Performance of Confined Masonry Buildings in the February 27, 2010 Chile Earthquake

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1. Background

Confined masonry has been widely used for housing construction in Chile. Housing construction includes low-rise single family dwellings (up to two-storey high), and medium-rise apartment buildings (three- to four-storey high). Examples of confined masonry buildings are shown in Figure 1. Chile has a long history of confined masonry construction practice, starting in the 1930s, after the 1928 Talca earthquake (M 8.0). Initially, confined masonry was used for the construction of low-rise building (single family dwellings), while the construction of medium-rise apartment buildings started in the 1980s in the capital Santiago, and in 1990s in other urban areas.

Good performance of low-rise confined masonry buildings in the 1939 Chillan earthquake (M 7.8) paved the path for the use of this construction technology in Chile ever since. Low-rise confined masonry construction maintained a good performance track record in other Chilean earthquakes, including the 1985 Lloleo earthquake (M 7.8). However, three- and four-storey confined masonry buildings had not been exposed to severe ground shaking in Chile prior to the February 2010 earthquake.

a)

b)

Figure 1. Typical confined masonry buildings in Chile: a) a single-storey rural dwelling, and b) a four-storey apartment building.

Design and construction of confined masonry buildings in Chile has been regulated by NCh 2123.Of1997 (revised in 2003). Seismic design of buildings is addressed by NCh433.Of96. Before 1997, the 1940 document "Ordenanza General de Urbanismo y Construcción" had been used for confined masonry construction.

Single-family dwellings are in the form of either one-storey detached houses or two-storey houses in the row (townhouses). Plan dimensions for a typical unit are approximately 5 by 6 m, while the clear floor height is on the range of 2.2 to 2.3 m. Kitchen and living room in two-storey houses are located at the ground floor level, while the bedrooms are located at the second floor. The front facade of the house is usually perforated with openings, while the transverse walls are solid. One- and two-storey dwellings have timber floors and pitched timber roofs with timber gables.

Typical plan dimensions for three- and four-storey apartment buildings are: length from 25 to 30 m and width from 5 to 8 m. There are typically four to six apartments per floor (depending on the building size). An additional wall in the longitudinal direction (in addition to the exterior walls) is provided in buildings with larger width. The distance between transverse walls ranges from 5 to 8 m. These buildings usually have concrete floors and pitched timber roofs with timber trusses supported by tie-beams running at the perimeter. Concrete floors are either cast-in-place or precast, with large hollow masonry blocks (called *bovedillas* in Spanish) laid horizontally between precast reinforced concrete beams (this is known as "Tralix" system).

Confined masonry buildings are characterized by masonry walls and reinforced concrete (RC) confining elements (tie-columns and tie-beams). The walls are constructed first, followed by casting of concrete in RC tie-columns, as shown in Figure 2. The walls are usually 140 mm thick and are built using hollow clay blocks (hollow clay tiles) in cement mortar. A typical hollow clay block is shown in Figure 3 a. Hand-made clay bricks are used for interior wall construction in some three- and four-storey high confined masonry buildings. Hollow concrete blocks are scarcely used in the earthquake affected area, however these blocks appear to be used in the northern part of Chile. There is no information on the mechanical properties of masonry units found in the field, however NCh 2123 specifies the minimum compressive strength for clay brick units in the range from 4 MPa (hand-made bricks) to 11 MPa (machine-made bricks). The specified strength of concrete blocks is 12 MPa (based on the net area). The minimum compressive strength for masonry cement mortar is specified as 10 MPa.

a) b)

Figure 2. Key features of confined masonry construction: a) foundation construction (note tie-column reinforcement cages extending from the foundations), and b) wall construction.

