CONFINED MASONRY BUILDINGS: THE CHILEAN EXPERIENCE

M. Astroza[(1)], F. Andrade[(2)], M.O. Moroni[(3)]

[(1)] *Universidad de Chile, mastroza@ing.uchile.cl*
[(2)] *Universidad de Chile, fandrade@ing.uchile.cl*
[(3)]*Universidad de Chile. mmoroni@ing.uchile.cl*

Abstract

Confined masonry construction was introduced in Chile in the late 1930s. Confined masonry structures had a great performance during the 1939 Chillan earthquake, providing the first real test for this type of construction during large earthquakes. At that time, the Ordenanza General de Construcciones included some design requirements for low-rise confined masonry buildings up to 2 stories high. A more rational code based on the allowable stress method was published in 1997 and provided design requirements for buildings up to 4 stories high. The confined masonry structures built in Chile in the last 20 years have followed the prescriptions of this code.

The great earthquakes that hit the central and north part of Chile in 1985, 1987, 1997, 2010, 2014 and 2015 have shown the excellent behaviour of these structures subjected to seismic loading, when their design fully satisfied the code requirements. Contrarily, the use of partially confined masonry walls, built with a tie-column at one end of the masonry panel and a vertical tensile bar at the other end, may lead to collapse or, in many cases, to significant damage.

Key components of confined masonry buildings are the horizontal (tie-beam) and vertical (tie-column) reinforced concrete elements. These components are cast in place after the masonry wall panels are built. Typical values for shear strength of Chilean masonry are between 0.5 and 1.0 MPa and the average value of wall density index (ratio between the cross-sectional areas of all walls in one direction and the total floor area of the building) is about 3.5%.

Analytical models to simulate the seismic behaviour of confined masonry buildings have been calibrated with low-amplitude vibration data (i.e., ambient vibrations) and seismic records obtained at the Community Andalucía Building, the only confined masonry building instrumented with an accelerometer network in Chile.

Keywords: confined masonry; seismic behavior; codes; Chile; mega earthquakes

1. Introduction

Reinforced concrete (RC) and masonry are the most used materials for housing construction in Chile (see Fig 1). Masonry is used especially for social dwellings built in all regions of Chile. During economic crisis and after the 2010 Mw=8.8 Maule earthquake the construction activities decreased considerably. In addition, since 2013 the government stopped the construction of social dwellings and funds dedicated to that item are now distributed directly to people who buy housing to private companies. This may explain the reduction in masonry compared to RC construction, built-up area expressed in square meter.

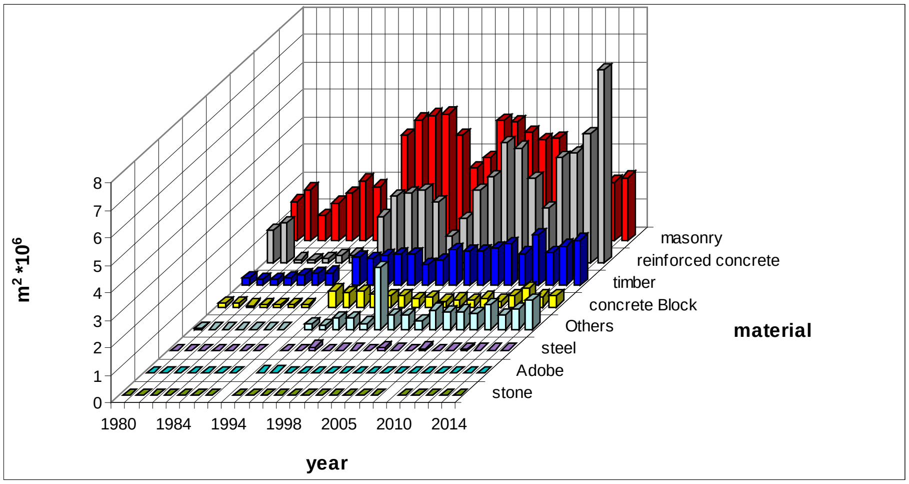


Fig. 1 Built-up area per year according to type of material (Source: INE)

On the other hand, Chile has a long history of confined masonry construction practice. The use of confined masonry in Chile started in the 1930s, after the 1928 Talca earthquake (M_w 7.7), for construction of low-rise single-family housing. Construction of medium-rise apartment buildings started in the 1970s in Santiago, the capital city; it spread out in the 1990s to other urban areas along the country. For instance, between 1992 and 2002 about 10% of the total social dwellings built in the Region VI used confined masonry walls [1].

