Performance and Seismic Vulnerability of Masonry Housing Types Used in Chile

M. O. Moroni¹; M. Astroza²; and C. Acevedo³

Abstract: Masonry is the most used construction material, especially for residential dwellings built in all regions of Chile, up to four stories high. The masonry wall’s reinforcement of these buildings can be classified into three types: confined, reinforced and hybrid. Although buildings with confined masonry walls have limited shear strength and ductility, they have demonstrated acceptable seismic behavior. Experience for buildings with the other two types of reinforcement has been different; during the 1985 Llolleo earthquake several buildings had severe damage due to design and construction deficiencies. In this paper a description of key structural features, construction process, seismic resilient features and deficiencies of masonry housing types used in government-subsidized low-income dwellings is provided. In addition, various seismic vulnerability indices that characterize their seismic behavior are calculated. The evolution over time of such indices shows that nowadays a large amount of buildings are likely to be damaged after a severe earthquake.

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CE Database subject headings: Masonry; Shear walls; Design standards; Seismic hazard; Chile; Construction materials.

Introduction

Chile is a country of 15,000,000 inhabitants and annual income of US $5,000 per capita that is shaken frequently by severe earthquakes that represent big challenges to its earthquake engineering community. Shear walls have been the structural system most used for residential buildings, in combination with high wall density ratios, in order to assure appropriate seismic behavior. With regard to construction materials, Fig. 1 shows the evolution of different types of materials used in the last two decades. Reinforced concrete and masonry with different reinforcement are the most used materials. Houses made of timber can be found mostly in the south of Chile.

The combination of masonry walls with different types of reinforcement gives rise to three types of buildings: confined masonry buildings, reinforced masonry buildings and hybrid masonry buildings. Fig. 2 shows the distribution of these typologies versus the number of stories, obtained from a database that includes 284 low-income housing prototypes designed since the mid-1950s (Gómez 2001). Each of these prototypes acts as a model for hundreds of dwellings actually built. Confined masonry buildings represent 16.3%, reinforced masonry buildings represent 25.7% and hybrid masonry buildings represent 50.1%. Reinforced concrete buildings or houses of timber represent a smaller percentage.

The database includes architectonic characteristics of the buildings, some general information such as height, number of floors, number of units, unit and floor area, total weight, total cost and some key structural features like wall dimensions, quality of material, amount of reinforcement and foundation characteristics. The information was collected mostly from Ministry of Housing files, with some from municipality archives and some from structural design firms. In this paper, the characteristics of the masonry buildings are presented, some vulnerability indices that characterize their seismic behavior are calculated and the evolution over time of those indices is also addressed.

Characteristics of Housing Types

Independent of the type, these low-income buildings are regular with regard to both plan and elevation and are configured mainly with shear walls tied together at floor levels by reinforced concrete beams. At least two lines of walls are present along each principal direction; along the longitudinal direction they are located at the perimeter whereas along the transverse direction there is also a median wall. The wall thickness varies between 140 and 200 mm depending on the size of the bricks and whether there is or is not stucco cover. The brick dimensions and compressive strength, $f_{ck}$, are presented in Table 1. The height-to-thickness ratio of the shear walls at each floor level varies between 12 and 17. Usually the concrete strip footing does not have reinforcement, unless the soil is clay or silt. When any dimension in the building plan is longer than 20 m, reinforced concrete walls at least 1-m long must be located at each end in order to avoid cracking due to shrinkage of the floor system. These floors generally consist of cast-in-place reinforced slabs with a thickness between 100 and 120 mm. The average house unit area is 40 m².

Confined Masonry Buildings

Confined masonry buildings consist of load-bearing unreinforced masonry walls commonly made of clay units or concrete blocks,
confined by cast-in-place reinforced-concrete vertical tie columns, as is shown in Fig. 3. These tie columns are located at regular intervals and connected together with reinforced-concrete horizontal tie beams that are cast after construction of the masonry walls. Tie columns and tie beams prevent damage due to out-of-plane bending effects and improve wall ductility. Tie columns have a rectangular section whose dimensions typically correspond to the wall thickness \( \approx 150–200 \) mm and depth of 200 mm. Both tie columns and tie beams must have at least four 10-mm diameter longitudinal reinforcements. Stirrups of 6 mm diameter must be spaced 100 mm at the extremes and 200 mm at the center of the elements. The allowable stress method is used for design according to NCh2123.OF97. The typical masonry shear strength is 0.5–1 MPa.

