Study on Partial Collapse of a Five Story Reinforced Concrete Building during the 2010 Chile Earthquake

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SUMMARY:
After the 2010 Chile earthquake about 20 reinforced concrete (RC) buildings were declared inhabitable and four buildings faced demolition order within the city of Santiago, which is located more than 300 km away from the epicenter. The objective of this paper is to present the main findings of, and the lessons learned from, a preliminary study on the partial collapse of a 5-story residential building. To achieve this goal, the seismic design of this building was verified using the response spectrum method following the Chilean seismic design code at the time of the earthquake. The building was designed with a mixed RC and confined masonry structure; however the parking level was structured mostly with RC in order to maximize parking spaces. It is concluded that: (1) walls at the basement level had inadequate demand/capacity ratios, (2) confinement requirements in RC walls needs to be revised, and (3) vertical irregularity must be incorporated in Chilean code.

Keywords: Reinforced concrete, case study, Chile earthquake, collapse, vertical irregularity

1. INTRODUCTION

On February 27, 2010, one of the largest earthquakes ever recorded with a magnitude $M_w = 8.8$ struck the central region of Chile. The inventory of engineered buildings (mostly reinforced concrete) located in the affected area is large: approximately 10,000 buildings of three or more stories were constructed since 1985, 2000 of which have nine or more stories (INE 2009). However, just 10% of this building inventory suffered some degree of damage (Bonelli et al. 2011), about 80 buildings suffered severe structural damage and only 4 buildings collapsed, either totally or partially (EERI 2010). While these observations clearly indicate that the performance of engineered buildings can be certainly considered satisfactory, it is nevertheless always instructive to analyze the behaviour of damaged structures, particularly the behaviour of collapsed structures where the objective of the seismic code was definitely not met. The objective of this paper is to present the main findings of, and the lessons learned from, a preliminary study on the collapse of the residential 5-story Don Tristan building. This building was a mixed reinforced concrete (RC) and confined masonry (CM) structure located in southwest suburbs of Santiago, more than 300 km away from the epicenter.

2. DESCRIPTION OF THE BUILDING

Don Tristán building was built in 2005 and its configuration is representative of Chilean mid-rise residential buildings. The basement story, which was partially buried, was designed for vehicle parking and the four upper stories were designed for residential purposes. The plan dimensions of the building were 54.1 m in the longitudinal direction and 13.1 m in the transverse direction (Fig. 1). The total height of the building, measured at the west side from the foundation level to the top edge of the roof parapet, was 13.96 m. The height of the basement story was 2.67 m and the height of higher stories was 2.47 m. The building was founded on silty sand which was classified as soil type II according to NCh433, the Chilean seismic design code (INN 1996). This soil is defined as hard soil with shear wave velocity larger than 400 m/s in the upper 10 meters.

The building was designed with mixed RC and CM walls; however the basement level was structured mostly with RC walls in order to maximize parking spaces. The plan view of the basement and first story is shown in Fig. 1. The wall thickness was 15 cm for RC and 14 cm for CM. This low thickness of RC walls is typical of Chilean buildings constructed in the last decade. The slab thickness was 12 cm and it considers a concrete floor cladding of about 5 cm. The ratio of RC wall area to floor area in the longitudinal direction was 1.55% and 0.99% for the basement and first story, respectively. The ratio of RC wall area in the transverse direction was 1.45% and 1.08% for the basement and first story, respectively. On the other hand, the ratio of CM wall area to floor area was 0.05% and 1.27 % in the longitudinal direction, and 0.00% and 1.04% in the transverse direction for the basement and first story, respectively. The RC wall ratios at the basement story in both directions are in the lower level of Chilean residential buildings where wall ratios varies from 1.5% to 3.5%, with a mean ratio of 2.8% (Guzmán 1998, Moroni 2011).