RC tie-columns are usually built at 3 to 3.5 m spacing, although NCh 2123 permits the maximum spacing of 6 m. Typical cross-sectional dimensions range from 140 to 200 mm (length), while the depth is equal to the wall thickness (usually 140 mm). A toothed interface between the walls and the tie-columns was observed in new and existing buildings, as shown in Figure 3 b. Toothing enhances an interaction between masonry walls and RC confining elements. The longitudinal reinforcement in tie-columns consists of four bars (8 to 10 mm diameter) - note that NCh 2123 prescribes the minimum bar size of 10 mm. The ties are typically provided at 150 to 200 mm spacing. In most cases where the ties have been exposed, the spacing was found to be uniform up the tie-column height, although NCh 2123 prescribes a closer spacing in the end zones. The tie size ranges from 4.2 mm diameter (for prefabricated reinforcement cages) to 6 mm (for reinforcement cages assembled at the site). RC tie-beams have similar reinforcement and dimensions like tie-columns. In buildings with timber roofs, the width of tie-beams at the roof level exceeds the wall thickness (e.g. 200 or 250 mm wide tie-beams and 140 mm thick walls).

a) b)

Figure 3. Confined masonry construction: a) a typical hollow clay unit, and b) a toothed wall-to-tie-column interface.

Typical steel grade used for the RC confining elements is A44-28H (280 MPa yield strength), however in recent years A63-42H steel grade has also been used (420 MPa yield strength). In some cases, prefabricated reinforcement cages are used; these are fabricated using high-strength steel of AT56-50H grade (500 MPa yield strength). The minimum specified concrete strength for confining elements is 16 MPa (based on the cylinder strength).

Note that the above description are related to confined masonry buildings of recent vintage (1990s and newer). The characteristics of older buildings of the 1950s and 1960s vintages are somewhat different. A few damaged buildings of older vintage were observed in Santiago, but most of the damaged buildings outside the metro Santiago area were of more recent vintage.

2. Damage Observations

By and large, confined masonry buildings performed very well in the February 2010 earthquake. Most one- and two-storey single-family confined masonry dwellings did not experience any damage, with the exception of a few buildings which suffered moderate damage. A similar statement can be made for three- and four-storey confined masonry buildings: large majority of buildings remained undamaged, however a few buildings suffered severe damage, and two three-storey buildings collapsed. The key damage patterns are discussed below.

2.1. Masonry Walls

The most common damage pattern observed in masonry walls was <u>in-plane shear cracking</u>. This type of damage was mostly observed at the bottom floor level of three- and four-storey buildings, as shown in Figure 4 a and b. In a very few cases, this damage pattern was observed in single-family dwellings, again at the first floor level of a two-storey building, as shown in Figure 4 c. Shear cracking was observed in walls built using all types of masonry units, including hollow clay tiles, clay bricks, and concrete blocks. Note that the walls in the building shown in Figure 4 a and b were built using hollow clay blocks, while the wall shown in Figure 4 c was built using clay bricks.

a)

b) c)

Figure 4. In-plane shear failure of masonry walls: a) a three-storey building in Cauquenes experienced severe damage at the ground floor level; b) a closer look into damaged walls in the same building, and c) shear cracking in a brick masonry wall of a two-storey single family house.

A few instances of <u>out-of-plane wall damage</u> were observed. Most notable example was a three-storey building in Cauquenes in which the damage was concentrated at upper floors, as shown in Figure 5. The building had RC floors and a timber truss roof. Cracks in the third-floor wall panel extended into the RC tie-beam, as shown in Figure 5 b (note that the tie-beam is wider than the wall). The out-of-plane damage was observed in the transverse direction of the building. The same building suffered extensive damage in the longitudinal direction at the ground floor level (see Figure 4).

a) b)

Figure 5. Out-of-plane damage of masonry walls in a three-storey building: a) second floor level, and b) third (top) floor level.

2.2. Reinforced Concrete Confining Elements

Failure of RC tie-columns was observed in three- and four-storey apartment buildings in which masonry walls suffered severe damage at the ground floor level. Absence of ties at the tie-beam-to-tie-column joint, and inadequate anchorage of longitudinal reinforcement caused shear failure at the tie-column ends (see Figure 6 a). Buckling of longitudinal reinforcement was observed where size and/or spacing of ties at the tie-column ends was inadequate and the masonry crushing was severe, as shown in Figure 6 b. The buckling took place due to excessive axial compression stresses in tie-columns which developed after the masonry was crushed at the wall toe.

a) b)

Figure 6. Failure of RC tie-columns: a) shear failure at the ends of a RC tie-column (note an absence of ties in the joint area), and b) buckling of longitudinal reinforcement at the base of a RC tie-column.

