

# Seismic Response of Reinforced Concrete Buildings in Concepcion During the 2010, Maule Earthquake

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## SUMMARY:

This article describes the response of eight shear wall buildings that experienced severe structural damage in Concepcion during the February 27<sup>th</sup>, 2010 Chile earthquake. Damaged elements within the lateral force-resisting systems of the buildings were less than 10%, suggesting little inelastic redistribution. It was also localized mostly in shear walls of lower stories and in the first basement. The most affected structures were rather new buildings founded on soft soils and presenting some degree of vertical and/or horizontal irregularity. Key aspects to understand the observed damage are: geographical orientation, vertical and horizontal irregularities, boundary reinforcement detailing, axial loads, and brittle cyclic behaviour. Geometric and response building indices such as vibration periods and regularity indices were estimated for all buildings. Additionally, four critical structures are subjected to a building-code type analysis and results presented. It is concluded that current shear wall design provisions need to be revised.

*Keywords: Reinforced Concrete, Earthquake Damage, Damage Assessment, Shear Wall, Chile Earthquake*

## 1. INTRODUCTION

The 27 February 2010 Maule earthquake caused extensive damage in about 60 midrise reinforced concrete buildings. In the Bio-Bio Region, which includes the city of Concepción, the estimated number of severely damaged buildings is close to 20. The high concentration of damaged buildings in and around Concepción provides an opportunity to conduct a comparative study of similar buildings subjected to similar ground motions, and dissimilar earthquake response. A specific group of eight shear-wall buildings located in Concepción are considered.

This article presents results obtained from a larger initiative that involved different field visits, detailed damage surveys and stabilization and demolition solutions. The work developed led to extensive information about materials, a thorough survey of the state of internal damage in buildings, analysis of structural drawings and specifications, and development of linear structural models. Space limitations preclude complete reporting of the findings and more details can be found elsewhere (Westenenk et al, 2011).

The seismic performance of some shear wall buildings was certainly unexpected. These buildings, so positively evaluated after the 1985 Chile earthquake (Riddell et al 1992), showed this time larger vulnerability and our hypothesis is that they were not capable of dissipating the energy consistent with the R-factor used in their design. These results are useful because there is little information available in the literature of severely damaged shear-wall buildings. Thus, information presented in this article has an archival purpose also.

The scope of this study is to identify trends in the seismic performance, and to propose an interpretation of the building response. Results of linear response and code-type analysis are presented for 4 buildings that showed critical structural conditions.

## 2. BUILDING INVENTORY AND PROPERTIES

The city of Concepción is 105 km SSW from the epicenter and was considerably affected by the earthquake. The strong ground shaking led to several shear-wall buildings severely damaged. The 8 buildings summarized in Table 2.1 belong to this group.

**Table 2.1.** Reinforced concrete buildings considered

Building	# of stories (above grade + below grade)	Year built	Geographic coordinates	Longitudinal axis	Soil type
AA-1	20+1	2002	36°49'06.98''S 73°02'28.44''W	64°N	II
AH-2	15+2	2009	36°50'12.45''S 73°06'06.97''W	85°W	III
CM-3	18+1	2005	36°49'06.98''S 73°02'41.49''W	62°W	II <sup>(1)</sup>
TL-4	17+1	1973	36°49'42.70''S 73°03'10.62''W	62°W	III
PR-6	12	2006	36°49'14.42''S 73°03'42.90''W	62°W	III
PP-7	10	2004	36°47'46.81''S 73°05'05.77''W	4°N	III
RT-8	10	2006	36°47'44.29''S 73°05'05.67''W	4°N	III
TO-9	21+2	2008	36°49'45.97''S 73°03'18.45''W	62°W	III <sup>(2)</sup>

(1): Original classification was type III; (2): Original classification was type II (Example of soil I is rock, II stiff gravel, and III soft silts)

The eight buildings use reinforced concrete shear walls as their seismic-force-resisting system. The orientation of their longitudinal axis is denoted in clockwise direction from the indicated cardinal point. In the case of CM-3 and TO-9, soil classification results differ from the original classification during building design. Soil type II represents a dense gravel or clay with shear wave velocity larger than 400 m/s in the upper 10 m, and type III is unsaturated gravel or clay with shear wave velocity less than 400 m/s (INN 1996). Figure 2.1 presents representative building plans and photographs of some of the analyzed buildings. More complete information may be found elsewhere (DICTUC 2010).

