Analysis of Damages and Behavior of the Masonry Buildings Due to February 27th 2010 Chilean Earthquake.

P. E. Alcaino, C. Castro & N. K. Valdebenito
School of Civil Engineering. Pontificia Universidad Catolica de Valparaíso, Valparaíso, Chile. E-mail: pablo.alcaino@ucv.cl

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SUMMARY:
Approximately 2/3 of Chilean people live in masonry buildings, which are regular, symmetric, rigid and low rise structures based on shear walls with a very simple design. Since past earthquakes, many lessons have been learned and the Chilean codes improved. However, due to 2010 Chilean earthquake, in some neighbourhoods, were observed buildings without damages and others identic and nearby buildings with severe and brittle damages.
This study included 218 buildings of 19 neighbourhoods on 6 cities. A statistical analysis of damages and their correlation with age, floor's number, materials and local geotechnical conditions was developed; the design of 5 types buildings were checked considering the Chilean codes. Nonlinear analysis were made including the bending and shear behaviour and using 20 acceleration 2010 Chilean earthquake's records. The main conclusions of this study were that the design and the nonlinear analysis can explain the damages or non-damages in buildings.

Keywords: Chilean Earthquake, Masonry walls, Brittle Failure, Damages.

1. INTRODUCTION

Approximately 2/3 of Chilean people live in masonry buildings, due to the masonry construction is cheaper than reinforce concrete or steel construction. The main characteristics of those masonry buildings are that they are regular, symmetric, rigid and low rise structures (no more than seven floors high) based on shear walls, which have a very simple design: since March 03 earthquake of 1985, located close to Viña del Mar (centre coast of Chile), the Chilean engineers and the government collected the past experience (since 1939 Chillán earthquake to 1985 Viña del Mar earthquake) (Hidalgo, P. 1989) and they've developed design codes for confined masonry (INN, 1993 and INN, 2003) and for reinforced masonry buildings (INN, 1997 and INN, 2003). The both codes are based on allowable stress design (ASD); the confined masonry design code includes mainly the Chilean and Latin-American experience and practices. Another hand, the reinforced masonry design code is more similar to UBC (1982).

However, due to the recent earthquakes, in particular due to the 27th February Chilean earthquake (Mw 8.8), unexpected several damages were observed in a significant number of masonry buildings, mainly damages due to shear stress and "short-columns" effect, even in a few cases, reaching the collapse.

This study started from the initial assessment of structural damages due to the earthquake in terms of severity, extent, and number of affected people: the main objective in this moment was to inspect the low-cost buildings subsidized by the government. A structural engineers team inspected two hundred and eighteen buildings of nineteen different neighbourhoods with reported damages on six severely affected cities by the earthquake and a complete list of observed failure modes was registered. An interesting observation from this preliminary study was that in some neighbourhoods were observed buildings without or with minor damages and others identic and nearby buildings with severe and brittle damages on the walls. Then a more advanced study was developed: a statistical analysis of the damages and their correlation with the age, floors number, materials and local geotechnical conditions of the buildings was made; the design of five types buildings were checked considering the Chilean codes and nonlinear analysis (push over) on those buildings were made including the bending and shear wall behaviour and using twenty acceleration records of the 2010, February 27th Chilean earthquake.

The main conclusions of this work were that the design and the nonlinear analysis can explain the damages or non-damages in buildings under study, because they have a linear and brittle behaviour and the performance points were very close to the collapse level, then, the great difference between the good and bad behaviour of nearby buildings is explained by little differences particular conditions of the each structure.

2. METHODOLOGY

2.1. Assessment of damages in inspected buildings

After the earthquake in six affected cities Cauquenes, Talca, Constitución, Curicó, Rengo and Santiago were made visual inspections over 218 buildings base on shear masonry walls. The relative location of those cities is shown in Figure 1.