The use of this type of construction started during the 1940s, after the 1939 Chillán earthquake, for dwellings and apartment buildings of up to four floors. Confined masonry walls have limited shear strength and ductility compared to reinforced concrete walls. Nevertheless, typical housing buildings have exhibited good earthquake resistance, as is documented later in this paper.

**Reinforced Masonry Buildings**

Masonry walls reinforced with steel bars placed in the vertical cores of specially shaped ceramic masonry units or concrete blocks represent the basic form of this type of housing. Horizontal bars are placed in horizontal mortar joints. Although the cores and voids containing reinforcement should be filled with grout, this is not always done. Most of the time, cores and voids are only filled with horizontal joint mortar. In addition, the size of the hollow in the ceramic unit is quite small so it is difficult to fill. Concrete blocks, mostly used in the north of Chile, have larger hollow cells, but they have water leakage problems so since 1997 their use has been forbidden in central Chiloean Metropolitan Region.

The use of this type of construction started during the 1970s, with it being widely used for dwellings and apartment buildings of up to four floors. Until 1986 there was no seismic design code for this structural type, so its behavior during the 1985 Llolleo earthquake was quite bad. After that, the NCh1928 code was developed based on UBC-1979 and the experience gained by looking at the seismic behavior of those structures in previous earthquakes. By 1993, the last version of NCh1928.OF93 was published (INN 1993), and rigorous requirements were specified. Since then, the use of this type of construction has been less frequent, in part due to economic reasons.

**Hybrid Masonry Buildings**

Masonry walls with a variety of reinforcement types represent the basic form of this housing type. Some reinforced concrete walls may be used in the lower floors in buildings three or four stories high. Generally, along the longitudinal direction there are reinforced masonry walls or partially confined walls with a reinforced concrete column at one end and a tensile bar at the other end, as shown in Fig. 4. This latter solution is frequently used when there are openings. The reinforced walls typically have a 10-mm diameter bar at each end plus 8-mm diameter bars distributed along the wall. Although the maximum allowed spacing between bars is 84 cm, it is frequently between 120 and 150 cm. The diameter of the extreme bars is less than what is required by the NCh1928.OF93 code for dwellings higher than two stories (minimum diameter must be 12 mm). Horizontal reinforcement does not accomplish a minimum steel ratio of 0.06%. Along the transverse direction, walls may be partially reinforced or confined with reinforced concrete columns. Typically the wall thickness is 14 cm in the facade and 15 cm in the interior walls.
Regular use of this type of construction started during the 1980s, and at present it is the most common type for dwellings and apartment buildings of up to four floors. A seismic design code for this type of buildings does not exist, but the Ministry of Housing has some specifications for one- or two-floor dwellings. Designers of this type of building feel that some requirements of NCh1928.of93 or NCh 2123.of97 are too strict, so they just follow the Ministry of Housing requirements even for higher buildings.

Seismic Performance

Masonry buildings with some type of reinforcement have been built since 1930. This was a consequence of the publication of the first Chilean seismic code that enforced the use of reinforcement in masonry walls, considering the bad behavior observed during the 1928 Talca earthquake, magnitude $M = 8.0$, in unreinforced masonry buildings. After that, several earthquakes with magnitude larger than 7.0 have hit masonry buildings, thus allowing compilation of valuable information about their seismic performance.

The first earthquake in this series occurred in 1939, magnitude $M = 7.8$. In Chillán where the Modified Mercalli Intensity (MMI) = 1X, about 3,500 dwellings were inspected, of which only 4.5% of them were of confined masonry type. Sixteen percent of the confined masonry houses inspected, 65% of adobe houses and 57% of unreinforced masonry houses collapsed or partially collapsed leaving, a death toll of 30,000. On the other hand, 53% of the confined masonry houses inspected, 80% of reinforced concrete buildings and 4% of houses of timber were undamaged (Del Canto et al. 1940).