Good performance of low-rise confined masonry buildings during the 1939 Chillan earthquake (M 7.8) paved the road for the widespread use of this type of construction in Chile. Low-rise confined masonry construction maintained a good performance record in past earthquakes, including the 1985 Llolleo earthquake (M_w 8.0). A limited number of medium-rise confined masonry buildings were located in the epicentral zone of the 1985 earthquake, thus it can be considered that three- and four-story confined masonry buildings had not been exposed to severe ground shaking in Chile prior to the February 2010 earthquake (M_w 8.8).

Usually, Chilean masonry buildings have a regular and symmetric structural configuration and a rectangular floor plan with walls distributed in both orthogonal directions. Story height varies between 2.20 and 2.30 m. Average wall density index (ratio between the cross-sectional areas for all the walls in one direction and the total floor area of the building) is about 3.5%.

This paper presents an overview of the use of confined masonry in Chile, some historical remarks, the development of the Chilean Design Code, and a discussion on their performance during strong earthquakes based on field inspection and analysis of records obtained at an instrumented building. Finally, present and future challenges associated with the application of confined masonry in Chile are discussed.

2. Main Components of Confined Masonry Buildings

The main components of confined masonry buildings are masonry panels and RC confining elements (tie-columns and tie-beams). Unreinforced masonry panels are constructed first, one story at a time, followed by the cast in-place RC tie-columns, as shown in Fig. 2a. Finally, RC tie-beams are constructed on top of the masonry panel, simultaneously with the floor/roof slab construction. The wall thickness depends on the type of masonry units used. Most common masonry units used in Chile for confined masonry walls are machine–made multi-perforated clay brick (usually 140 mm wall thickness), followed by the hand-made solid clay bricks (usually 150 mm wall thickness). Hollow concrete block units (usually 150 mm wall thickness) are rarely used because its high permeability and low bond between mortar and the unit have caused serious moisture problems and poor seismic behavior [2].

A toothed interface between the masonry panel and tie-columns is used, as shown in Fig. 2b, improving the integration between masonry panel and RC confining elements and preventing vertical cracking at the wall ends or out-of-plane collapse. The construction sequence, presence of toothing, and size and detailing of RC confining members are the main differences between confined masonry and RC frame construction with masonry infill walls.

(a)

(b)

Fig. 2 – Confined masonry construction: (a) Masonry wall is built first and (b) a toothed wall-to-tie-column interface

A mixed of cement, hydraulic lime, and sand is used as mortar. Rules governing its properties were approved in 2001. Before that, the lack of regulations led to use inappropriate mortars, affecting the bond mortar-unit and causing damages during even moderate earthquakes. The introduction of premixed mortars was another step in the right direction.

RC confining members have an important role in enhancing the overall building stability and integrity. These members can effectively contain damaged masonry walls in-plane and out-of-plane, and ensure adequate wall connections to adjacent floors/roofs and foundations. RC tie-columns are spaced at 3.0 to 3.5 m, although Chilean code NCh2123 [3] allows a larger spacing up to 6.0 m. Tie-columns have a rectangular cross section whose dimensions typically correspond to the wall thickness (150 to 200 mm) and a depth equal to 200 mm. Both tie-columns and tie-beams are reinforced with a minimum of four 10 mm diameter longitudinal reinforcing bars and 6 mm diameter stirrups spaced at 100 mm at top and bottom of the tie-column and at 200 mm in the middle.