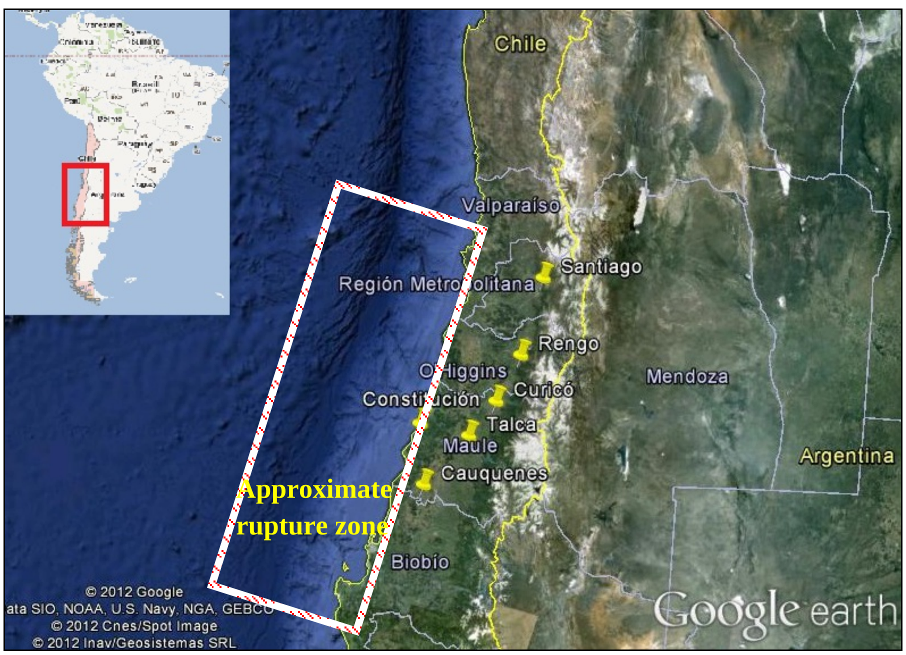


Figure 1. Locations where the masonry buildings were inspected (from Google earth).

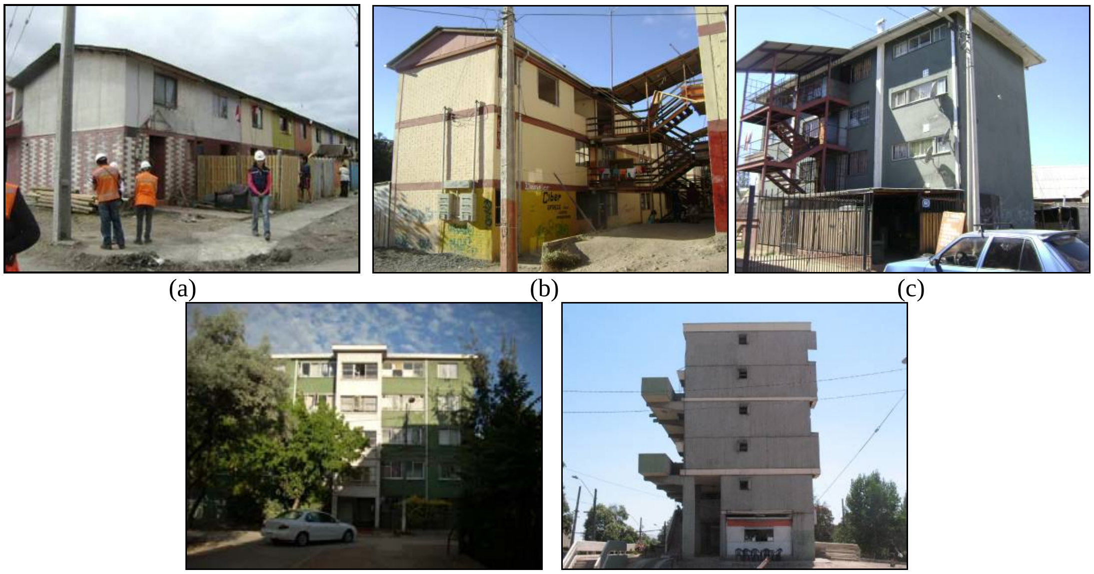
In the Table 2.1, are presented some general aspects of the inspected buildings: location (city), identify code, year of construction number of stories, masonry type, type of masonry unit and number of buildings in each condominium. The photographs of the Figure 2 show some representative buildings for more common high.

The inspection had the propose to identify the structuration, materiality, quality of construction, local geotechnical conditions (for example, superficial groundwater level, closeness of slopes, special topography, evidence of liquefaction, between others) and to assessment the structural damages, considering the failure mechanism, severity, extend, and recurrence. For the assessment of those structural damages three scales of different authors were used considering the equivalence shown in the Table 2.2. In this Table, also is presented a little description of each damage level considered.