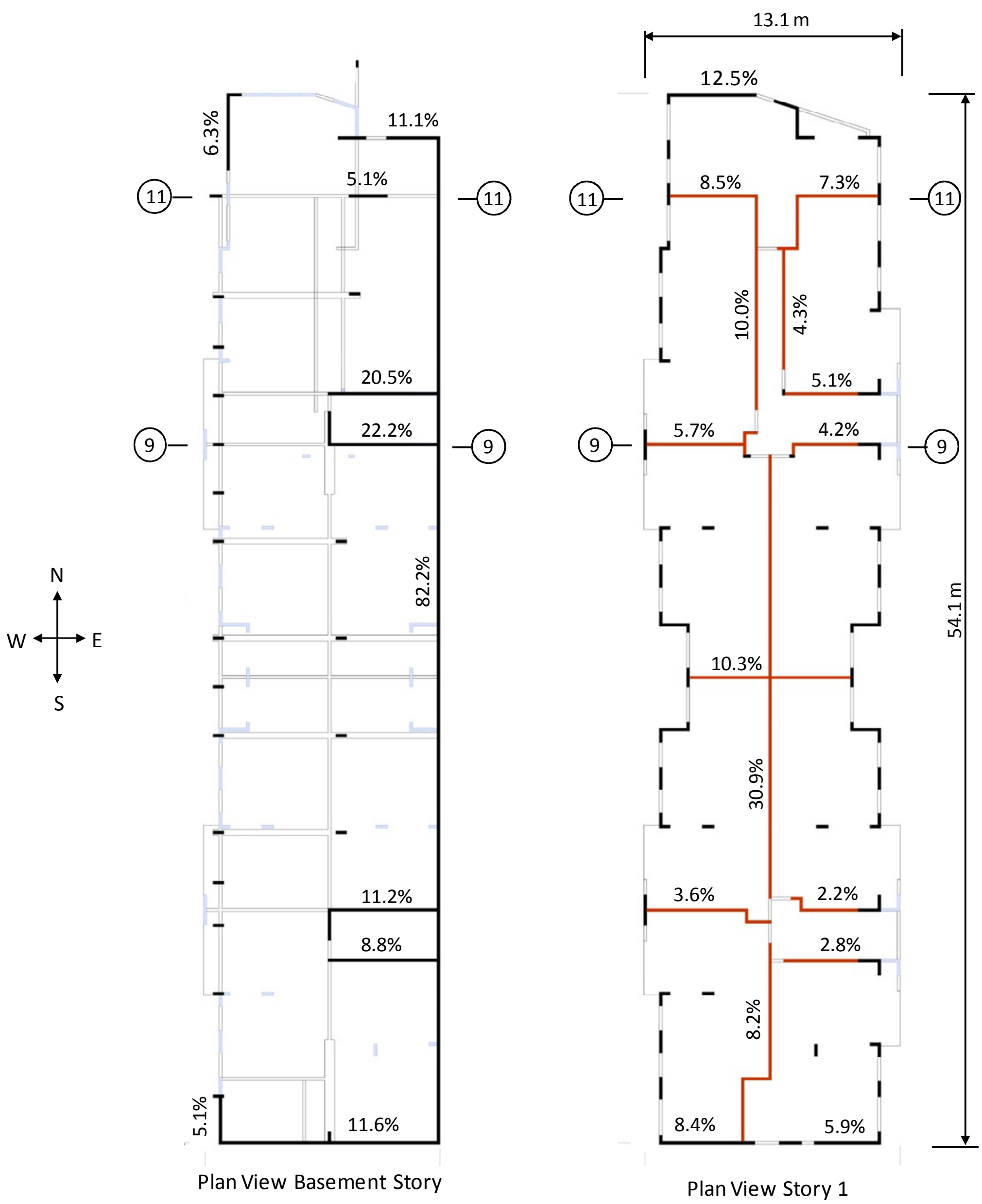

Figure 1. Plan view of basement and story, and percentage of story shear taken by longitudinal and transversal walls. RC elements are shown in black and CM elements in red.

Vertical irregularity of Don Tristán building is evident from the plan view (Fig. 1), where only 44% of wall area of the first story was continuous towards the basement story. In the longitudinal direction, the CM walls located at the center corridor of the first story did not continue towards the basement story. Additionally, in the longitudinal direction of the basement story, most wall area is concentrated in the perimeter wall of the east side, which induces torsion. In the transverse direction, most of CM walls located at the west side of the first story did not continue towards the basement story, and slender RC walls of 55x15 cm were detailed at the west side at the basement story (shown in black in Fig. 1). The vertical irregularity of the building can also be observed in the elevation of axis 9 and 11 in Fig. 2. In both axis, RC was considered for the basement story and mostly CM was considered for the upper story. Additionally, there is a large vertical discontinuity of the walls between the first story and the basement.