In buildings with flexible diaphragms, e.g. timber roofs, the width of tie-beams at the roof level exceeds the wall thickness, e.g. 200 or 250 mm, to prevent the occurrence of an out-of-plane failure mechanism. Typical steel grades used for the RC confining elements are A44-28H (280 MPa yield strength) and A63-42H (420MPa yield strength) for the reinforcing cages assembled at the site, while AT56-50H grade (500 MPa yield strength) is used for the prefabricated reinforcing cages.

When any dimension in the building plan is longer than 20.0 m, RC walls at least 1.0 m long must be located at each end to avoid cracking in the masonry panels due to shrinkage of reinforced concrete slabs.

Generally, floor systems consist of cast-in-place RC slabs with a thickness between 100 and 120 mm or precast slabs with large hollow-masonry blocks laid horizontally between precast RC beams. Usually, unreinforced concrete foundations are used, unless in the presence of soft soil conditions (e.g., clay or silty soil), RC plinths are used to reduce settlements.

3. Code Development

A committee appointed by the government after the 1928 Talca earthquake, issued the first design guideline for this type of construction in 1931 [4]. This document regulated the construction of 1 or 2 story confined masonry dwellings, requiring that masonry walls should include RC elements and limiting the maximum spacing between tie-columns and tie-beams and their minimum dimensions.

Subsequently, new knowledge in structural seismic design and the experience gathered after the 1958 Las Melosas and the 1960 Valdivia earthquakes, led to the publication of the Code NCh433.Of72, Standard for Seismic Design of Buildings. This code limited the allowable masonry shear stress to 0.5 MPa plus 10% of compressive stresses produced by the gravitational permanent loads. In the case of hollow masonry units, stresses should be calculated considering the effective contact area.

After the 1985 Llolleo earthquake, the Chilean code NCh433 was modified and a new version was published in 1986. During its discussion, it was recognized that the provisions of the Ordenanza [5] were insufficient to design confined masonry buildings of 3- and 4-story high. Therefore, an appendix with recommendations for the design of any confined masonry building, based on the Mexican and Argentinian standards, was included in the draft. Finally, the appendix became the basis document of the current code NCh2123.Of93 “Confined Masonry - Requirements for the design and calculation” [6].

The NCh2123 code specifies, among other prescriptions: (a) the allowable shear load capacity of a confined masonry wall, based on the basic masonry shear strength of the masonry and the vertical load applied on it, (b) the size and the minimum quantity of longitudinal reinforcement, (c) the amount and spacing of stirrups in confining elements, (d) the minimum wall thickness, (e) the tie-column spacing, (f) the required reinforcement elements around the panel openings. The allowable shear load capacity is about fifty percent of the diagonal cracking panel load.

Experiments show that confined masonry walls present a performance similar or better than reinforced masonry walls, have larger deformation capacity after reaching the diagonal cracking load when the crack propagation into the tie-columns is avoided. Based on this evidence, the elastic demand reduction factors for confined masonry buildings are larger than those for reinforced masonry buildings with partial filling of voids.

In general, Chilean confined masonry buildings are quite stiff and have regular floor plan with a high wall density. Depending on the seismic zone, they must resist a base shear of at least 10 to 22% of the total seismic building weight.

4. Earthquake Performance

Seismic behavior of masonry buildings depends mainly on the quality of the materials (mortar and units) and the type, amount, and distribution of reinforcements. However, in recent years, it has become evident that the effect of the focal mechanism of the earthquake also plays an important role. In effect, intraplate intermediate depth earthquakes produced more damage due to its high frequency content and higher amplifications of seismic waves than subduction interplate earthquakes [7-10].

The 1939 Chillán earthquake, an intermediate depth intraplate earthquake, produced the collapse of almost all unreinforced masonry buildings built since the early twentieth century in the epicentral area of the earthquake [11]. This situation has not been observed during interplate earthquakes, such as 1906 Valparaiso earthquake [12], 1928 Talca earthquake [13], 1960 earthquakes in southern Chile [14] or 1985 Llolleo earthquake [15], in which many low-rise unreinforced masonry buildings resisted the shaking and are standing until today. Although unreinforced masonry buildings are not currently constructed, it is useful to study its behavior in order to establish the seismic demands for rigid and brittle buildings, such as masonry buildings, when small amount of reinforcements are used.