Table 2.1. Details of inspected buildings.

Location	ID Code	Construction (year)	# Stories	Masonry type	Masonry unit	# Buildings
Cauquenes	A1	1993	2[(1) (2)]	Confined	Craft clay brick	31
Constitución	A2	1997	2[(1) (2)]	Confined Reinforced	Craft clay brick Hollow clay brick	3
Constitución	A3	1993	2	Reinforced	Hollow clay brick	3
					Subtotal	37
Cauquenes	B1	1993	3	Confined Reinforced	Craft clay brick Hollow clay brick	4
Cauquenes	B2	1997	3	Confined Reinforced	Craft clay brick Hollow clay brick	9
Constitución	B3	1997	3	Confined	Hollow clay brick	10
Constitución	B4	1995	3	Confined	Hollow clay brick	10
Curicó	B5	1994	3	Confined	Craft clay brick	10
Curicó	B6	1996	3	Confined	Craft clay brick	14
Santiago	B7	1960	3	Confined	Concrete block	1
					Subtotal	58
Constitución	C1	1995	4	Confined	Craft clay brick	4
Talca	C2	2003	4	Confined	Hollow clay brick	8
Talca	C3	1967	4	Confined	Craft clay brick	13
Talca	C4	1970	4	RC frames Confined	Craft clay brick	8
Santiago	C5	1960	4	Confined	Craft clay brick	10
					Subtotal	43
Talca	D1	1958	5	Confined	Craft clay brick	4
Talca	D2	1970	5	Confined	Craft clay brick	31
Santiago	D3	1960	5	Confined	Concrete block	6
Santiago	D4	1960	5	Confined	Concrete block	20
Talca	D5	1950	5	RC frames Confined	Craft clay brick	14
					Subtotal	75
Santiago	E1	1950	6	RC frames Confined	Craft clay brick	3
					Subtotal	3
Santiago	F1	1960	7	RC frames Confined	Craft clay brick	2
					Subtotal	2
					Total	218

(1) First story of reinforced masonry and the second story with light materials.
(2) Firewall in the second story of confined masonry.



(a) (b) (c) (d) (e)

Figure 2. Photographs of buildings of (a) 2, (b) 3, (c) 4, (d) 5 and (e) 6 stories.

Table 2.2. Equivalence of used damage scales.

AIS (2002)	Figueroa (2000)		Tomazevic (1999)	
Damage level	Grade	Description	Id	Description
Without damage	0	Without damage	0	Without damage
	1	Thin horizontal cracks in the 50% or more of the base of wall. Rupture of stucco.		
Soft	2	Horizontal cracks across to 100% base of wall. Diagonal cracks are short and thin (e<0.4 mm)	0.25	Formation of first diagonal crack. (70% of high)
Moderate	3	Diagonal cracks between steel bars doesn't affect the compression zone (e<1.5 mm)	0.5	Diagonal network cracks. The maximum strength is developed
Strong	4	Diagonal cracks (e<5mm) affect the compression zone. Failure of some horizontal steel bars.	0.75	Thickness of cracks increase. Partial failure of compression zone (about 50%)
Several	5	Failure of compression zone. Many horizontal steel bars are cut. There have a main diagonal crack.	1.00	Several, non repairable damage Or collapse of the wall.