### 2.1. Building Properties

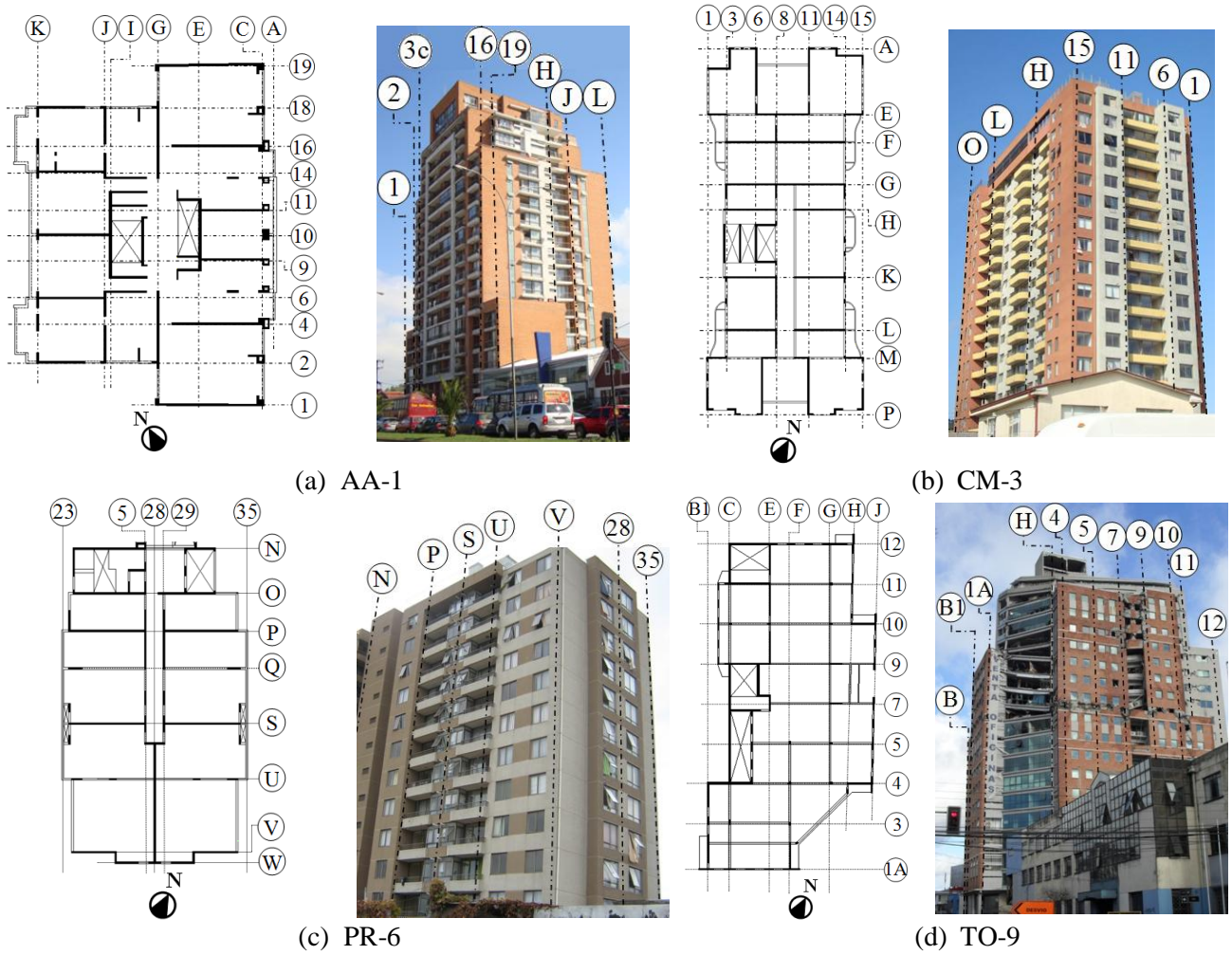
This section summarizes some basic geometric building properties. It is important to note that each of buildings PP-7 and RT-8 are composed by two different blocks each separated by a construction joint, so figures and tables will differentiate them by indices “a” and “b”. The considered building properties were: (i) plan widths  $b_x$  and  $b_y$  of the building in the X- and Y-directions, respectively; (ii) the plan aspect ratio  $b_1/b_2$ , where  $b_1$  and  $b_2$  are the maximum and minimum widths of the floor plan in each story, respectively; (iii) the slenderness ratio  $h/d$ , where  $h$  = building height and  $d = \min(b_x, b_y)$ ; (iv) the story surface  $A$ ; (v) the nominal weighted average wall thickness  $\bar{e}$  (Ref); (vi) the nominal X- and Y-direction densities of shear wall area  $\rho_x$  and  $\rho_y$ ; and (vii) the total density of vertical elements  $\rho_z$ . Table 2.2 summarizes mean values for all these building parameters. Please note that the mean slenderness ratio correlates very well with the observed damage.

Additional vertical and plan irregularities indices have been calculated and are presented elsewhere (Westenenk 2011). Note that all buildings are classified as irregular at least by one of the available indices (SEAOC 1999). These indices reach higher values especially for buildings AA-1 and TO-9.