More recently, the 2010 Maule earthquake was the first Chilean experience about the behavior of medium-rise confined masonry buildings, in which a few of them collapsed. Based on field observations, the behavior in modern masonry buildings is controlled by the shear capacity regardless of the type of reinforcement, because the architectural design facilitates the use of short and strongly coupled walls. The type of reinforcement makes a difference in the deformation capacity after the diagonal cracking occurs.

More information on the seismic behavior of confined masonry structures during the most important Chilean earthquakes in the last 75 years is given below:

1939 Chillán earthquake. This earthquake corresponds to a subduction intermediate depth intraplate earthquake with a magnitude 7.8 [16] and it was one of the most destructive events occurred in Chile [7]. According to a damage assessment conducted by a commission appointed by the government at that time [17], the observed behavior of one-story masonry buildings in Chillan, located in the epicentral area, is summarized as follows:

• Out of a total of 154 dwellings confined masonry, representing 4.5% of the total housing, 83 were in good condition, 49 damaged, 5 semi-destroyed and 17 totally destroyed, i.e, 54% experienced no damage. Fig. 3 shows a house of this type currently in use.

• The adobe and unreinforced masonry brick houses, representing 86.8% of households in the city, were seriously affected. Regarding adobe houses, 764 were damaged, 177 semi-destroyed, 1240 demolished, i.e., out of a total of 2181 houses, 100% were affected and 57% of them seriously damaged. This situation was similar for unreinforced masonry houses, resulting 44% of 844 homes collapsed.

Fig. 3 - Old confined masonry house currently in use in Chillan [11]

1965 La Ligua earthquake. This earthquake corresponds to an intraplate earthquake of intermediate depth of magnitude 7.1 with the epicenter about 140 km from Santiago [18]. Again, the effects of the earthquake were severe in adobe and unreinforced masonry houses, the latter behaving particularly bad, even though most of them were reinforced with concrete tie-beams at the top of the walls forming a horizontal grid. About 21,000 masonry houses collapsed and 71,000 had to be repaired. This was an unexpected behavior for one-story masonry buildings; although most of them did not have tie-columns at ends of masonry panels and at wall intersections. The results were especially bad in houses made of hollow concrete blocks, units that started to be used in those years. Typical patterns of damage were: in-plane shear failure, cracks by out-of-plane flexure, vertical cracks by lack of bond between masonry and concrete elements, damages in beam-column joints and vertical cracking at wall intersections. Some of the observed damage is shown in Fig. 4.

Fig. 4 - Observed damage in concrete block masonry panel during 1965 La Ligua earthquake

1971 Papudo earthquake. This event was an interplate earthquake of magnitude 7.5, which hit mainly the Region V of Chile, affecting practically the same zone of the 1965 La Ligua earthquake. Again, masonry houses were damaged and of particular interest was the case of the destruction of about 1,000 houses located in the valley of the Choapa River (see Fig. 5). The walls of these one-story houses were built with hand-made clay bricks, reinforced at the top of the walls with RC beams and with some vertical tensile steel bars placed at the wall intersections, within holes perforated in the bricks. The effects of poor quality of soils [19] and inappropriate design of the foundations made the damage more severe.

Fig. 5 - Observed damage during 1971 Papudo earthquake

1985 Llolleo earthquake. This event corresponds to an interplate earthquake of magnitude 7.8. Its effects extended from Illapel to Talca, causing 147 victims and 2,000 injured in an exposed population of six million people. In the housing sector, about 66,000 houses were destroyed and 127,000 were damaged, especially in rural area and old part of cities. For instance, in Melipilla city there were 10,800 houses damaged, 7,560 of them were adobe houses, leaving 89% of the population homeless [15].

After the earthquake, the Ministry of Housing appointed a special committee to review the seismic effects on social dwellings. About 84,000 units were reviewed, mostly located in Santiago, concluding that 50% of them suffered structural damage. Confined masonry buildings represent 27% of the total housing reviewed and 74% (9,928 units) of them were slightly damaged, but no collapses occurred.