2.2. Modelling and analysis

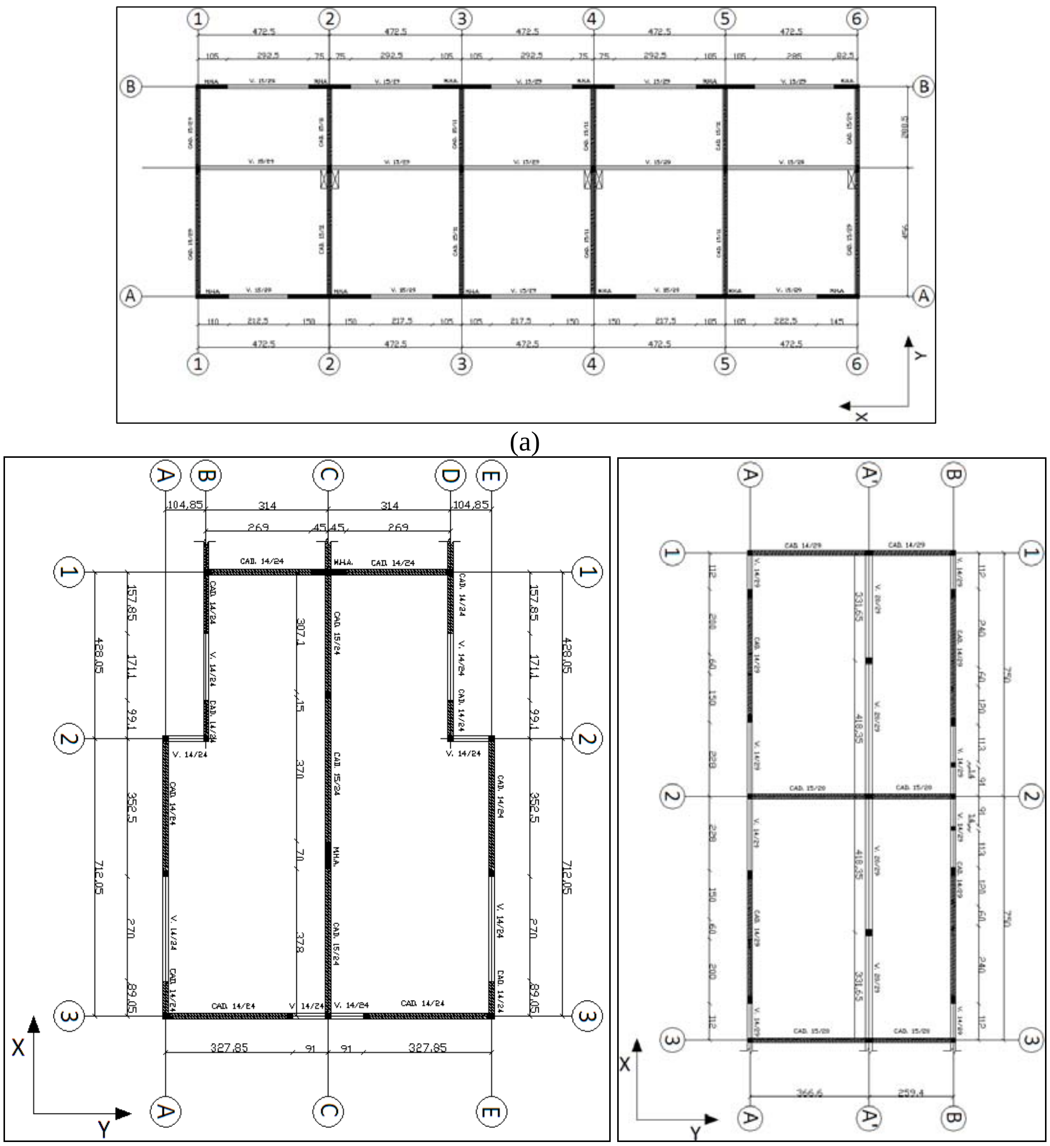
For the modelling of studied buildings, was used the software SAP2000 V14.1. The walls were modelled using frame elements including the shear stiffness. As example, the Figure 3 presents structural plants of buildings. For to include a correct length of the beams, in the extreme of the walls other frame elements without mass and with a great stiffness were used. The Figure 4, shows the extrude view of finite element models of the same three buildings type shown in Figure 3.

The non-linear behaviour was considered in the push over analysis using flexural P-M hinges and shear hinges. For each wall, the flexural P-M hinge was defined from M-ϕ curve, while that the shear hinge was obtained according the tri-linear curve proposed by Alcocer and Flores (2001).

In order to determinate the performance point of each model, were used 20 corrected acceleration record of the 2010, February 27th Chilean earthquake (you can found the records in http://ssn.dgf.uchile.cl/seismo.html) for to construct the demand spectrum. Some characteristic of the used records are listed in Table 2.3.