### 2.2. Dynamic Building Properties

Three-dimensional linear models of each of the study buildings were developed using ETABS (2011). Mass and structural elements dimensions were based on nominal values from building drawings. Core and reinforcement samples were extracted from each building and tested according to ASTM standards to determine material properties. Seismic weight has been considered as indicated by the National Seismic Code (NCh 433 Of.96, 1996), i.e., dead loads plus 25% of the live loads. Table 2.3 lists the first three calculated vibration periods in the X-, Y-, and  $\Theta$ -directions, modal masses in these directions, and the total seismic weight. First modes modal masses are larger than 40%; however,

torsional mass in TO-9 is substantially smaller than for other buildings.



**Figure 2.1.** Typical building plan drawings, building photos and building axes

**Table 2.2.** Summary of average building properties

ID	$\bar{b}_x$	$\bar{b}_y$	$\bar{b}_1/\bar{b}_2$	$\bar{h}/\bar{d}$	$\bar{A}$ (m <sup>2</sup> )	$\bar{\rho}_x$	$\bar{\rho}_y$	$\bar{\rho}_z$	$\bar{e}$ (m)
AA-1	22.7	24.3	1.20	2.73	488	3.57	2.48	6.15	0.19
AH-2	26.1	33.9	1.34	1.87	848	2.44	2.34	4.88	0.20
CM-3	41.2	18.7	2.23	2.95	691	2.57	2.39	4.96	0.20
TL-4	19.9	19.9	1.03	2.61	332	4.09	3.68	7.77	0.27
PR-6	14.2	24.2	1.70	2.42	294	2.32	2.73	5.05	0.15
PP-7a, RT-8a	26.3	14.4	1.83	1.94	267	2.88	2.59	5.47	0.15
PP-7b, RT-8b	21.8	14.4	1.52	1.69	255	2.67	2.51	5.18	0.15
TO-9	24.5	44.5	1.83	3.25	855	1.95	2.91	4.90	0.24

**Table 2.3.** Dynamic properties of the analyzed buildings

ID	$T_x$	$T_y$	$T_\theta$	X-direction modal mass (%)	Y-direction modal mass (%)	$\Theta$ -direction modal mass (%)	Seismic weight (ton)
AA-1	0.62	0.71	0.58	55.6	52.9	45.3	9850
AH-2	0.70	0.58	0.78	61.9	63.6	44.2	11900
CM-3	0.56	0.80	0.68	45.4	57.1	36.9	12300
TL-4	0.61	0.77	0.88	62.1	59.8	65.2	7510
PR-6	0.50	0.33	0.39	60.3	70.0	63.4	2940
PP-7a, RT-8a	0.22	0.36	0.27	70.5	65.9	68.6	4000
PP-7b, RT-8b	0.23	0.34	0.24	71.6	69.2	71.3	1970
TO-9	0.93	0.53	0.28	48.3	49.1	22.14	17900

### 3. DAMAGE AND LESSONS FROM EARTHQUAKE BEHAVIOR

In this section characteristic damage observed in the study buildings is presented and discussed. Only partial damage summaries can be presented here (Westenenk et al 2011).

#### 3.1. Damage in Individual Buildings

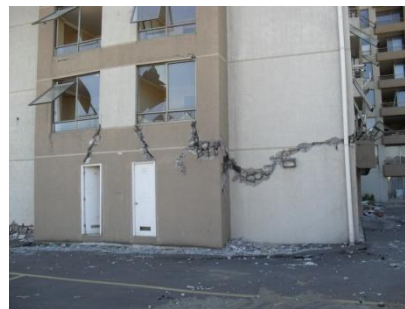
Shown in Table 3.1 is a brief qualitative description of the damage observed in each of the buildings. Most buildings present damage in shear walls that initiated on boundaries and propagated through the web once buckling of the longitudinal bars were not able to provide the required bending moment and axial capacity.

**Table 3.1** Summary of earthquake response in buildings

<i>AA-1</i>	This building had a significant vertical irregularity at several levels, but wall damage concentrated in the lower two stories. Wall setbacks at the lower level may have contributed to flexure-axial crushing failures below the setback.
<i>AH-2</i>	This building sustained wall crushing and global out-of-plane wall displacement in the first story and in basement levels of EW oriented walls. Damage resulted in building settling especially toward the North-west corner of the building, resulting in sloping floors in several units.
<i>CM-3</i>	This building sustained failures in multiple EW-oriented walls in the first two stories and the basement level. Failures involved shear cracking through multiple piers separated by openings, flexural compression failures (Figure 3.1.a), and shearing failures due to discontinuity in wall reinforcement.
<i>TL-4</i>	Shear walls sustained combined shear and flexure/axial failures in the lower story walls oriented in the EW direction. Failures also occurred in height due to flexural coupling of spandrels framing out-of-plane into the edges of L-walls at building corners.
<i>PR-6</i>	This building comprised two similarly configured wings separated by a construction joint. The wing with longitudinal axis oriented approximately EW, was relatively undamaged. The wing along lines N-S sustained severe wall flexural and shear damage in the EW (transverse) walls, with heaved soils around the foundation suggesting uplift. Figure 3.1.b shows wall damage in this building.
<i>PP-7</i> <i>RT-8</i>	These identical adjacent buildings sustained similar shear and flexure/axial damage, concentrated in the first two stories in EW-oriented walls.
<i>TO-9</i>	This building is unique for its multiple vertical and plan irregularities. Damage was relatively light in the lower twelve stories. Partial story collapses occurred at each of levels 12, 16, and 20, each story with a significant vertical or plan irregularity. Additionally, wall piers sustained shear failures over a significant height of the building (Figure 3.1.c).