Most of the masonry buildings with severe damage were three- or four-story high and they did not have tie-columns at one wall end or around window openings. In many cases, the only reinforcement was one steel vertical bar located inside of a hollow unit (see Fig. 6), which did not satisfy the minimum recommended by the reinforced masonry code UBC-1979. This occurred because prior to 1986, there was no Chilean design code for reinforced and confined masonry buildings.

Fig. 6 - Observed damage during 1985 Llolleo earthquake in the second floor of building located in Santiago

For this earthquake the observed damages included:

- Diagonal stepped bed-joint cracking in the masonry panel due to a bad bond between mortar and units.

- Propagation of panel diagonal cracks into top and bottom of tie-columns.

- Crushing of hollow masonry units with large percentage of voids and thin shells and webs in the most stressed zones of the masonry panel.

-Horizontal cracking at the joint between masonry wall and the RC floor slab or foundation

-Cracking in walls due to out-of-the-plane seismic loads when the masonry panels were not properly confined or the separation between tie-columns was too large.

- Cracking, crushing and disintegration of concrete at the tie-beam/tie-column connections when reinforcement detailing was inappropriate.

- Inadequate quality of masonry materials (mortar, grout) and poor workmanship.

1997 *Punitaqui earthquake:* This intermediate depth intraplate earthquake, magnitude Mw = 7.1 [20] caused damage in the IV Region, especially in adobe houses. Damage to masonry houses was reduced in number and severity. According to information provided by the Regional Municipality of Region IV [21], from a total of 839 masonry houses, 11.4% were destroyed, 26.1% had risk of collapse, 11% had moderate damage, and 51.5% had light or no damage. This statistic does not distinguish the type of reinforcement and relates mainly to hybrid type construction. A damaged one-story house with little reinforcement is shown in Fig. 7.

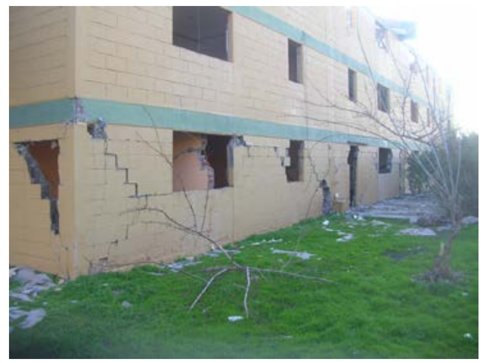
Considering the magnitude of the event, the amount of damage was higher than expected due to the mechanism that generated the earthquake, confirming what was observed earlier in earthquakes of this type, i.e., the 1939 Chillán and 1965 La Ligua earthquakes.

Fig. 7 - One-story house damaged during 1997 Punitaqui earthquake

2010 Maule earthquake. Overall, the behavior of confined masonry buildings was very good in this M=8.8 interplate earthquake. Most one- and two-story single-family confined masonry dwellings did not experience any damage, with the exception of a few buildings which suffered moderate damage. A large majority of three- and four-story confined masonry buildings also performed well, however a few of them suffered moderate to severe damage.

A few medium-rise confined masonry buildings of older vintages suffered moderate and repairable damage. For example, diagonal cracking in masonry walls was observed in four-story buildings in Huemul II complex, located in Santiago and built in 1947; the same damage occurred in the 1985 Llolleo earthquake and was subsequently repaired [22]. Similar damage patterns were observed in the complex of four-story buildings called Roto Chileno (built in Santiago in 1960) and in the Costanera Norte complex, built in Talca in 1956. The masonry walls of these old buildings were built using hand-made solid clay bricks and a low-strength mortar (cement:lime:sand mix ratio 1:1:6). In addition, RC tie-columns were provided only at wall intersections but not around the openings, thus these old buildings can be considered as "partially confined". Examples of severe damaged partially confined masonry buildings are shown in Fig. 8.