Figure 7 shows details of tie-column damage in a collapsed three-storey building in Santa Cruz. The building lost its first floor and the photo shows damage at the second floor level. The masonry wall panel experienced shear failure, and caused the failure at the ends of adjacent tie-columns.

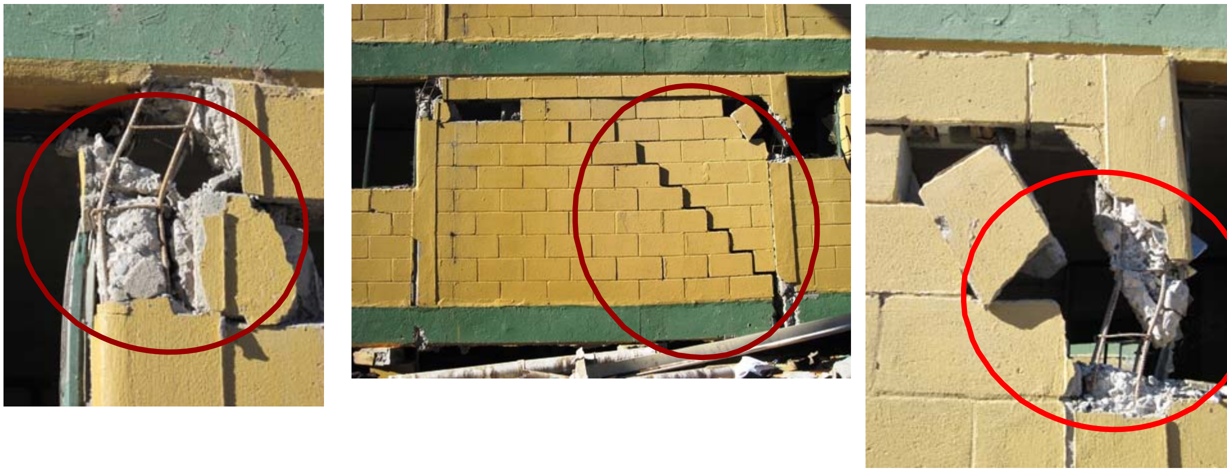


Figure 7. Failure of RC tie-columns caused by the shear failure of the masonry wall panel.

2.3. Performance of Multi-Story Confined Masonry Buildings

Evidence from past earthquakes and findings of experimental studies have confirmed that earthquake-induced lateral forces in multi-storey confined masonry buildings peak at the ground floor level, thereby causing significant shear cracking. The collapse of confined masonry buildings subjected to severe ground shaking may take place due to a soft story effect similar to that found in RC frames with masonry infills, as illustrated in Figure 8.

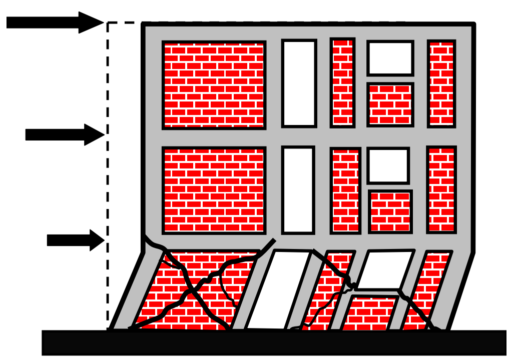


Figure 8. Soft-story collapse mechanism for multi-story confined masonry buildings (Alcocer et al., 2004).