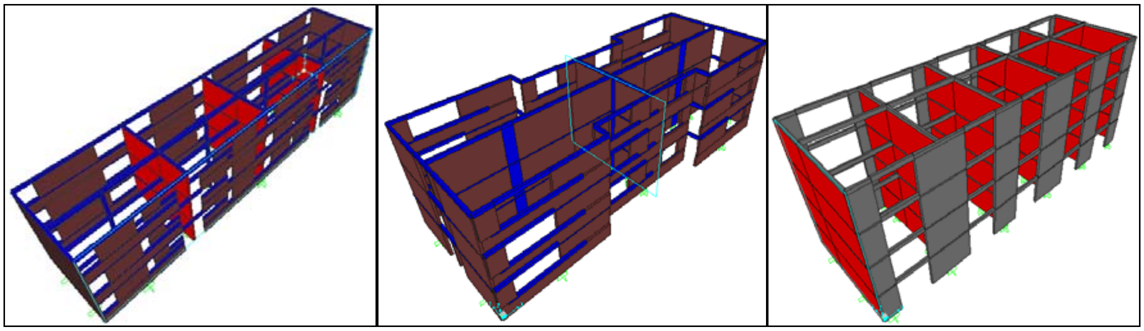
Table 2.3. Some characteristics (PGA) of used acceleration records of 2010, February 27th Chilean Earthquake.

Record	PGA N-S (g)	PGA (E-W)
1 and 2	0.20	0.23
3 and 4	0.23	0.27
5 and 6	0.29	0.33
7 and 8	0.19	0.13
9 and 10	0.24	0.24
11 and 12	0.31	0.23
13 and 14	0.65	0.61
15 and 16	0.57	0.78
17 and 18	0.35	0.25
19 and 20	0.47	0.48



(a) (b) (c)

Figure 3. (a) Building type B1, (b) Building type B3 and (c) Building type C1



(a) (b) (c)

Figure 4. (a) Model of building type B1, (b) Model of building type B3 and (c) Model of building type C1

3. RESULTS AND DISCUSSION

From the first part of this work, the complete list of damages, its possible causes and its correlation with age, number of stories, materiality and local conditions was presented and published in Alcaino and Valdebenito (2011). Based on the failure mechanism observed, the main conclusion in this paper was that the more probable behaviour of Chilean masonry buildings is linear elastic with brittle failure mode. In consequence two identical buildings can have very different level of damages due to little differences in its conditions: for example, Alcaino and Valdebenito (2011) showed that a very local and special geotechnical conditions may be able to determinate the level damage developed by brittle systems, as shown in Figure 5. On the contrary, those conditions which are not unique to each building have not significant correlation with the damage level of these structures, as shown in the graphs of Figure 6.

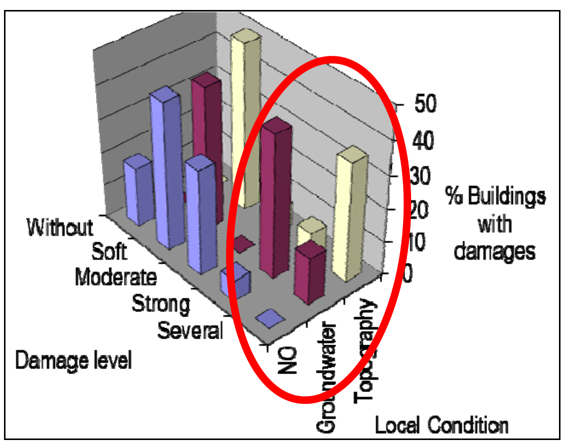


Figure 5. Assessment of influence local geotechnical special conditions in the damage level of buildings.