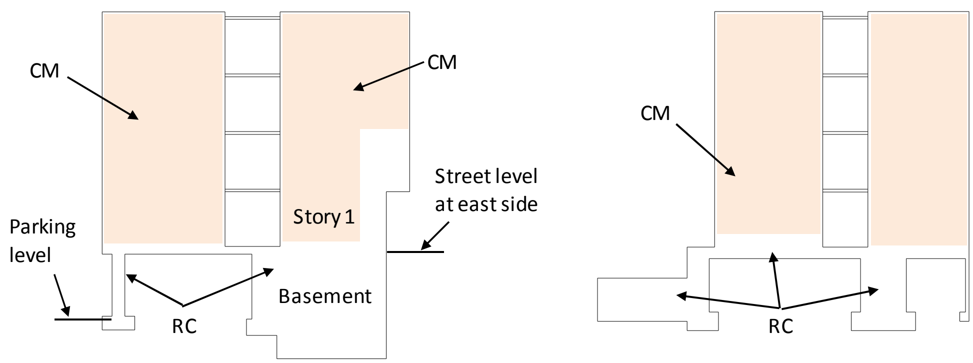

Figure 2. Vertical irregularity in elevation of axis 9 (left) and 11 (right)

3. OBSERVED DAMAGE

During the 2010 Chile earthquake, all the columns located in the basement story along the northern half of the west side and the longitudinal wall located at the north-west corner lost their capacity to carry gravity loads, and were completely crushed by the upper stories as these felt directly on the ground (Fig. 3). The east-south part of the building, however, did not fall because the perimeter walls located along the east and south sides remained basically undamaged. Hence, huge differential vertical displacements were induced between the falling part of the building and the part that did not fall. These large differential displacements destroyed many slab edges, particularly at the boundary between the falling and non-falling parts of the building (Fig. 3). The falling part also rotated along a horizontal longitudinal axis as it felt, which caused further damage at the walls located along the northern half of the east side (Fig.4).

The described observations suggest that the collapse of Don Tristán building can be characterized as a partial weak-story failure at the (partially buried) first story. Unfortunately, possible reasons why some vertical elements at the first story lost their load-carrying capacity could not be observed in the field because these elements were totally destroyed.

No persons were present at the basement story at the moment of the earthquake. Several cars were crushed by the falling part of the building, but none of them exploded. Residents reported a strong natural gas smell right after the earthquake, but no fires started. As a result, the collapse of Don Tristan building did not cause human casualties. The building was declared inhabitable immediately after the earthquake by public officials. Access was restricted, even to their residents, and was permitted on a limited basis only for the recovery of personal belongings and for inspection purposes. The building was finally demolished in July 2011.

Figure 3. Damage of Don Tristán building after the 2010 Chile earthquake: (left) general view from the south-west, and (right) the first floor level resting directly on the ground at the basement level (the remains of totally crushed columns can be observed)

Figure 4. Northern part of the east side of DonTristán building after the 2010 Chile earthquake

4. VERIFICATION OF CODE COMPLIANCE

In this section, the building seismic design is verified using a modal spectral analysis according to the NCh433, the seismic design code at the time of the building construction (INN 1996). The building was located in seismic zone 2, which corresponds to an effective acceleration of 0.3g, it was founded in soil type II and the importance factor is 1.0. The force reduction factor (R* in the Chilean code) is computed for each direction of the building and a value of 3.154 and 3.405 are used for the longitudinal and transverse direction, respectively. This reduction factor is obtained from the seismic code considering a mixed RC-CM building were RC walls take less than 50% of the story shear.

4.1. Analytical Model

A three-dimensional finite element model of the building was constructed using the commercial software ETABS (2005). The model contains 9200 nodes and 7500 elements and is shown in Fig. 5. The RC and CM walls are modelled using shell type elements. The RC slabs are also modelled using shell type elements and assuming rigid diaphragm at each story level. The walls are fixed at the base and no flexibility of the soil is considered.