For the first time in Chile, two three-story confined masonry buildings collapsed in an earthquake after developing a soft story mechanism. One of the collapsed buildings was located in Santa Cruz, where the maximum observed seismic intensity on the MSK scale was 7.5, out of maximum 9.0 reported in the earthquake-affected area (Astroza et al., 2010). The building was a part of the building complex consisting of 32 identical buildings (two rows of 16), as shown in Figure 9 a. Several factors influenced seismic performance of this building and likely led to its collapse, such as inadequate wall density (less than 1 % calculated on a floor basis) combined with low-strength masonry walls, absence of RC tie-columns at the openings, and inadequate size of RC tie-columns. In this building, exterior walls were built using hollow concrete blocks, and the interior ones using hand-made solid clay bricks. Wall thickness was 150 mm and the size of RC tie-columns was 150 by 150 mm. The collapsed building lost its ground floor, as shown in Figure 9 b.

a) b)

Figure 9. Collapse of a three-story confined masonry building in Santa Cruz: a) building complex, and b) a building that collapsed at the ground floor level.

The other building was located in Constitución, a city affected both by the earthquake and the subsequent tsunami. The maximum observed seismic intensity on the MSK scale was 9.0 (significantly higher than Santa Cruz) (Astroza et al., 2010). The collapsed building was a part of a complex which included three buildings (A, B, and C) built on the hill, close to a steep slope, as shown in Figure 10. Building C located closest to the slope (5 m distance on the west side) collapsed, while buildings A and B suffered damages of various extent. It appears that several owner-driven renovations took place before the earthquake - for example, a door opening appears to had been made in a load-bearing masonry wall at the ground floor level of the collapsed building.

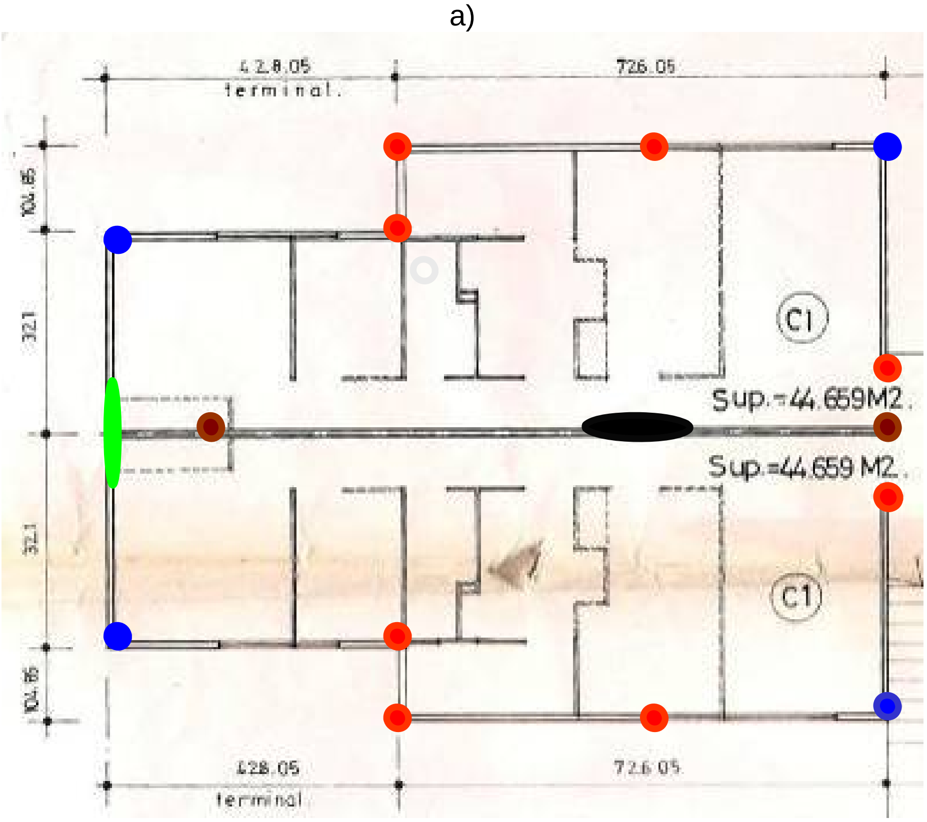
a) b)

Figure 10. Collapse of a three-story confined masonry building in Constitución: a) an aerial view of buildings A, B and C, and b) a steep slope on the west side of the building complex.