Later, during the 1965 La Ligua earthquake, magnitude $M = 7.1$ and maximum MMI = VIII–IX, about 21,000 adobe and masonry houses collapsed and 71,000 had to be repaired. This was unexpected behavior for masonry houses although some of them did not have vertical tie columns. The results were especially bad in those houses made of hollow concrete blocks, which were very rigid and too sensitive to the high frequency content of the earthquake. The typical pattern of damage included in-plane shear failure, out-of-plane bending failure, bond failure between masonry and concrete elements and damage in column–beam joints and in connections of perpendicular walls (Monge 1985).

Due to the 1971 Papudo earthquake, magnitude $M = 7.5$ and maximum MMI = IX, about 1,000 one-story hybrid masonry houses in Choapa Valley partially collapsed. The masonry walls had tie-reinforced concrete beams and some tensile bars at the extremes of the walls and/or at the corners of joints between walls (Monge 1985).

The 1985 Llolleo earthquake, magnitude $M = 7.8$, that hit the central part of Chile, the most populated region with 6,000,000 inhabitants, left 66,000 collapsed houses and 127,000 damaged units, mostly adobe houses. The total number of existing units in that area at that time was 774,530, and about 170,000 of them were low-income housing. None of the latter collapsed although some reinforced masonry buildings had to be demolished after the earthquake. The Ministry of Housing appointed a special committee to review the seismic effects on 140 government-subsidized low-income projects, about 50,000 units; this committee was also in charge of retrofitting or strengthening the damaged buildings (Flores 1993).

### Table 2. Damage Categories

<table>
<thead>
<tr>
<th>Level of damage</th>
<th>Extent of damage</th>
<th>Action to take</th>
</tr>
</thead>
<tbody>
<tr>
<td>Light nonstructural damage</td>
<td>Fine cracks on plaster, falling of plaster on limited zones</td>
<td>It is not necessary to evacuate the building; only architectural repairs are needed</td>
</tr>
<tr>
<td>Moderate structural damage</td>
<td>Small cracks on masonry walls, falling of plaster block in extended zones. Damage in nonstructural members, such as chimneys, tanks, pediment, and cornices. Structure resistance capacity has not been reduced noticeably. Generalized failure in nonstructural elements</td>
<td>It is not necessary to evacuate the building; only architectural repairs are needed in order to ensure preservation</td>
</tr>
<tr>
<td>Severe structural damage</td>
<td>Large deep cracks in masonry walls, widespread cracking in reinforced concrete walls, columns and buttress. Inclination or falling of chimneys, tanks, and stair platforms. The structure resistance capacity is partially reduced</td>
<td>The building must be evacuated and raised. It can be reoccupied after retrofitting. Before architectural treatment is undertaken structural restoration is needed</td>
</tr>
<tr>
<td>Heavy structural damage</td>
<td>Wall pieces fall down, interior and exterior walls break and lean out of plumb. Failure in elements that connect buildings portions. Approximately 40% of essential structural elements fail. The building is in a dangerous condition</td>
<td>The building must be evacuated and raised. It must be demolished or major retrofitting work is needed before it can be reoccupied</td>
</tr>
</tbody>
</table>
Table 3. Percentage of Masonry Dwellings with Severe or Heavy Structural Damage, 1985 Llolleo Earthquake

<table>
<thead>
<tr>
<th>Number of stories</th>
<th>Confined masonry</th>
<th>Reinforced masonry</th>
<th>Hybrid masonry</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.00</td>
<td>0.000</td>
<td>—</td>
</tr>
<tr>
<td>2</td>
<td>0.02</td>
<td>0.25</td>
<td>—</td>
</tr>
<tr>
<td>3–5</td>
<td>0.22</td>
<td>0.63</td>
<td>0.36</td>
</tr>
</tbody>
</table>