The dead load of the buildings is obtained in ETABS automatically from the self weight of the elements. However, additional dead load was introduced in the model to represent the floor cladding, non structural partitions and the stucco on RC and CM walls (Vizcaíno 2011). For the live load 2.0 kN/m^2 is considered at the slab, except for the hallways and balconies were 2.5 kN/m^2 is used according to the Chilean design code.

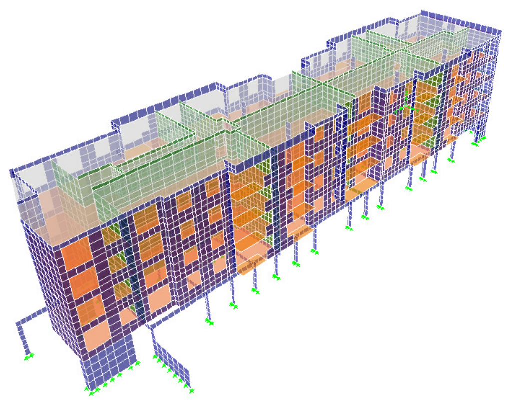

Figure 5. Finite element model of the building (from the west side)

4.2. Dynamic Results

The seismic weight of the building, which include 25% of the live load, is 25,300 kN and the fundamental period in the transverse, longitudinal and rotational directions are 0.181, 0.140 and 0.124 seconds, respectively. The building is stiff and the fundamental periods are shorter than *N*/20 (in seconds), where *N* is the number of stories. This empirical formula correlates well with Chilean buildings built before the 90s (Wood 1991). The mass participation ratios for the first three modes are shown in Table 1, were coupling of the longitudinal and rotational modes is evident from the table. The modal spectral analysis of the building was conducted using the first 12 modes, which involves mass participation ratios larger than 97% for each direction of analysis. These ratios are larger than the 90% required by NCh433

Table 1. Dynamic properties of the building

Mode	Period (sec)	Mass participation ratios (%)		
		Longitudinal	Transverse	Rotational
1	0.181	1.5	69.8	9.3
2	0.140	59.2	5.3	15.7
3	0.124	17.5	3.9	55.4

From the dynamic analysis considering the elastic spectra reduced by the *R** factor, the roof displacement is 0.13 and 0.23 cm for the longitudinal and transversal direction, respectively. These values are low but were expected due to the short period of the building in both directions. The interstory drifts measured at the center of mass are listed in Table 2, where a maximum drift of 0.034% is obtained for the transverse direction at stories 1 and 2. This drift is about six times lower than the drift limit of 0.20% required by NCh433. The larger interstory drift obtained at any point of the floor plan, when accidental torsion is included, is also listed in Table 2. In this case, 0.052% is obtained for the transverse direction of the basement. The relative drift at this level, with respect to the center of mass, is 0.052-0.024 = 0.028%, which is about three times lower than the limit of 0.1% that NCh433 recommends to control torsion.

The computed base shears for the longitudinal and transverse direction are 13.9% and 15.4% of the seismic weight of the building, respectively. These values are larger than the minimum base shear of

5% and less than the maximum base shear of 16.5% required by NCh433 for this structure.

The percentage of the story shear that takes the principal walls in the longitudinal and transverse direction for the basement level and first story, when accidental torsion is ignored, is shown in Fig 1. For the transverse direction of the basement level, most of the story shear (90.5%) is distributed within seven walls, where the most demanded wall takes 22.2% of the story shear. For the longitudinal direction of the basement level, 82.2% of the shear wall is taken by the perimeter wall of the east side, which induces torsion to the building. For the transverse direction of the first story, 52.3% and 47.7% of the story shear is taken by CM and RC walls, respectively. For the longitudinal direction of the first story, 59.9% and 40.1 of the story shear is taken by the CM and RC walls, respectively. Since RC walls takes less than 50% of the story shear in the first story, a reduced R* that takes this fact into account is considered in this dynamic analysis, according to NCh433. It is important to notice that the discontinuity of the vertical elements between the basement level and first story implies that the shear force needs to be distributed throughout the slab from. However, shear damage in the slab could not be observed by the authors because access to the interior of the building was not allowed.