Constitución



Santa Cruz

Fig. 8 - Observed damage during 2010 Maule earthquake

On the other hand, some modern multi-story partially confined masonry buildings had an anticipated bad behavior and experienced significant damage at the ground floor level [23]. For the first time in Chile, two three-story partially confined masonry buildings collapsed in an earthquake, causing the death of six adults and four children. Several complexes built with this type of masonry walls had to be demolished in different locations, such as Santa Cruz, Constitución, and San Antonio.

2014 *Iquique earthquake.* Although this was an 8.4 magnitude earthquake, in general damage was rather limited. Regarding confined masonry buildings, buildings with non-appropriate reinforcement located on non-competent soils suffered damage. For instance, in complex “Los Alelies” (5-story buildings), some partially confined walls were damaged specially in those buildings located in non-competent soils, and in Edificio Los Cóndores the damage occurred in short columns [24]. In complex Dunas I, four story-high building experienced damage at the first floor due to the presence of a central opening without any confinement [25].

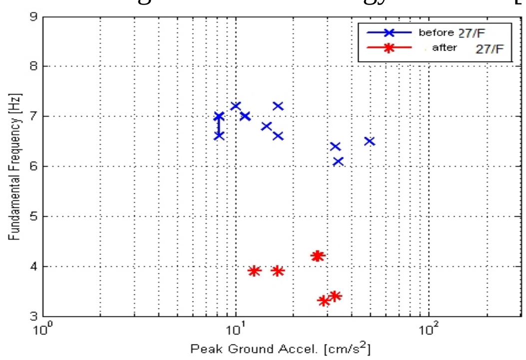
2015 *Illapel earthquake.* Again the damage in masonry buildings were rather scarce in this M=8.4 interplate earthquake and more limited than damage observed in the 1985 Llolleo earthquake. A similar situation was also observed in this zone during the interplate earthquakes of Illapel 1943 and La Serena 1975 [26], and can be correlated with the variation of subduction angle of the Nazca plate beneath the continental plate [27, 28].

5. Analysis of microtremor and earthquakes records in instrumented buildings

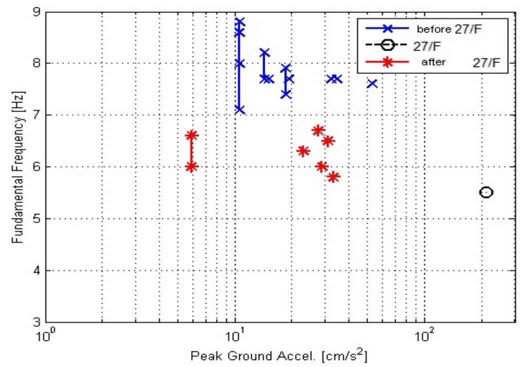
Using ambient vibrations, Astroza et al. [29] determined that fundamental periods of undamaged confined masonry buildings varies between 0.095 and 0.177 sec and that Eq. 1, an empirical formula that depends on the total weight of the building (P in [ton]) and on the wall area in one direction (A in [m^2]) can be used to predict the period of the first two modes in both horizontal directions:

$$T = 0.0147 \sqrt{(P/A)} \qquad (1)$$

A 4-story social apartment building, located in Santiago, built with RC walls in the first floor and confined masonry walls in the upper floors, was instrumented with accelerometers in 1992. During the 2010 Maule earthquake, some cracks in a wall of the second floor oriented in the longitudinal direction of the building were apparent and peak acceleration larger than 1.0g was recorded at the fourth floor in the same direction. A linear finite element model of the building was developed and subjected to free field records obtained at the site, in order to determine the seismic demand on the walls that could explain the observed cracks. As validation criteria for the model, it was required to reproduce the identified fundamental frequencies and the lateral displacements at the fourth floor. The seismic demand on the walls estimated from the FE model, was much larger than the strength prescribed by the design codes. The predominant frequency values varied before and after the earthquake. The lowest frequency value in the transverse direction, was obtained during the Maule earthquake, see Fig. 9. These observations indicate the occurrence of lateral stiffness degradation, which was partially recovered although no retrofit strategy was used [30].