Escobar (1986) correlated the damage in low-income housing with the soil conditions. With that in mind, he inspected about 84,000 units made of different materials, typology and number of floors, mostly located in Santiago. A level of damage, defined in Table 2, was assigned to each unit, concluding that about 50% of the total had some type of damage. Confined masonry buildings of one to four stories represented 16% of the inspected dwellings, reinforced masonry buildings one to four stories represented 14.5%, hybrid masonry buildings three to five stories represented 15.2%, one-story unreinforced masonry houses 30% and three- to five-story reinforced concrete buildings 24.2%. Table 3 shows the percentage of units with severe or heavy structural damage with respect to each class for one, two or three to five stories. The lack of tie columns at one end of the walls or at the opening extremes explains most of the damage in confined masonry walls. In reinforced masonry buildings damage was due to construction problems, like incomplete grouting, since only the hollows with reinforcement were filled, bad quality mortar and lack of horizontal reinforcement.

More recently, in the 1997 Punitaqui earthquake, magnitude 6.8, damage occurred in one-story hybrid masonry houses located in the epicentral zone, where the MMI intensity was VII–VIII. Also two-story hybrid masonry houses located in Ovalle (MMI=VII, hipocenter distance=83 km) and one-story hybrid houses located in Illapel (MMI=VI–VII, 100 km from the epicenter) had moderate damage. The latter was due to differences in settlement on a sloped terrain. Fig. 5(a) shows a damaged house at the epicentral zone and Fig. 5(b) shows a damaged house located in Ovalle Region.

Based on previous experience, the following deficiencies have been identified in confined masonry walls: (1) limited shear strength, (2) limited ductility, (3) lack of tie columns at all openings that diminishes the shear strength and displacement capacity postshear cracking, (4) excessive separation between tie columns or lack of tie beams that may cause out-of-plane damage and (5) shear cracks that propagate through the tie columns and reduce the stiffness and resistance capacity. To prevent this last effect NCh2123.OF97 requires that stirrups closer together should be used at column ends.

In reinforced masonry walls it is difficult to achieve good anchoring and bonding conditions especially if poor mortar instead of grout is used or cell sizes in clay units are inappropriate. With respect to hybrid masonry, the shear strength is not known and therefore shear failure is more likely to occur, which makes it difficult to get flexural ductile failure. Besides, the lack of reinforced concrete tie column may cause brittle shear failure and out-of-plane bending effects. The tensile steel bar in one end does not represent a proper tie column.

Strengthening procedures include improving confinement of the masonry wall with reinforced concrete tie columns, coating the damaged wall with shotcrete over wire mesh anchored to the masonry and replacing bricks that have been damaged. The experience acquired during the 1985 earthquake showed that the repair cost for one-story hybrid masonry houses was about 28% of the original cost; for three-story reinforced masonry buildings it was about 22.5% of the original cost and for three-story confined masonry buildings was about 7.8% of the original cost.

Seismic Vulnerability Rating

Different parameters by which to estimate the seismic vulnerability of buildings have been proposed, but few of them have been related to damage after severe earthquakes. Wall density, $d$, calculated as wall area in each direction, divided by floor area, $A_p$, has been the parameter most used to characterize reinforced concrete and masonry buildings. Fig. 6 shows the evolution over time of parameter $d$ calculated at the first floor, for both directions for all masonry buildings included in the database. The average wall

Table 4. Relationship between Level of Damage and Wall Density per Unit Floor after 1985 Llolleo Earthquake

<table>
<thead>
<tr>
<th>Level of damage</th>
<th>$d_a$ (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Light</td>
<td>$&gt; 1.15$</td>
</tr>
<tr>
<td>Moderate</td>
<td>0.85–1.15</td>
</tr>
<tr>
<td>Severe</td>
<td>0.50–0.85</td>
</tr>
<tr>
<td>Heavy</td>
<td>$&lt; 0.50$</td>
</tr>
</tbody>
</table>
The wall density per unit weight per floor, $d_{np}$, is another indicator of expected seismic behavior for these types of buildings. It is calculated as the ratio of the wall cross-sectional area in the first floor to the total weight of the structure. To guarantee that the displacement capacity be greater than the displacement demand, the wall density per unit weight per floor must be about 0.012 m$^2$/ton (Moroni et al. 2000). On the other hand, observed structural performance suggests that wall density per unit weight per floor greater than 0.013 m$^2$/ton will ensure the occurrence of only moderate damage when subjected to earthquakes of magnitude larger than 7 (Astroza et al. 1993). Several buildings do not fulfill this requirement, as can be seen from Fig. 9 where the variation of $d_{np}$ over time is shown.