Table 2. Computed interstory drift

Story	Interstory drif at center of mass (%)		Larger interstory drift at any point (%)	
	Longitudinal direction	Transverse direction	Longitudinal direction	Transverse direction
4	0.008	0.026	0.011	0.031
3	0.011	0.031	0.013	0.033
2	0.013	0.034	0.014	0.035
1	0.012	0.034	0.012	0.035
B	0.007	0.024	0.007	0.052

4.2. RC Walls Design Verification

The strength of the resisting elements is verified according to NCh433, which uses the forces from the modal spectral analysis. The strength of the RC walls are computed following ACI-318 (2005), however, confinement at the wall boundaries was not required by the Chilean code at the time of the building construction. For this building, the concrete was specified as H20 ($f'_c = 16$ MPa) and the steel as A630-420H ($f_y = 420$ MPa).

The critical elements of the building are the 55x15 cm RC walls located at the west side of the basement. The detailing of these walls is shown in Fig. 6, which considers 4ϕ12 vertical bars at the boundary and stirrups ϕ8 spaced at 20 cm. The interaction diagram for combined axial load and bending moment of the 55x15 cm wall, located at the west side of the basement of axis 9 (Fig. 2) is shown in Fig. 6. In this diagram, demand/capacity (D/C) ratio is 1.77 and the maximum factored compressive load from the load combinations represents $1.1f'_c$. The large compressive load in this wall is induced by the overturning moment which is generated by the seismic action in the transverse direction. To prevent brittle compressive failure in RC walls, NCh433 was modified in 2011 introducing a limit in the axial stress of walls of $0.35f'_c$ (2011).

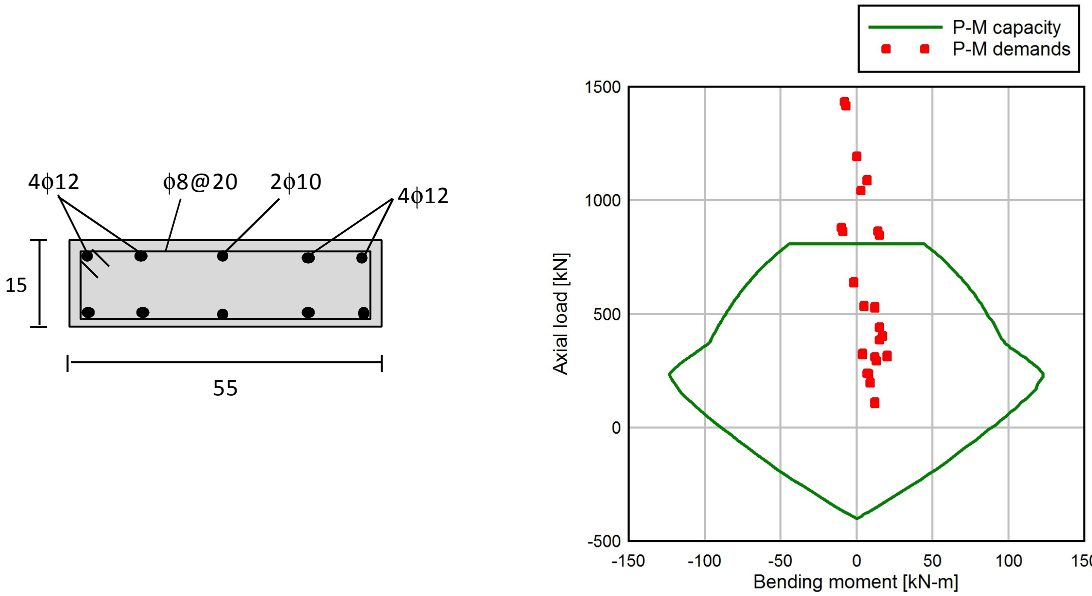

Figure 6. Typical detailing of 55 x 15 cm walls and confined concrete at the stirrups level (left) and interaction diagram of 55x15 cm wall in the basement of axis 9 (right)

The frequency distribution of D/C ratios for combined axial load and bending moments of the basement walls is shown in Fig. 7, where the D/C ratio is the quotient of the distance between the origin and the demand point, and the distance between the origin and the capacity curve. For the transverse direction 10 walls has D/C ratios larger than 1.0 and for the longitudinal direction the D/C ratios of every wall are less than 1.0. The basement walls with D/C ratio greater than 1.0 are shown in Fig. 8, where most of this walls are located in the west side and the center of the building, which is consistent with the observed damage (Fig. 3).