The collapsed building shifted by approximately 1.5 m in the north direction (towards the slope), as shown in Figure 11 a. Note that building B experienced more extensive damage than building A. The damage in all buildings was more pronounced in the north-south direction (transverse direction of the building plan). The collapse was initiated by the shear failure of the transverse walls, which was followed by the out-of-plane failure in the longitudinal walls (horizontal cracks due to out-of-plane seismic effects were observed in the longitudinal wall of building B). The walls were constructed using 140 mm thick hollow clay block units. RC tie-columns had different cross-sectional dimensions depending on the location: length ranged from 140 to 200 mm, and depth was equal to the wall thickness. In addition, a few wide RC columns (700 to 900 mm long and 140 mm wide) lightly reinforced with nominal vertical and horizontal reinforcement were also found in these buildings.

These wide columns replaced tie-columns at some locations (this practice is followed in medium-rise confined masonry construction in Chile). A typical floor plan is shown in Figure 11 b. It is believed that the building location and geotechnical effects, in addition to a low wall density in the north-south direction (less than 1 % calculated on a floor basis), were the key factors contributing to the collapse.

a)



RC Tie-Columns:
P1= 15x14 cm
P2 = 20x14 cm
P4 = 15x15 cm
P5 = 70x15 cm
P6 = 90x14 cm

b)

Figure 11. A collapsed three-story confined masonry building in Constitución: a) building C (located closest to the slope) lost the ground floor and shifted by approximately 1.5 m away from the plinth towards north, and b) a typical floor plan for Building C showing the locations of RC tie-columns.

3. Causes of Damage

Damage in confined masonry buildings affected by the February 2010 earthquake can be attributed to one of the following major causes:

- Inadequate wall density,
- Inadequate quality of masonry materials and construction,
- Deficiencies in detailing of reinforcement in RC confining elements,
- Absence of RC confining elements at openings, and
- Geotechnical issues.

3.1. Inadequate Wall Density

Excessively low wall density was one of the main causes of damage and/or collapse in three- and four-storey apartment buildings. Wall density is the key parameter that influences seismic performance of confined masonry buildings. It can be determined as a ratio of the cross-sectional areas of walls in one principal direction and the total floor area of the building. The required wall density depends on the seismic hazard, masonry shear strength, type of soil, and the number of stories. Preliminary estimates of wall density ratios for the collapsed and severely damaged confined masonry buildings indicate that wall density was on the order of 0.7 to 0.8% per floor. The amount of walls thus appears to be excessively low for buildings located in high seismic hazard area (peak ground acceleration of 0.4g or higher) having walls built using hollow clay blocks. EERI (2010) recommends wall density ratio in the range of 2 to 3% for one-storey buildings with these characteristics (and would need to be substantially higher for a three-story building).

3.2. Inadequate Quality of Masonry Materials and Construction

A substandard quality of masonry construction was observed in a few severely damaged buildings. The quality of hollow clay blocks (hollow clay tiles) was generally found to be good. Poor quality of hollow concrete blocks was observed in a few damaged apartment buildings, as shown in Figure 12 a. In some instances, excessively thick mortar bed joints (on the order of 30 mm) were observed in brick masonry walls, as shown in Figure 12 b. Hand-made clay bricks used for wall construction in these cases were characterized by a substantially lower strength compared to machine-made bricks. As a result, it is expected that the compressive and shear strength of such masonry is very low.

a)

b)

Figure 12. Poor quality of masonry construction: a) low-strength concrete blocks, and b) excessively thick mortar bed joints in brick masonry construction.