(a)



(b)



(c)

**Figure 3.1.** Typical severe wall damage in buildings (a) CM-3; (b) PR-6; and (c) TO-9

### 3.2. Trends and Lessons in Seismic Building Behavior

Although the 8 buildings considered in this article are localized in the same region, damage patterns observed in them also occurred in other buildings at distant places with different base motions and soil conditions. This is indeed a proof that the observed repetitive damage is somewhat invariant to local anomalies, and that there is probably a more general design explanation to this damage. This section analyzes 5 critical aspects that may be correlated with the observed damage.

#### 3.2.1. Orientation of damage walls

It is apparent that observed damage in shear walls tends to concentrate mainly in one principal direction of the building. Damage is concentrated in walls oriented in the E-W direction for most cases. This behavior can be attributed to the fact that the E-W direction coincides with the transversal axis of the buildings. Figure 3.2a shows the severely damaged walls of building AH-2.

#### 3.2.2. Vertical and horizontal irregularities

Vertical and horizontal irregularities were a common factor present in most of the damaged buildings. AA-1 and TO-9 were characterized by having large horizontal and vertical irregularities. A common vertical irregularity pattern observed in most damaged buildings was the flag-shaped walls. This type of walls is characterized by having a reduced length at lower levels, mainly in first stories and parking levels. The flag-shaped walls produce high stress concentrations at the discontinuity region of the wall, which leads to localization of damage (Figure 3.2.b). Another aspect of vertical irregularity is found where two different walls become coupled through lintels in upper and/or lower stories. This irregularity is common as openings are closed in some stories. The coupling shear stresses induced at the continuous wall in top and bottom stories are very high causing both walls to fracture (Figure 3.2c). Moreover, beams supported by orthogonal walls (Figure 3.2d) induced damage to the walls since the cyclic motion tends to pull out the beam and punch the orthogonal walls. Furthermore, damage was also observed in spandrel beams coupled with transverse shear walls (Figure 3.2.e).

#### 3.2.3. Structural detailing of walls and construction issues

Typical Chilean construction changed after the success of the March 3, 1985, Chile earthquake (Wood 1991). Taller buildings were constructed with thinner walls as a result of not limiting axial stresses in walls. This proved to be a critical aspect in this earthquake, as thin shear walls subjected to high axial-flexural demands became overcrowded with longitudinal steel reinforcement at boundaries, are poorly confined, and showed to be brittle. Confinement of the boundary elements was almost inexistent. Typical detailing in wall boundaries considers the transversal steel with bar legs bent in 90°, but without hooking the longitudinal bars. Other construction problems and detailing issues that may have triggered the observed damage were identified, especially for building TO-9 where poor anchorages between beams and walls were frequently observed.