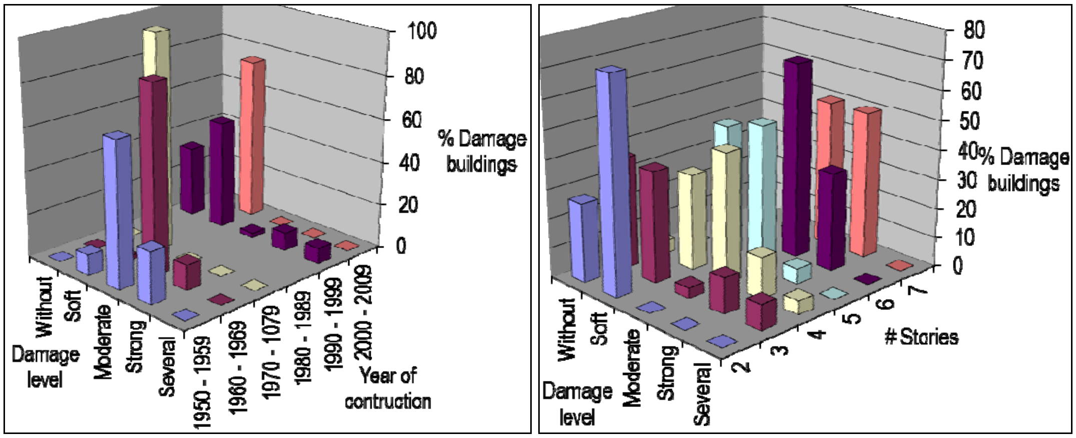
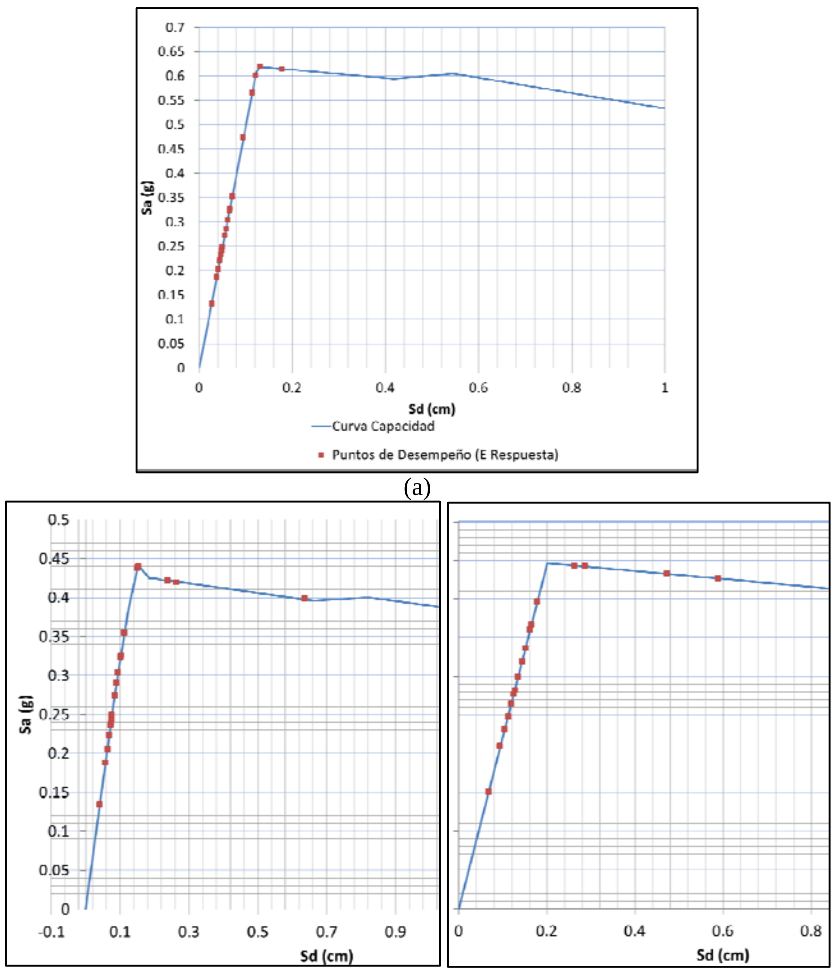


Figure 6. Assessment of influence of no unique conditions in the damage level of buildings.

After, when the design was checked and the push over completed, was possible to verify that the behaviour of the buildings under study is very linear with a brittle failure, as shown in Figure 7. It also indicates the performance point for each demand spectrum. You can observe that many performance points are close to or in the failure.



(a)

(b) (c)

Figure 7. Performance points: (a) B1, x-axis, (b) B3, axis y, (c) C1, axis y

As example and validation of the methodology, the results of the non-linear analysis and the conclusions of Alcaino and Valdebenito (2011), the level of damages predicted by the model was compared with the observed damages. The Figure 8 shows the comparison between the modelled performance point for the building type B1 and its real observed damages. Similarly, the Figures 9 and 10 show the comparison for buildings type B3 and C1.