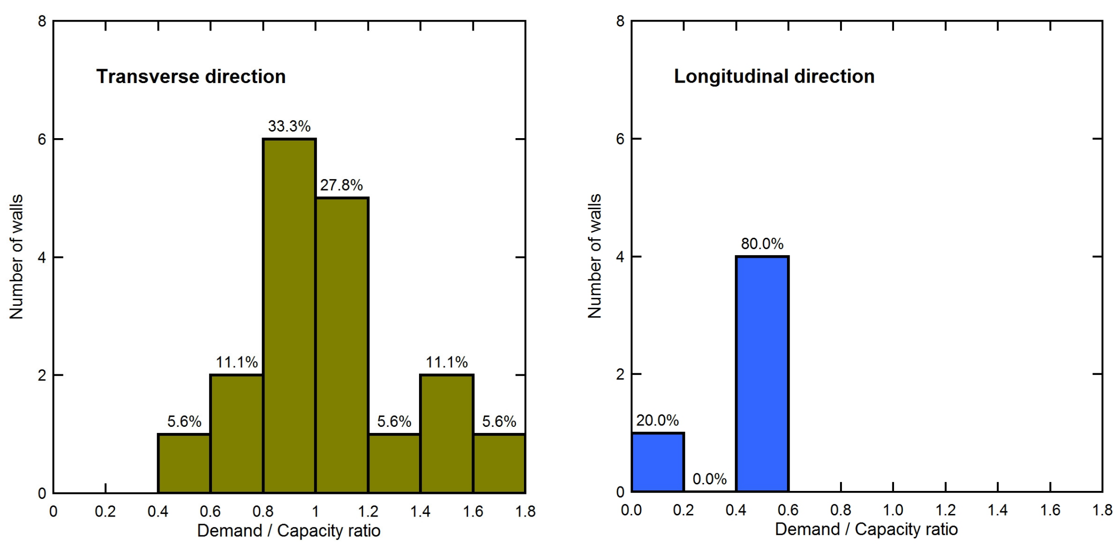

Figure 7. D/C ratio histogram for combined axial load and bending moment for basement walls

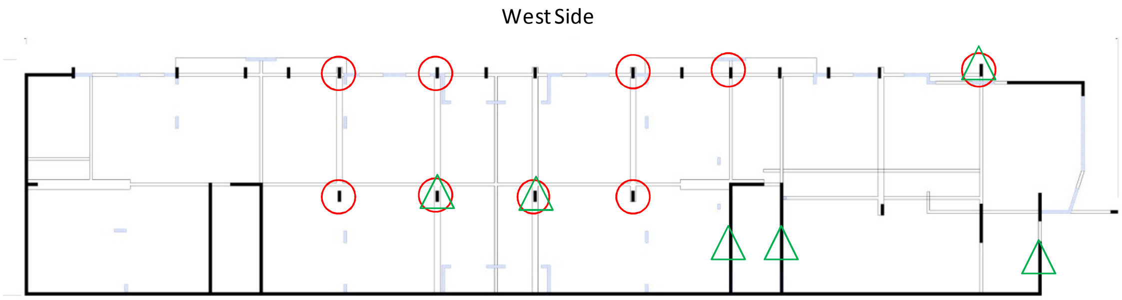

Figure 8. Walls of the basements with combined axial load and bending moment D/C ratios > 1.0 (red circles) and with shear stress beyond ACI limit (green triangles)

The shear stresses of the RC walls of the basement are also verified. For the transverse direction, six walls exceeded the shear stress limit of $0.66\sqrt{f_c^{'}}$ (MPa) required by ACI. These walls are marked with green triangles in Fig. 8. For the longitudinal direction, the walls of the basement do not exceed the shear stress limit.