#### 3.2.4. Damage propagation within the structure

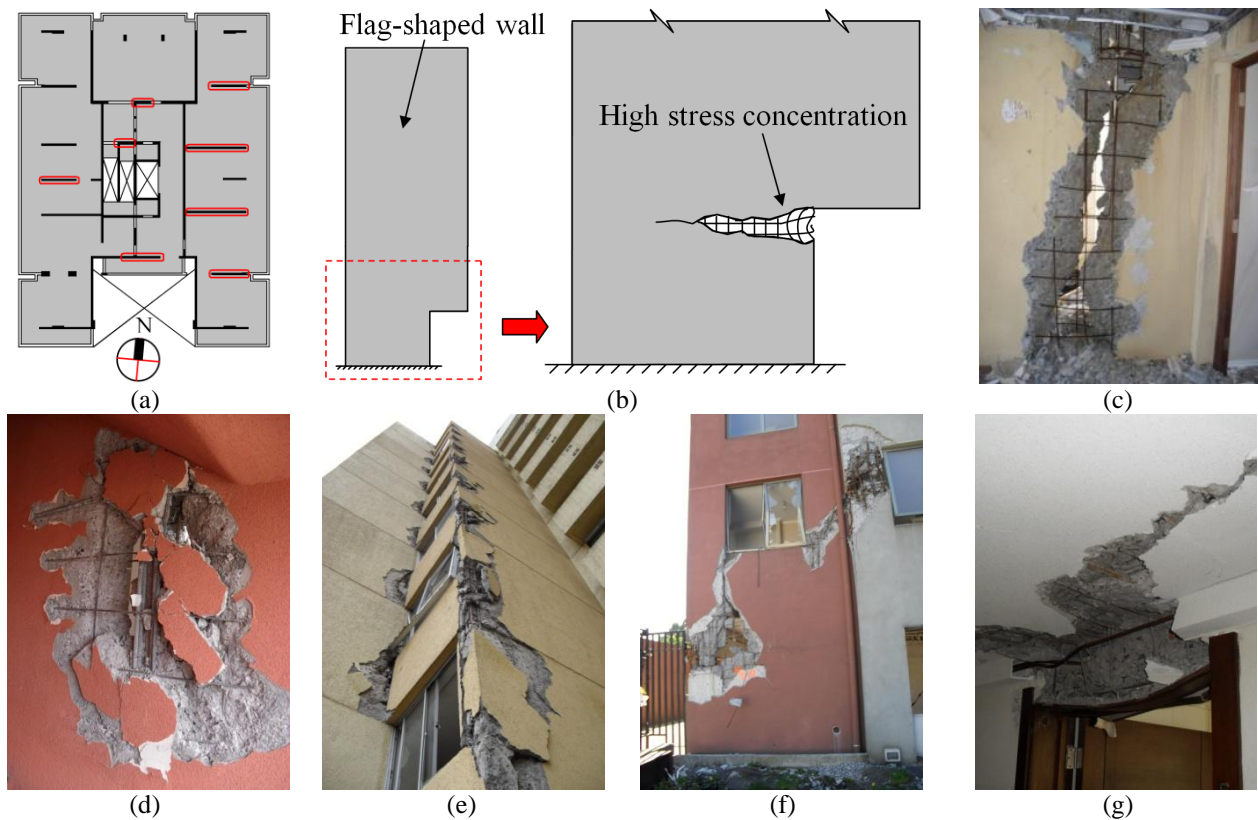
Buildings CM-3 and PR-6 evidenced multistory critical sections with damage (Figure 3.2f). The fracture along the critical section follows the simplest and least demanding failure mode. This type of damage is somewhat difficult to capture in design because structural models and design procedures treat building elements as one-story units. Therefore, buildings should always be analyzed and designed as 3D systems, considering all possible failure mechanisms crossing stories and connecting weak elements in neighboring stories. Propagation of damage into orthogonal wall elements and slabs was also observed in building PR-6.

#### 3.2.5. Energy dissipation

After 1985, the architectural topology changed by drastically reducing or eliminating coupling beams. Among several reasons, this was caused due to the difficulty to convince owners that it was hard to prevent damage in these elements even if they were heavily reinforced. Unfortunately, with this elimination a very good source of energy dissipation in the building was eliminated. Since the building must dissipate energy to reduce forces, in many cases the slabs turned into flat coupling beams and though they underwent severe structural damage, their dissipation capacity was small. Examples of



such behavior are shown in Figure 3.2g. The elimination of coupling beams also increased the flexibility of buildings inducing heavy damage in some non-structural elements.



