, 2010)

Sector	O'Higgins Region [a]	Maule Region [a]	Bío Bío Region [a]	Total[b]
Agriculture-Forestry	20.9	16.0	15.6	2.3
Fishing	0.0	0.1	22.5	0.3
Mining	5.1	0.3	0.5	0.5
Manufacturing	2.8	4.6	20.4	5.1
Electricity, gas and water	4.6	13.8	20.0	1.2
Construction	5.6	4.2	10.3	1.5
Retail, restaurants and hotels	4.8	2.0	4.5	1.3
Transportation and communications	3.2	3.98	8.1	1.6
Financial and entrepreneurial services	1.6	1.58	4.5	1.3
Housing properties	3.0	3.6	8.4	0.9
Personal services	2.7	4.2	11.0	2.2
Public administration	3.2	3.9	10.2	0.8

[a] Percentage of national GDP per sector
[b] Percentage of total national GDP

4.2. Aggregated effects

Despite the restriction to access many industrial facilities and observe the damage, the effect of the earthquake on the industrial facilities can be indirectly assessed. Fig. 3 shows a pronounced drop in the industrial production post-earthquake in the affected industry (blue line), in contrast with the industry not affected (red line), according to statistics of the National Institute of Statistics (INE, 2010). A year later, the affected industry is barely reaching the levels of the not-affected industry, due to the "still damaged production capacity, mainly of the glass and cellulose plants" (INE, 2010). According to the same agency, el industrial production index of the Bio Bio region registered a decrease of 31,2% during the first trimester of 2010, influenced by the effects of the earthquake on the cellulose, steel, oil refining, and fishing industries.

A similar trend can be observed when looking at the industrial electrical consumption. Considering the industries and population in the affected area, it follows from the reduction in electrical consumption that nearly one third of the industries were not functioning for weeks or even months.

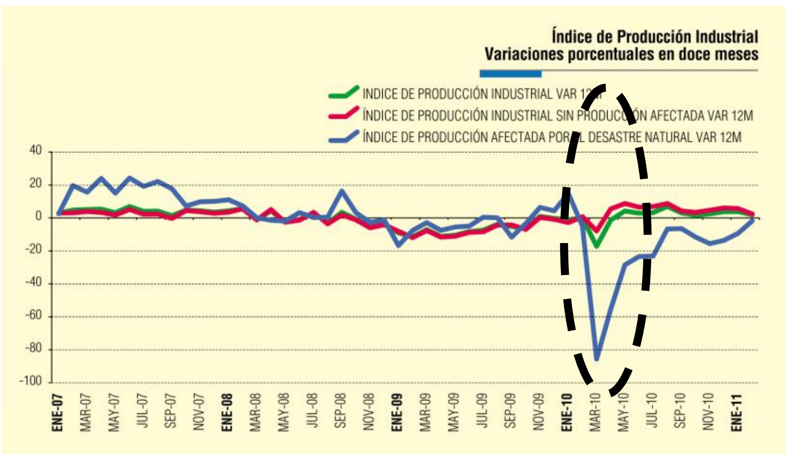

Figure 3: Effects of the earthquake on the industrial production (INE, 2010)

4.3. Performance of industrial structures

The description of the effects of the earthquake observed on the field is reported here by industrial facility type, mostly for facilities with building-like structures. A more detailed description including other facilities can be found in Herrera et al (2012).

4.3.1. Mining facilities

The earthquake did not particularly affect the mining industry, given that most of the mining activity takes place in the Andes Mountains in the north part of the country.

4.3.2. Pulp and paper plants

In general, no significant damage was observed in the structures. Most problems occurred in structures older than 20 years, where the structural members had very limited ductility. Typical damage observed was buckling of braces and fracture of gusset plates (Fig. 4). Some industrial buildings had permanent deformations due to the brace buckling and the lack of redundancy. Some failures of conveyor belts due to pedestal damage (Fig. 5) or partial collapse of an intermediate support structure were also observed. Fractured anchor rods could be observed at the base of columns and mechanical equipment, as well as the spalling or crushing of the leveling grout or the concrete underneath base plates. Other common damage observed was the buckling of shell structures, such as chimneys and tanks. However, most structures and building designed according to NCh2369 (INN, 2003) had light or no damage. The most significant structural damage occurred in a boiler building, where the seismic stoppers could not contain the displacement of the boiler, leading to the impact of the equipment with the structure.

(a) Brace buckling (b) Brace-to-gusset connection fracture

Figure 4: Brace failures

Figure 5: Stretching of anchor rods and pedestal failure

4.3.3. Steel mills

Localized severe damage was observed at the Huachipato steel mill, near Concepcion. Fig. 5 shows some of the problems on this industry, which was one of the most severely affected facilities. Nonetheless, in the analysis of the damage, consideration must be given to the fact that this installation is a very large, old facility with a large number of structures, located close to the epicenter and which has sustained events such as the 1960 earthquake of the South of Chile (Steinbrugge y Flores, 1963).