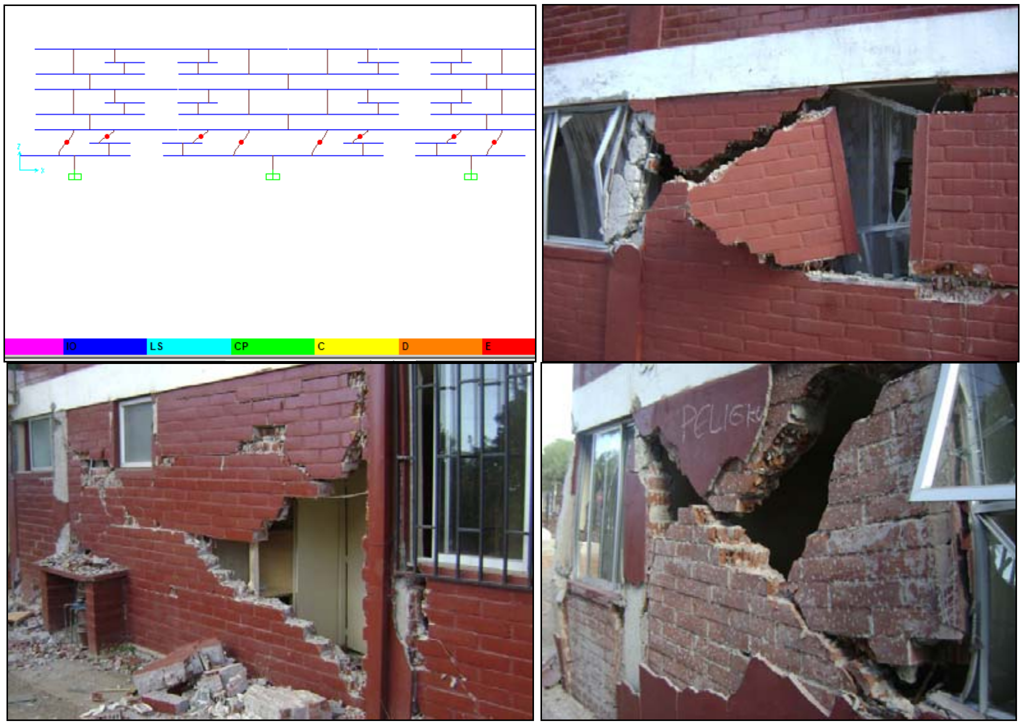


Figure 8. Comparison between the damages predict with the model and the observed damages in building B1.

Figure 9. Comparison between the damages predict with the model and the observed damages in building B3.

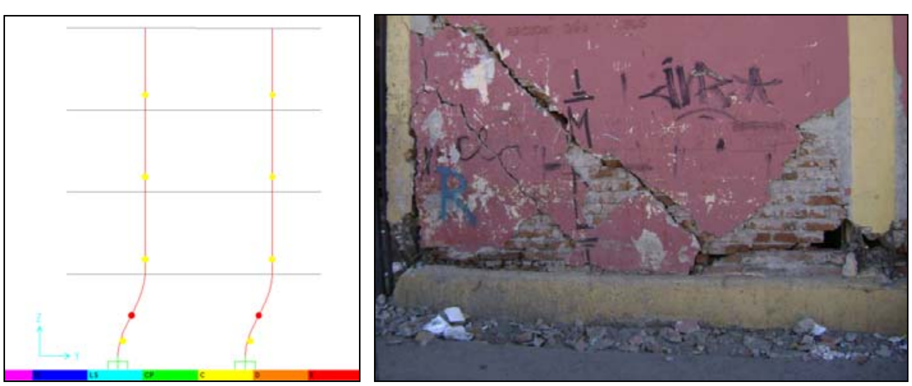


Figure 10. Comparison between the damages predict with the model and the observed damages in building C1.

4. CONCLUSIONS

Considering the design of masonry buildings, a complete analysis of behaviour and the damages due to 2010 February 27th Chilean earthquake was developed. This analysis showed that the masonry buildings, designed according Chilean codes, have elastic and brittle response therefore, two identic buildings may have very different damage levels depending of his particular conditions.

On the other hand, the finite elements model, considering simple models for the flexural and shear hinges, subject to non-linear analysis (push over) was a good predictor of damages.

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