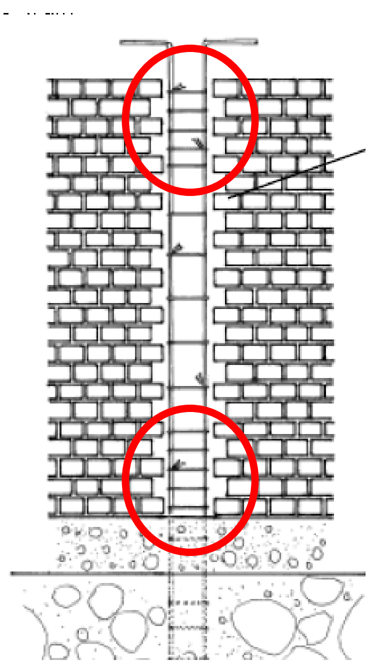
3.3. Deficiencies in Detailing of Reinforcement in RC Confining Elements

In general, the concrete quality was found to be satisfactory, with a few exceptions. Exposed RC confining elements (tie-beams and tie-columns) in damaged buildings provided an opportunity to examine detailing of reinforcement in these elements. A few important deficiencies were observed, including i) inadequate confinement at the ends of tie-columns, ii) absence of ties in the joint region, and iii) discontinuous longitudinal reinforcement at the RC tie-beam intersections.

Inadequate confinement at the ends of RC tie-columns was observed in most cases when these regions were exposed due to earthquake damage, as shown in Figure 13 a. According to the NCh 2123, a closer tie spacing of 100 mm is required in tie-column end regions compared to that in the middle portion of the column (200 mm). A typical detail showing tie spacing requirements for end regions is shown in Figure 13 b. This practice was not observed in any of the damaged buildings. A possible reason is the use of prefabricated reinforcement cages with uniform tie spacing up the tie-column height. An enhanced confinement in the end zones of RC tie-columns is expected to prevent a premature buckling when increased axial compression stresses develop in localized areas where masonry is completely disintegrated, as discussed in Section 2.2.

Absence of ties in the joint region was observed in all cases where joints were exposed (see Figure 14 a). This deficiency could cause a shear failure in the joint region, as shown in Figure 14 b. Chilean codes do not contain specific requirements for providing additional ties in the joint region, which could be similar to a detail shown in Figure 14 c.

a)

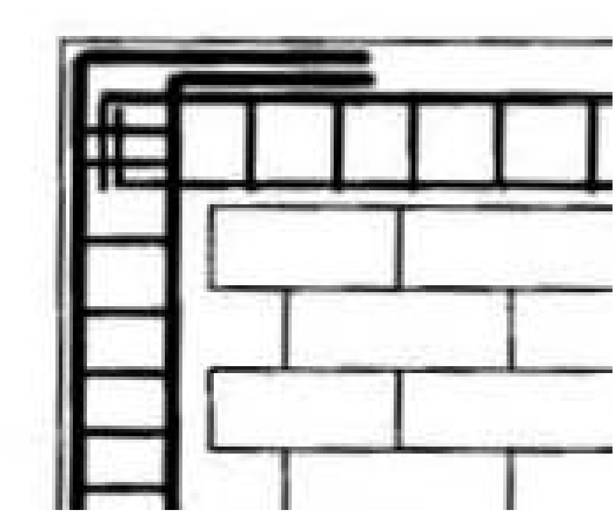


b)

Figure 13. Inadequate confinement in the end zones of RC tie-columns: a) buckling of longitudinal reinforcement (note an excessively large tie spacing), and b) a closer tie spacing prescribed at the ends of RC tie-columns (Blondet, 2005).

a)

b)