**Figure 3.2.** Trends in seismic building behavior

#### 4. STATISTICS OF BUILDING DAMAGE

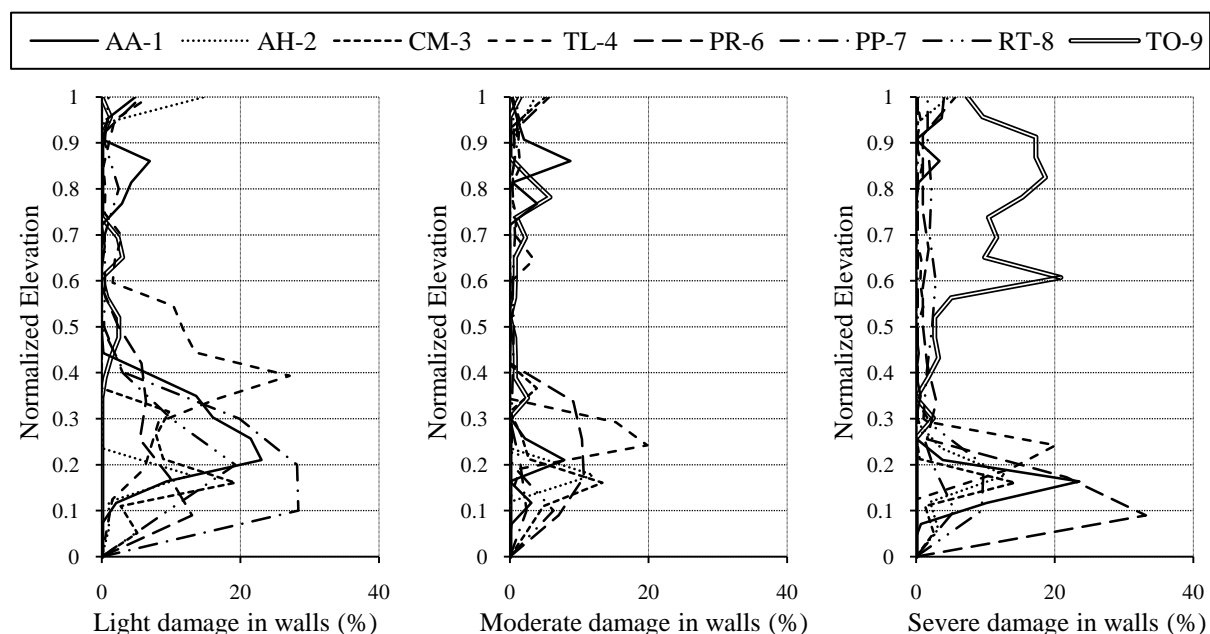
Several teams visited the study buildings and conducted a thorough damage survey considering every structural element in each building. Damage was defined as being light, moderate, or severe using a qualitative assessment of the estimated percentage loss of resistance due to damage. The classification “Light Damage” was assigned to elements with minor cracking and appeared capable of maintaining their original strength. “Moderate Damage” referred to elements with structural damage reducing strength on the order of  $1/4^{\text{th}}$  of the original capacity. “Severe Damage” was assigned to elements that appeared to have lost more than half of their original capacity.

Figure 4.1 plots damage percentages for shear walls classified with light, moderate, and severe damage versus the normalized height of the structure. The damage is measured by the volume of damaged elements relative to the shear wall volume in each story. Buildings show larger damage in lower stories, with the exception of TO-9, which had a  $12^{\text{th}}$  story failure, with additional damage in upper stories. Table 4.1 shows the total percentage of damaged elements in each building relative to the total volume of elements in the structure, classified by damage category. As expected, damage tends to concentrate in the lower stories, but the damage pattern varies among structures. Consolidated damage is largest in Building AA-1 and TO-9 and reaches values of the order of 12%. Therefore only a small volume of structural elements of any structure was compromised, suggesting inefficiency to distribute damage through the structure.

The damage data also can be organized according to type, location, and geometry of the damaged elements. A wall was designated as having shear damage if presented visible damage involving diagonally oriented cracks. A wall was designated as having flexural damage if the damage mainly

involved horizontal cracks or crushing. This classification is somewhat subjective since damage usually involves some combination of shear, axial force, and flexure.

Global building results show that more walls have shear than flexural damage. Although shear failures are well distributed among the buildings and present in all of the intensities defined above, flexural damage is localized in lower stories and is usually severe. Building TO-9 showed about 50% of wall damage in discontinuous walls. This significant percentage insinuates some of the configuration problems present in this structure, as the wall thickness reduction from 0.25m to 0.20m in this wall.



**Figure 4.1.** Distribution of percentages of (a) light, (b) moderate, and (c) severe damage in walls

**Table 4.1.** Damage to structural elements (walls, beams, slabs and columns)

ID	% Light Damage	% Medium Damage	% Severe Damage	% Total Damage	Story with more damaged elements		
					Story	% of total damage	% of severe damage
AA-1	4.24	2.75	5.33	12.31	2	1.66	0.68
AH-2	0.95	0.56	0.93	2.44	1	0.71	0.24
CM-3	1.31	0.89	4.40	6.60	-1	1.14	0.82
TL-4	3.51	1.60	1.27	6.38	2	1.38	0.57
PR-6	2.65	3.64	3.50	9.79	1	2.28	1.34
PP-7	4.46	0.54	1.08	6.07	1	1.98	0.35
RT-8	3.52	1.70	2.51	7.73	2	1.75	0.52
TO-9	0.34	0.74	9.16	10.23	16	1.15	1.02

## 5. BUILDING CODE TYPE ANALYSIS

A complete code review of the structural design was performed in buildings AA-1, CM-3, PR-6 and TO-9 with the purpose to obtain an idea of the nominal structural condition of each building. These buildings were selected because they have the largest percentage of severely damage elements. Nominal Demand/Capacity (D/C) ratios were computed for all elements. The results obtained are analyzed and contrasted with observed damage to validate any correlation with nominal strength deficit. Displacement and interstory drifts were also calculated for all the 8 buildings and compared with code provisions. The design spectrum given by the Chilean design code was used for modal analysis and 90% of the total seismic weight was considered in each direction of analysis. Modal superposition used the CQC method and a constant modal damping ratio of 0.05.