A more belabored procedure to qualify the seismic vulnerability of reinforced concrete buildings has been established by the Ministry of Construction in Japan that includes three levels of screening (Hirosawa et al. 1993). At each level, two numerical indices are evaluated: $I_s$ (seismic index of structure) and $I_{so}$ (seismic judgement index of structure). A larger value of $I_s$ indicates better seismic performance of the building. The buildings with values of $I_s$ (or larger than $I_{so}$) can be considered safe enough against severe earthquakes. This procedure has also been used in structural vulnerability studies of hospitals with masonry and concrete shear walls (PAHO 2000).

Using our data, the first level of the screening procedure has been applied to a subset of masonry buildings (Acevedo 2002; Aguirre 2002). The $I_s$ index was evaluated at the first story and at each direction of the building using

$$I_s = E_o \times S_d \times T$$

The basic structural subindex, $E_o$, that depends on the ultimate horizontal strength, ductility and type of failure considered was calculated using

$$E_o = \left[ \alpha_1 (C_{ma} + C_{mar}) + \alpha_2 \times C_w \right] F$$

$$C_w = \left( \frac{f_c^2}{200} \times \left( \frac{12A_{m3} + 10A_{m4}}{W} \right) \right)$$

**Table 5. Strength Reduction Factor**

<table>
<thead>
<tr>
<th>Characteristic</th>
<th>$\alpha_1$</th>
<th>$\alpha_2$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Infilled masonry walls or partially confined or partially reinforced walls control failure</td>
<td>1.0</td>
<td>0.7</td>
</tr>
<tr>
<td>Reinforced concrete walls control failure</td>
<td>0.0</td>
<td>1.0</td>
</tr>
</tbody>
</table>
The $C_w$ term in Eq. (4) is related to the ultimate shear strength of concrete walls. The multipliers for $A_m^3$ and $A_m^4$, reinforced concrete wall areas, reflect the fact that more slender walls ($A_m^4$) have less shear resistance. $C_m$ and $C_m^a$ are related to the ultimate shear strength of confined or partially reinforced masonry walls, respectively. $C_m^a$ is calculated as in Eq. (5), but with $A_m$ replaced by $A_m^a$. The three terms are normalized with respect to the total weight of the building, $W$. Values for $\alpha_1$ and $\alpha_2$ depend on the type of wall that controls failure and are given in Table 5. The factor $F$ is taken as 0.8 when the partially reinforced wall area, $\sum A_m^a$, represents more than 25% of the total masonry walls, $F = 1$ for all other cases.

The structural design subindex, $S_d$, reflects the grade of irregularity of the building shape and the distribution of stiffness. $S_d = 1.0$ unless clearances of the expansion joint are less than the minimum values prescribed by NCh433.Of96 (INN 1996), in which case $S_d = 0.9$. The time subindex, $T$, reflects the deterioration of the building strength and ductility. From field inspection, $T$ was given a value of 1 for all buildings.

The seismic judgement index, $I_{so}$, represents earthquake demands and it depends on the local site conditions and the admissible level of damage. The seismic judgement index given in Table 6 represents the seismic demand prescribed by NCh433.Of96 for seismic zone 2 and soil type II for different values of the modification factor, $R$.

Fig. 10 shows the variation of seismic index $I_s$ for buildings up to four stories. Seismic judgement index $I_{so}$ is shown for different $R$ values. When $I_s > I_{so}$ calculated with $R = 1$ the buildings should have elastic behavior, but when $I_s < I_{so}$ calculated with $R = 2$ or 3 the buildings are likely to be damaged when exposed to the design earthquake. In particular, severe damage may occur when $I_s$ is less than 0.27 ($R = 2$) and heavy damage may occur when $I_s$ is less than 0.18 ($R = 3$). The fall in $I_s$ values in the last 10 years, especially in buildings three or four stories high has been noticeable.