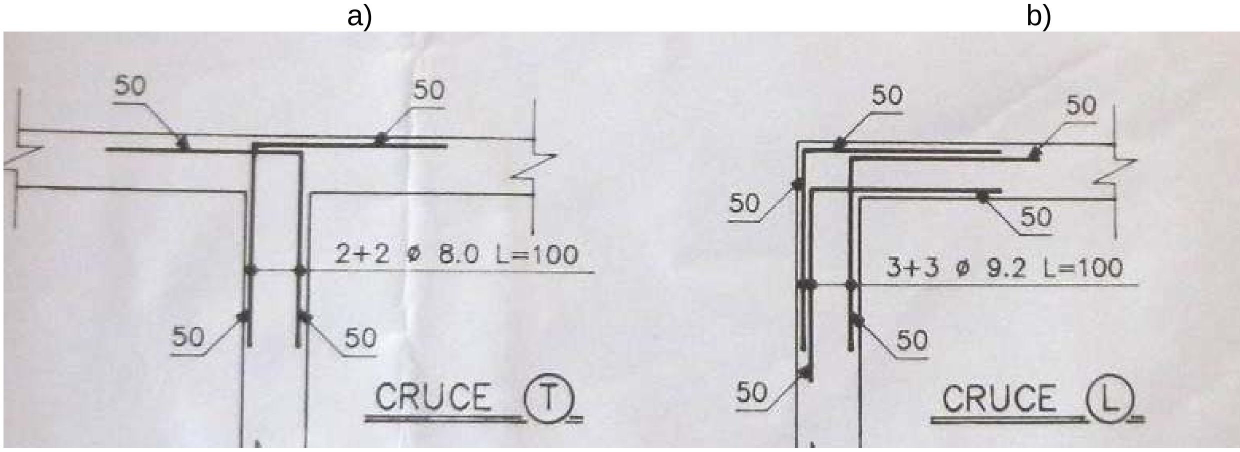
c)

Figure 14. Absence of confinement in the tie-beam-to-tie-column joint region: a) interior tie-column; b) exterior tie-column, and c) drawing detail showing additional ties in the joint region (Credit: A. San Bartolomé).

Discontinuous longitudinal reinforcement at the RC tie-beam intersections is shown in Figure 15 a and b. This is contrary to Chilean detailing practice for this location, as illustrated by a drawing detail shown in Figure 15 c. It should be noted that reinforcement cages for tie-beams and tie-columns are often assembled off the building site, however additional “continuity reinforcement” should be provided once the cages are placed in the final position.

a)

b)

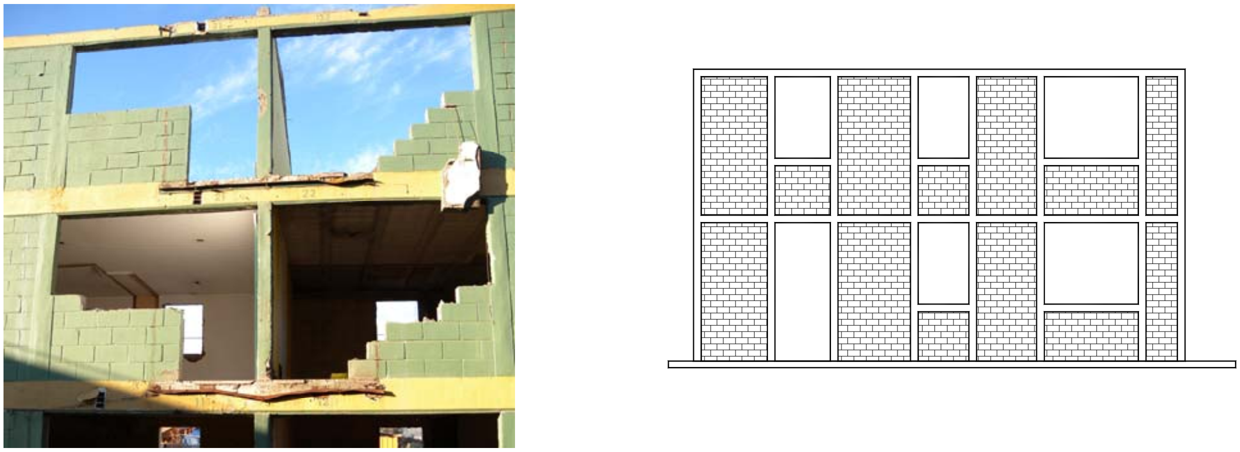


c)

Figure 15. Inadequate anchorage of tie-beam reinforcement: a) typical "corner" building; b) tie-beam intersection showing a discontinuity in tie-beam reinforcement, and c) a drawing detail showing "continuity reinforcement" at the intersection (plan view) – for a different building.