Please note that the code used in this review, NCh433-1996 (INN 1996) is the same as used for the design of the buildings. This code refers to ACI318-95 (ACI 1995) for the design of RC elements. Furthermore, all code analyses were based on the soil classification performed after the earthquake by the authors (Table 2.1). For building TO-9 the soil was originally classified as type II, and according to this study it should be type III (softer), implying a considerable larger seismic demand on the structure. Moreover, the original soil classification for CM-3 is soil type III and our study indicates soil type II. Therefore D/C ratios would increase considerably from the ones that will be shown later. A proper soil classification is particularly critical in these projects.

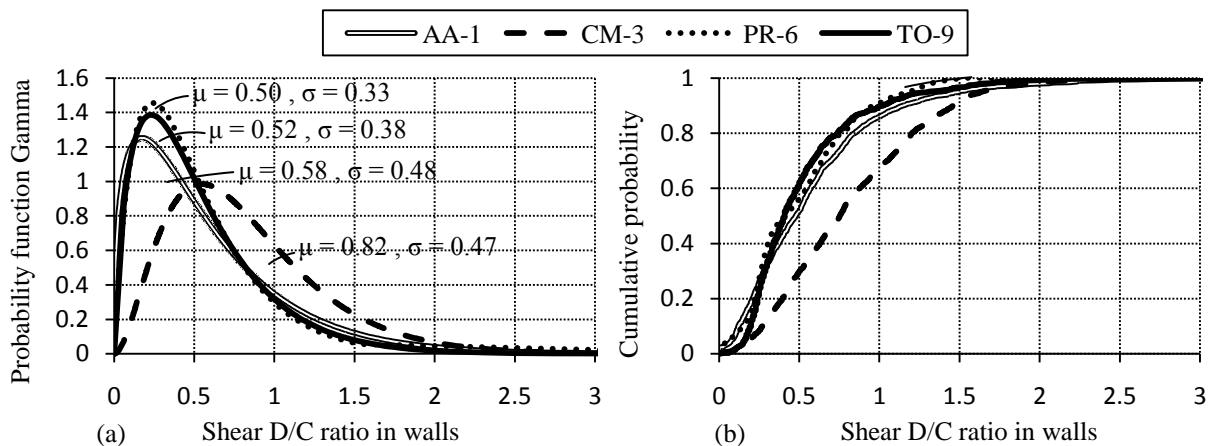
### 5.1. Results for Wall Analysis

Demand-capacity ratios in shear are defined as  $V_u/\Phi V_n$ . The results for a representative sample of walls distributed in all buildings are summarized in Table 5.1. The distribution of D/C ratios, its mean value, and standard deviation for the 4 buildings analyzed is presented also in this table. A similar analysis was performed for beams and for axial-flexural interaction in walls (Westenenk 2011). It is apparent from Table 5.1 that all buildings present walls with D/C ratios greater than one, which implies a deficit relative to code design. However, the number of elements with D/C greater than one is not large in general, which implies that the design of these structures is not balanced in terms of D/C requirements and probably just a few elements caused the high level of damage observed. Note that mean values for buildings AA-1, PR-6 and TO-9 range from 0.50 to 0.58, while for CM-3 is 0.82. This implied that the design of every element in building CM-3 is very close to 1, which leaves little space for damage redistribution in the structure, especially considering the brittle failures observed. Note that building TO-9 shows maximum code deficiencies on axis J, exactly the one that collapsed in story 12.

**Table 5.1.** Total results for shear in walls

ID	N° of walls analyzed	D/C < 1 (%)	1 < D/C ≤ 1.25 (%)	1.25 < D/C ≤ 1.5 (%)	D/C > 1.5 (%)	$\mu_{D/C}$	$\sigma_{D/C}$
AA-1	1458	85.87	5.76	3.43	4.94	0.58	0.48
CM-3	1383	67.25	14.68	9.69	8.39	0.82	0.47
PR-6	593	90.89	5.23	3.54	0.34	0.50	0.33
TO-9	1152	89.67	4.86	2.08	3.39	0.52	0.38