**Table 6. Seismic Judgement Index, Seismic Zone 2, Soil Type II**

<table>
<thead>
<tr>
<th>$R$</th>
<th>$I_{so}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.35</td>
</tr>
<tr>
<td>2</td>
<td>0.27</td>
</tr>
<tr>
<td>3</td>
<td>0.18</td>
</tr>
</tbody>
</table>

$C_m^a = \left( \frac{0.6(0.45 \times \tau_o + 0.25 \times \sigma_o) \sum A_m}{W} \right)$

Fig. 11. Seismic index and wall density per unit floor for three- or four-story buildings

In Fig. 11, three- and four-story high buildings have been sorted from more to less vulnerable according to $d_a$ values, calculated in the weakest plan direction. In this case the wall area includes the masonry wall area as well as reinforced concrete shear wall area $A_{ci}$, if present. However, the latter has been converted to an equivalent masonry area, $A_{ma}$, by multiplying area $A_{ci}$ by factor $F_2$ given by an equation which considers both materials allowable stress. As suggested in NCh2123, the masonry shear strength, $\tau_o$, was considered to be 0.5 MPa while $\sigma_o$ corresponds to compressive stress due to dead and live loads. Compressive concrete strength, $f'_c$, varies between 16 to 22 MPa.

$$F_2 = \frac{0.29 \sqrt{f'_c}}{0.23 \tau_o + 0.12 \sigma_o}$$

Also in Fig. 11, the $I_s$ values for the same buildings and direction are shown. Although $I_s$ values have great dispersion, both parameters show a similar tendency. On the other hand, the wall density per unit floor does not take into account the ductility capacity of the buildings. In fact buildings with the same $d_a$ may be well confined or partially reinforced. In that sense, the ordering resulting from $I_s$ values may lead to a more reliable vulnerability diagnostic.

**Conclusions**

The main characteristics of low-income masonry buildings in Chile were described and different vulnerability indices were calculated. Evidence of damage shows that confined masonry buildings have appropriate seismic behavior. The situation for hybrid masonry buildings is different, and is a cause of great concern considering the large number of this type of building that is being constructed currently with designs that do not follow any code, and with seismic behavior that has proven to be unsatisfactory in previous earthquakes. The fact that so many buildings have $d_a$ less than 0.85% or $I_s$ less than 0.27 is another cause for concern.

**Acknowledgments**

The writers wish to acknowledge the University of Chile for financial support and the Ministry of Housing, Municipality of Pudahuel, and Consultant Engineering Offices that collaborated by providing information.
The following symbols are used in this paper:

- \( A_{w1} \) = reinforced concrete wall area;
- \( A_{ma} \) = masonry wall area;
- \( A_{m3} \) = sum of cross-sectional area of reinforced concrete walls without peripheral columns and wall slenderness \((H/L)\) less than or equal to 2;
- \( A_{m4} \) = sum of cross-sectional area of reinforced concrete walls without peripheral columns and wall slenderness greater than 2;
- \( A_p \) = floor area;
- \( d \) = wall density;
- \( d_n \) = wall density per unit floor;
- \( d_{np} \) = wall density per unit floor and unit weight;
- \( f'c \) = compressive strength of concrete;
- \( f'm \) = compressive strength of masonry;
- \( H_i \) = wall height;
- \( L_i \) = wall length;
- \( n \) = number of floors;
- \( R \) = modification response factor;
- \( W \) = total weight of building;
- \( \sigma_o \) = compressive stress due to dead and live loads;
- \( \Sigma A_{ma} \) = sum of cross-sectional area of confined or reinforced masonry walls;
- \( \Sigma A_{mar} \) = sum of cross-sectional area of partially confined or partially reinforced masonry walls; and
- \( \tau_o \) = masonry shear strength.

References