3.4. Absence of RC Confining Elements at Openings

Absence of RC confining elements (tie-columns) at openings was observed in several buildings, as shown in Figure 16 a. This resulted in more extensive damage of masonry piers between the openings, and is contrary to the NCh 2123 requirements and recommendations of other design documents (e.g. EERI, 2010), as illustrated in Figure 16 b. Presence of RC tie-columns at openings enables the development of compressive struts in masonry wall panels. This is the key mechanism for lateral load transfer in confined masonry walls. Masonry wall panels without RC tie-columns at both ends are not considered to be confined, and are not to be considered in wall density calculations (EERI, 2010).



a) b)

Figure 16. RC confining elements at openings: a) absence of confining elements, and b) confined openings (EERI 2010).

The effect of confinement at the ends of openings can be observed on an example of two apartment buildings located in Santiago. The building shown in Figure 17 a had RC tie-columns at the ends of the openings (note the concrete in the tie-column at the right was formed to mimic brick masonry appearance). The building shown in Figure 17 b had RC tie-column in the middle of the pier. The first building experienced moderate cracking, while the other building experienced severe cracking in almost all piers at the first storey level. Both buildings survived the earthquake and were inhabited at the time of the visit, in spite of apparent damage.

a) b)

Figure 17. In-plane shear cracking of piers in brick masonry walls: a) a confined masonry panel with RC tie-columns at both ends (note RC tie-column highlighted with a black ellipse), and b) unconfined opening (note RC tie-column at the middle of the pier highlighted with a red ellipse).

3.5. Geotechnical Issues

Geotechnical effects have contributed to damage and/or collapse of at least two confined masonry apartment buildings. The collapsed three-storey building in Constitución was described in Section 2.3. Building C located closest to the steep slope collapsed, while buildings A and B suffered damage of various extent. A localized influence of the unrestrained slope boundary and localized variations in sub-surface strata might have generated localized variations of horizontal (and possibly vertical) ground accelerations at the building site (Fannin, 2010). It appears that, before the earthquake, Chilean regulations did not prevent building construction in proximity of major slopes, however this has changed after the earthquake when a new emergency regulation was issued.

There was another building site in Constitución where geotechnical issues likely caused severe damage in confined masonry buildings. A complex of several three-storey buildings called Centinela was built in 1992 (750 apartments in total). The buildings were built on a moderate slope, in the proximity of a creek. All buildings, except for the one located closest to the creek (building B), remained undamaged and were inhabited at the time of the visit. Building B experienced severe damage, in the form of cracking in the walls and also floors at the ground floor level. Note that an identical building complex in other part of the city remained undamaged. These buildings were government-built (social housing), and construction quality issues were reported before the earthquake.

a) b)

Figure 18. A building in Constitución experienced severe damage: a) building B located closest to the creek located on the right side of the building, and b) extensive exterior and interior damage in the building.

4. Conclusions

By and large, confined masonry buildings performed very well in the February 2010 earthquake. With the magnitude of 8.8, February 27, 2010 earthquake is the most severe earthquake on the global scale which exposed confined masonry buildings to severe ground shaking. As of this writing, the final statistics on the total number of buildings affected by the earthquake and their typology is not available, but it is estimated that only about 1% of the total building stock was affected by the earthquake. This is considered to be a very good performance record for all building types, considering the earthquake magnitude and the spread of ground shaking over a large populated area of the country.

Most one- and two-storey single-family confined masonry dwellings did not experience any damage, except for a few buildings which suffered only a moderate damage. A similar statement can be made for three- and four-storey confined masonry buildings: large majority of buildings remained undamaged, however a few buildings suffered severe damage, and two three-storey buildings collapsed.

For the first time in Chile, three- and four-storey confined masonry buildings were exposed to severe ground shaking in the February 2010 earthquake. Lessons learned from the detailed studies on collapsed buildings and other deficiencies related to confined masonry construction observed in the earthquake-affected area will benefit design provisions and lead to improvements in construction practices of confined masonry buildings in Chile and other countries.

Acknowledgments

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