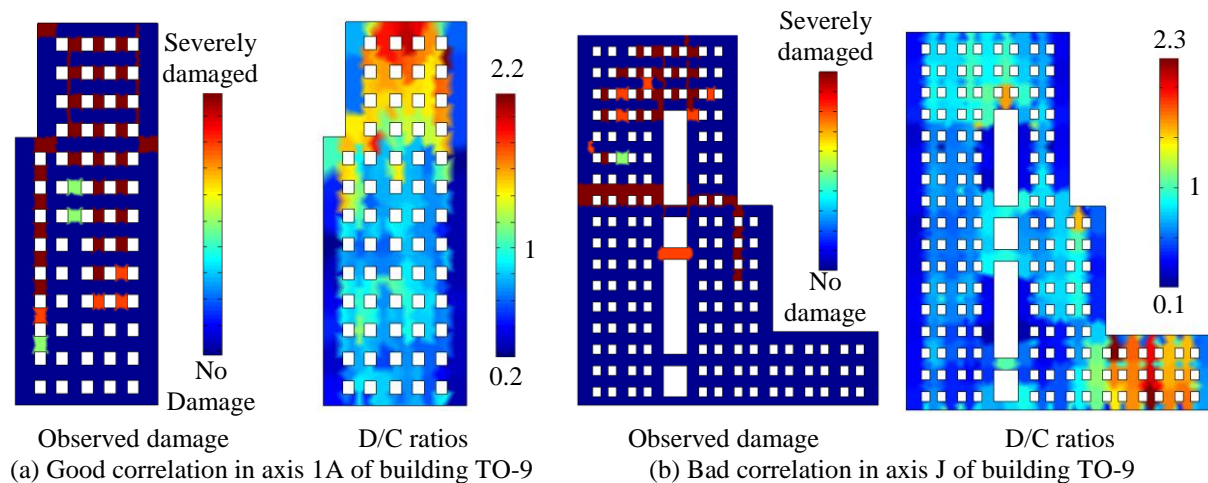
The histograms of shear D/C ratios in walls for each building analyzed were fitted to a gamma distribution probability density function as shown in Figure 5.1.a. Cumulative probabilities are shown in Figure 5.1b. Although buildings AA-1, PR-6 and TO-9 show similar D/C distributions, the CM-3 curve is well below and displaced from the others. This emphasizes the fact that CM-3 has a weaker design condition. This behavior is confirmed by the similar axial-flexural results regarding D/C ratios.



**Figure 5.1.** Damage data: (a) D/C histograms fitted to a Gamma distribution; and (b) cumulative probability for shear D/C ratios



Special damage drawings were prepared in order to physically compare the observed damage with that resulting from code analysis results. The comparison revealed that there are several cases in which the design deficit, represented by high D/C ratios, matches the damaged elements (Figure 5.2a). However, there are also cases in which damage cannot be explained by high D/C ratios (Figure 5.2b). This happens due to the load redistribution that occurs between elements: as one element collapses, load is redistributed within the structure.



**Figure 5.2.** Comparison of damage correlation results: (a) Good, and (b) bad correlation between observed damage and D/C ratios

## 5.2. Drifts and Displacements

As final and global design verification, nominal interstory drifts and displacements were checked in all 8 structures. Note that code NCh433-96 states that the (reduced) drift in the center of mass (CM) must be less than 0.002, and that maximum drift in any point must not exceed in more than 0.001 the drift at the CM. Results show that every building with a predominant shear-wall configuration meets the displacement and drift requirements of the code; which says, in light of this earthquake results, little about the quality of its structural configuration in plan and height.

## 6. CONCLUSIONS

This work summarizes the observations obtained in 8 reinforced concrete shear wall buildings that were severely damaged during the 2010 Chile earthquake. The study buildings range in height from 10 to 21 stories, with all but one constructed after 2000. These buildings have relatively high ratios of structural wall-to-floor area, as is typical in Chilean wall buildings. The structures have relatively short calculated fundamental vibration periods, ranging from 0.22s to 0.62s, consistent with the high wall-to-floor-area ratios.

Damage in these buildings is probably the result of multiple physical factors. There is some positive correlation between damage and building orientation, poor soil quality, plan and height building irregularities, slenderness ratio, lack of confinement, and small wall thicknesses.

Field observations suggest that the most serious damage occurred in the E-W oriented resisting planes, which coincides usually with the narrower transverse axis of the buildings. Survey results show that more walls had shear damage than flexural damage, being flexural failures more common in walls with high damage intensity. In most of the buildings, severe damage concentrated in the lower stories without significant redistribution of inelastic response. In one of the buildings, however, structural

irregularities apparently contributed to severe shear-related damage and collapse in the upper stories.

Based on the building-code type analysis developed it is apparent that there are D/C ratios that highly exceed one, hence, there is a design clear deficit. In some cases such deficit matches the damaged elements, but in others damage cannot be explained only by high D/C ratios. Although this observation implies that there might be problems in design and construction issues in these structures; it is apparent that inelastic (non-linear) analysis is mandatory in order to better understand how damage really occurred and propagated.

It is concluded that design codes must be revised relative to wall design provisions. New codes should deal with a better ground motion characterization in longer periods and stricter limits to axial loads and wall confinement requirements. Two aspects that seem simple and crucial to implement in the short term are: (i) a limit axial to loads in walls to prevent brittle behavior, and (ii) a limit to minimum wall thicknesses to improve constructability, structural integrity, and better confined boundary reinforcement.

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