(a) Conveyor belt collapse

(b) Brace fracture

(c) Buckling of chimney wall

(d) Stretching and fracture of anchor rods

Figure 5: Steel mill damage

4.3.4. Wine production facilities

The wine production in Chile is mainly stored in stainless steel tanks. A large portion of the losses was due to the wine spilled to the ground because of damage to stainless steel tanks and storage elements for bottled wine such as racks, bins and barrels.

Stainless steel tanks were either supported on legs or on concrete footings. Legged tanks are usually used to store up to 50,000 l of the highest quality wines. Damage observed on these tanks is shown in Fig. 6 and includes: buckling of legs, buckling of tank wall around the leg, failure of the height regulation mechanism. When the legs were not anchored to the floor, normally the tank would topple over after one leg fell into a gutter u other floor irregularity.

(a) Leg instability

(b) Strong leg-weak wall failure

Figure 6: Damage on legged tanks

Tanks on grade are normally used for volumes over 50,000 l reaching capacities of over 650,000 l in some cases. The damage observed on these tanks includes: tank wall buckling (either “elephant foot” or “diamond shaped”, shown in Fig. 7), anchorage failure, separation of the bottom of the tank from the walls and fracture of valve and fitting connections to the tank wall.

(a) “Elephant foot” buckling (b) “Diamond shaped” buckling

Figure 7: Tank wall damage

4.3.5. Light steel framing structures
Most of these structures were made of cold formed light gauge steel, and despite the limited ductility of these elements, withstood successfully the ground shaking. Light steel structures, such as the fish processing facilities in Talcahuano shown in Fig. 8, were affected by the tsunami.

Figure 8: Damage on fish processing plants in Talcahuano.

4.3.6. Small equipment and non structural elements
The largest proportion of losses was caused by damage to non structural elements, equipment, and building contents. The failure of these elements affected from the habitability of houses and apartments to the operation of several industrial facilities and the Santiago Airport (capital city of Chile). The level of damage observed was due to the insufficient detailing and installation practices, in terms of seismic resistance.

5. ANALYTICAL STUDIES

One industrial building designed according to the current specifications was modelled and analyzed under different earthquake records.
The mill building was not directly affected by the earthquake, because it is located in the north part of the country, but it is a structure representative of the industrial buildings designed in Chile and can, therefore, give an insight on the causes of the performance achieved by these structures. The building’s lateral load resisting systems are concentrically braced frames, as shown in Fig. 9. It has a

height of approximately 21.5 m with a larger floor area in the lower two stories, which are 3.25 m high each. The material is steel ASTM A36 and beams and columns are built up I shapes, while braces are either built up I shapes or XL shapes (starred angles). The dimensions of the structural members are shown inf Table 5.1. More details on this structure can be found in Astica (2012).

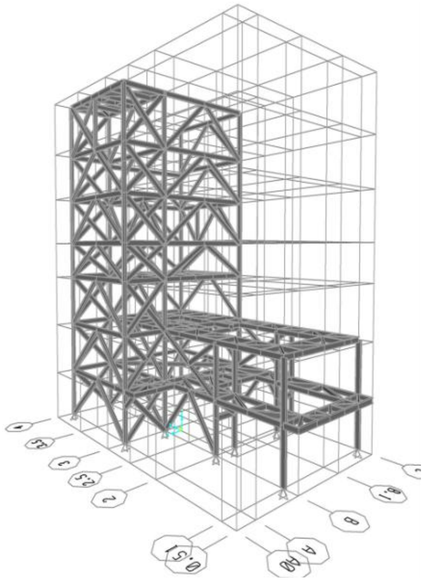

Figure 9: Industrial mill building.

Table 5.1. Industrial building member dimensions

	Shape	H (mm)	B (mm)	t_f (mm)	t_w (mm)	A (mm^2)	I_x x10^{-6} (mm^4)	I_y x10^{-6} (mm^4)	J x10^{-4} (mm^4)
Columns	HE250x42,4	250	250	8	6	5404	64,99	20,84	10,28
	H300x73,9	300	300	12	8	9408	163,40	54,01	39,48
	H300x127,7	300	300	22	12	16272	272,35	99,04	228,97
	H300x105,5	300	300	18	10	13440	230,34	81,02	126,04
	H250x68,9	250	250	14	8	8776	104,88	36,47	49,76
	HE200x33,8	200	200	8	6	4304	32,62	10,67	8,21
Beams	H200x150x30,6	200	150	10	5	3900	29,53	5,63	10,79
	H300x200x56,8	300	200	14	6	7232	124,67	18,67	38,65
	H400x400x159,5	400	400	20	12	20320	624,79	213,39	235,22
	H600x250x106,1	600	250	18	8	13512	881,98	46,90	107,13
	H400x250x80,1	400	250	16	6	10208	320,00	41,67	71,03
	HE250x50,1	250	250	10	6	6380	78,13	26,05	18,39
	H250x150x43,3	250	150	14	6	5532	64,02	7,88	29,14
Braces	HE200x33,8	200	200	8	6	4304	32,62	10,67	8,21
	XL80x80x6x7,0	80	80	6	6	888	1,62	0,51	1,68
	XL100x100x8x11,6	100	100	8	8	1472	4,16	1,34	4,98

The model of the building considered material and geometric nonlinearities, using distributed-plasticity, fiber elements for beams and columns and a model proposed by Uriz et al (2008) for the braces, modified to adjust for the slenderness ratios and shape of the members of the structure. The model was subjected to a series of ground motion records recorded during the February 27, 2010 events and previous large magnitude events. The response spectra of these records is shown in Fig. 10 together with the design spectra for soil Type I and seismic zone 3 of NCh2369 (INN, 2003).