5. ANALYSIS OF POSSIBLE CAUSE OF COLLAPSE

Based on previous results, the collapse of the building at the basement story could be explained because the demand of combined axial load and bending moment and the demand of shear exceeded the acceptable limits in critical RC walls. The large D/C ratio could be explained by the force reduction factor (R*) used in this study. According to NCh433 the reduction factor is given by

$$R^* = 1 + \frac{T^*}{0.1T_o + T^*/R_o}$$

where T^* is the fundamental period in the direction of analysis, T_o = 0.3 sec for soil type II. For mixed RC-CM buildings, R_o=9 if RC walls take more than 50% of the story shear at every level, or R_o=4 in other cases. If R_o=9 is considered in the dynamic analysis, less demand is obtained because the force reduction factor increases by 29% and 35% for the longitudinal and transverse direction, respectively.

Besides the limited strength of the basement walls, the confinement provided to the 55 x 15 cm walls is negligible because of the large stirrup spacing, the lack of cross ties, and the thin section of the wall. This limited ductility is not appropriate for the reduction factor considered. Using the Mander approach (Mander et. al 1988), the area of confined concrete at the stirrups level, shaded regions in Fig. 9, is 18% of the area of confined core. Considering the stirrups spacing of 20 cm, the area of effectively confined concrete core at midway between stirrups is only 1% of the concrete core. With such confinement, crushing of concrete is achieved at a strain of about 0.003 and a brittle compressive failure is expected beyond that strain.

The vertical irregularity of the building could also explain the building collapse by causing stress concentration and localized damage in the basement walls. Example of vertical irregularity of the building is shown in Fig. 2, where high axial load is induced from the overturning moment which is transferred from the CM walls of the first story to the slender RC walls of the basement. Another important vertical irregularity is located in the transverse axis at the center of the building. In this axis, RC walls at the basement transfer some of the shear force resisted by the center CM wall (10.3% of story shear is resisted by this CM wall in story 1, Fig. 1) to the foundation level. Therefore, the vertical discontinuity at this section results in excessive shear demand in the slender RC walls of the basement (Fig. 8).

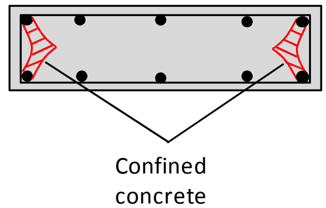

Figure 9. Confined concrete are at the stirrups level in the 55x15 shear wall

Regarding the ground motion, the only ground acceleration histories recorded during the 2010 Chile earthquake in the Maipu area are the ones recorded at the Maipu station, located just 2 km away from the building. Both horizontal components are depicted in Fig. 10. They are typical Chilean records in that the duration is long and near-fault effects are absent. Since soil conditions at the Maipu station are similar to those at the location of the building, and the Maipu area is not close to the main rupture area, these records give a good idea of the ground accelerations the building must have been subjected to during the 2010 Chile earthquake. PGA values of the N-S and E-W components are equal to 0.56 g and 0.49 g, respectively, which are significantly greater than the design PGA of 0.30 g indicated in NCh433. A comparison between the pseudo-acceleration response spectra (damping = 5%) of the Maipu records and the NCh433 elastic design spectrum can be seen in Fig. 11 (left), where it can be observed that, at least in the range of periods considered (up to 2 sec), the spectral ordinates of the Maipu records are always greater than the ones of the NCh433 design spectrum. In particular, the spectral ordinates of the records are essentially equal to that of the design spectrum at the first lateral period of Don Tristan building in the transverse direction (0.181 sec), but are 30% greater at the first lateral period in the longitudinal direction (0.140 sec). Therefore, the amount of damage in the transverse direction could not be explained by the level of ground shaking, even though the ground shaking appears to be greater than the level of ground shaking for which it was designed. However, the periods of the building may have increased by the induced damage, where the spectral ordinates of the records are much higher than the ordinates of the design code.