Longitudinal direction



Transverse direction

Fig. 8 – Predominant frequencies before and after 27/F earthquake

6. Lessons Learnt From Great Chilean Earthquakes.

In summary, to limit damage caused by earthquakes in confined masonry buildings the following actions must be considered:

- Low resistance of the masonry and its variability makes necessary to control the wall density and quality of materials (unit and mortar). Based on the Chilean seismic experience, a wall density per unit of floor plan greater than 0.85 % on each direction is recommended [31, 1].

- The confining elements must be located close enough to avoid out-of-plane damage, especially when wall thickness is less than or equal to 150 mm. Excessive spacing between tie-columns or lack of tie-beams may cause out-of-plane damage.

- Lack of tie-columns around window opening decreases the shear strength [32] and the post-shear cracking displacement capacity. When the area of an opening is less than 5% of the masonry panel area and it is not located near to one end of the masonry panel, its effect can be ignored.

- It is recommended to include closer stirrups at both ends of tie-columns and a minimum cross sectional area of tie-column to avoid diagonal crack propagation that may appear at the masonry panel. These design details may reduce the level of damage and the strength or stiffness degradation of the wall, avoiding residual deformations difficult to repair.

- The detailing of reinforcement bars is essential in the zones where confinement elements concur and has been the cause of bad behavior observed in many masonry houses since 1958 [33].

- The vertical reinforcement bars placed inside vertical holes located at the ends of the masonry panels, in replacement of external concrete tie-column, has been ineffective. This type of reinforcement should not be used in masonry buildings with three or more stories.

- The lack of RC beams on the top of walls has caused severe damage, therefore it is recommended to use any solution that ensures behaviour like the observed with a concrete tie-beam.
- The misbehaviour of masonry walls built with hollow concrete blocks during all the Chilean earthquakes, since 1965, suggests that its use should be avoided [34]. These walls suffer crushing after diagonal cracking takes place, causing significant post-cracking strength and stiffness degradation.
- The construction phase is very important and must be inspected. In this way, the concrete of tie-columns must be poured against a toothed vertical side of the masonry panel, to integrate the slender tie-columns with the masonry panel and ensure an appropriate joint between them. The placement of concrete must be made very carefully due to small dimensions of confining elements. The construction sequence of confined masonry walls helps the inspection of reinforcement placement at different construction stages, since the use of steel bars and concrete is limited only to confining elements. This is an advantage compared to other building technologies, e.g. reinforced masonry.

7. Conclusions

In the last 100 years, more than ten events with magnitude around 8.0 or larger have occurred in Chile. Three of them have taken place in the last six years. This scenario has provided an excellent opportunity to analyze the performance of different structural typologies. This paper focused on masonry buildings, most of them built in the northern (from XV to IV regions) and central regions (V to VII, including MR) of Chile, with different design solutions (i.e. unreinforced, partially confined or partially reinforced, and reinforced or confined masonry).

The observed seismic behaviour confirms that confined masonry buildings show a good performance if the recommendations of the Chilean Code NCh2123 are satisfied, in spite of the use of unqualified workmanship, as occur in developing country. A wall density per unit floor plan equal to or larger than 0.85% on each direction of the building is recommended.

Key factors that contributed to the presence of severe damages are the local site conditions together, a low wall density in one or both horizontal directions, and deficiencies in the size, reinforcement detailing, and construction of RC confining elements. In recent years, it has become evident that the effect of the focal mechanism of the earthquake also plays an important role.

Experimental research to study the seismic behavior of confined masonry walls subjected to in-plane lateral load, using different masonry units, boundary conditions, aspect ratios, opening sizes, lateral displacement patterns, and number of tie-columns, have been carried out in the last four decades. The experimental data allow to assess various parameters required to design this type of masonry typology. Nonetheless, only a few masonry buildings are instrumented with a local accelerometer network in the world. This situation represents a serious limitation to better understand the real seismic response of these buildings. In addition, research related to repair and retrofit strategies for confined masonry walls is also scarce.

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