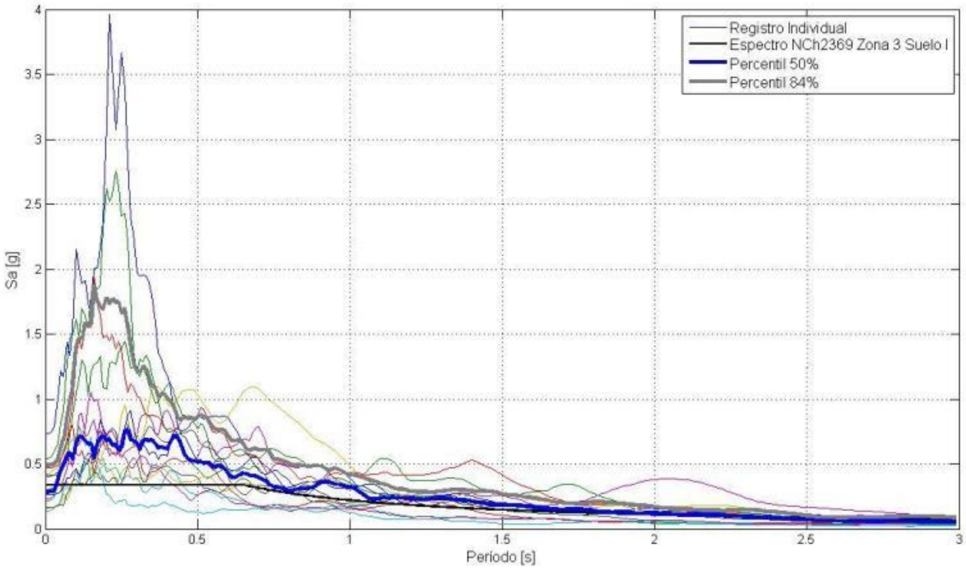

Figure 10: Response spectra of records used for the non linear time history analyses.

The model was first subjected to the natural records set, resulting in linear elastic response for all records. Next an incremental dynamic analysis was performed with the same records, in order to obtain the collapse of the structure. The response elastic response was the determined for the amplified collapse level records, in order to obtain the ductility factor (Rμ). The overstrength factor (Ω) was obtained by comparing the collapse capacity of the structure with the design base shear. The summary of the results for each record are presented in Table 5.2.

Table 5.2. Ductility R_{μ} and Overstrength Ω, factors obtained from IDA

STATION	DIRECTION	R_{μ} Dir. X	R_{μ} Dir. Y	Ω Dir. X	Ω Dir. Y
Pica	PICA_NS	1,01	1,38	5,11	7,80
	PICA_EW	0,99	1,35	6,07	7,81
Iquique 1-Etna	IQUI_EW	0,98	1,18	4,51	6,90
	IQUI_NS	0,99	1,78	5,31	6,98
Mejillones	MEJI_EW	1,00	1,08	4,67	6,08
	MEJI_NS	1,01	1,18	4,74	5,63
Puente Alto	PTEALTO_EW	1,00	1,03	5,02	6,56
	PTEALTO_NS	0,98	1,07	4,70	8,30
Hosp. Curicó	HCUR_EW	1,02	1,04	5,79	8,44
	HCUR_NS	1,06	1,67	6,17	7,62
Valparaíso	UTFSM_NS	1,07	1,81	7,56	8,95
	UTFSM_EW	1,16	1,29	8,76	11,94
U. de Chile Ed. Civil	FCFM_EW	1,00	1,09	3,90	7,02
	FCFM_NS	1,00	1,25	4,38	6,87
Average		0.98	1.03	4.38	5.63

The average values of these factors for each direction of analysis are also shown in Table 5.2. From these results, it is clear that this building has a large overstrength, but a limited ductility, confirming the conclusions drawn from the experimental observations.

6. CONCLUSIONS

The damage to steel structures observed in the aftermath of the Mw = 8.8 earthquake that struck Chile on 27 February 2010 can be considered to be minor from the structural standpoint. There was little evidence of significant inelastic demands on most structural members, few collapses of modern, properly engineered structures, and little or no loss of life associated with these failures. On the other hand, damage to equipment was extensive due to poor anchorage and bracing. The losses due to business interruptions were very large, particularly for large plants in the Concepción area, which were out of commission for several weeks (and in some cases months).

The adequate performance of steel structures can be attributed to the large overstrength that results from the application of the seismic design code.

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