Finally, Don Tristan building was just one of the many engineered buildings located in the urban Maipu area, which can be appreciated in Fig. 11 (right). Only a handful of these buildings suffered some degree of structural damage during the 2010 Chile earthquake, and the only other one that collapsed (actually a structural collapse, i.e., damage well beyond repair) was the Don Luis building, which is nearly identical to Don Tristan building. These observations strongly suggest that the reason for the collapse of Don Tristan building are related to its structural characteristics rather than to the level of ground shaking it was subjected to.

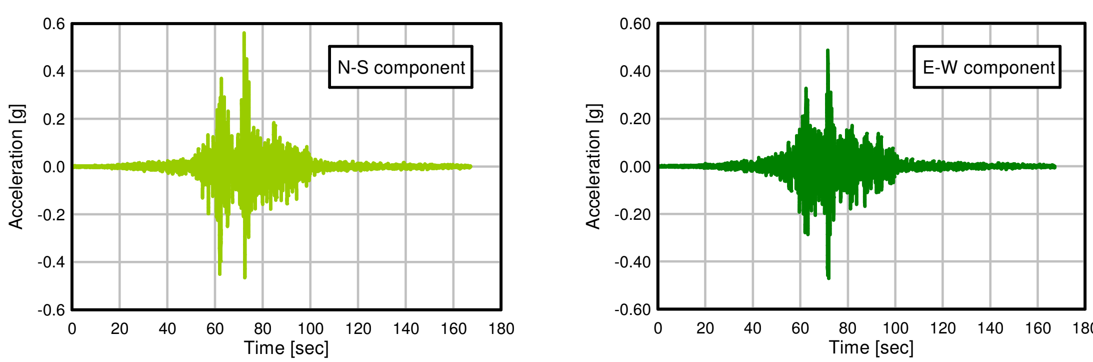

Figure 10. Ground acceleration histories recorded at the Maipu station during the 2010 Chile earthquake

5. CONCLUSIONS

From the study on the partial collapse of Don Tristán building the following conclusions are obtained:

(1) The D/C ratio for the combined axial load and bending moment, and the shear stress demand in

critical RC basement walls of Don Tristán building exceed the acceptable limits.

(2) The observed damage could not be explained by the excessive ground shaking because the spectral ordinates at the fundamental periods of the building are similar to that of the design code.

(3) The confinement requirements for RC walls need to be revised to prevent brittle behaviour. However, the confinement requirements and the limit of the axial stress in RC walls have already been modified in Chilean code after the 2010 Chile earthquake.

(4) The vertical irregularity at the basement stories in current residential buildings is somehow excessive. This irregularity induces concentration of stresses and localized damage. The vertical irregularity issue must be incorporated in Chilean design code.

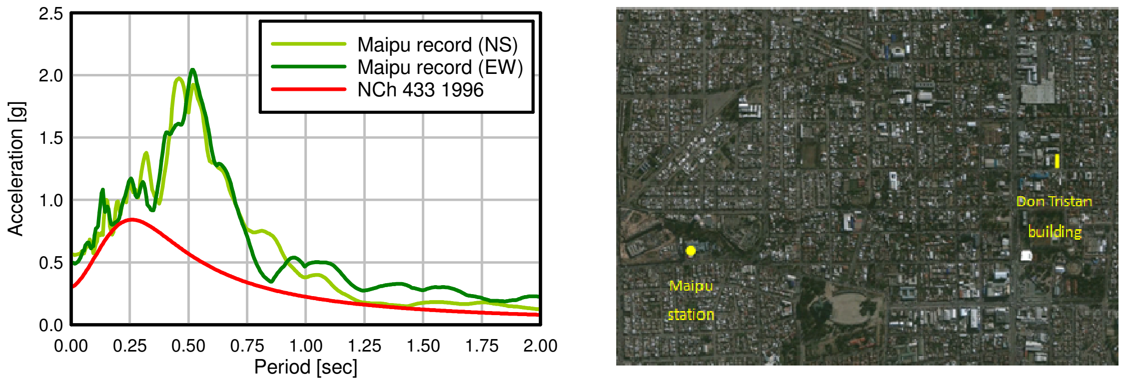

Figure 11. (left) Comparison between pseudo-acceleration response spectra and Code design spectra for 5% damping and (right) aerial view of the Maipu area

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