DEVELOPMENT OF CONFINED MASONRY SEISMIC CONSIDERATIONS, RESEARCH AND DESIGN CODES IN PERU

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Abstract

The paper presents the historic evolution of confined masonry in Peru, including the Code development, and the considerations for seismic design of confined masonry buildings. A brief summary of the experimental research done over the last 35 years is given, and the development of the Peruvian masonry design Code is reviewed. Confined masonry was used empirically since the early 1950s in Peru, but it started to spread after the 1970 earthquake (M7.8), in which unreinforced masonry constructions suffered significant damage. Also, the importance of the experimental research performed at the Peruvian universities is highlighted. The contribution of several experimental projects to the Masonry Code specifications is explained. The Code specifications regarding seismic design have been revised, since 1970 until the current 2006 Masonry Code (Norma E.070 in Spanish). This is the only Peruvian construction and design Code which covers aspects of performance-based seismic design. Finally, the challenges related to the proper application of the Masonry Code are discussed (deficient materials, construction characteristics, lack of construction quality control etc).

Keywords: Confined masonry; masonry design code; Peru; experimental research; seismic design

1. Introduction

This paper presents an overview of design and construction practice of confined masonry in Peru. Confined masonry wall consists of a masonry wall surrounded on its four sides by confining vertical and horizontal elements (typically made of reinforced concrete, RC). At the first level in the building, a concrete foundation is considered as the horizontal confining element. Concrete in the vertical elements (RC tie-columns) should be poured after the masonry panel is constructed.

The first use of such wall construction system in Peru may have been in 1940s, and it became more popular in the 1950s and 1960s [1]. The first formal Peruvian construction Code was released in 1970 [2], which contained empirical specifications regarding masonry constructions. Poor performance of unreinforced masonry constructions in the M7.8, May 31 1970 earthquake, with epicenter about 400 km north of Lima, revealed the importance of providing reinforcement for preventing failure of unreinforced masonry walls. Since then, seismic considerations became an important issue in the Peruvian regulations. The Basic Seismic Code was issued in 1977 and it contained provisions related to reinforced concrete, masonry, steel, and wood construction [3]. The first separate Masonry code document was issued in 1982 [4] called "Norma E.070-Albañilería" (in Spanish), with allowable stress design for masonry walls. The current Peruvian Masonry Code, issued in 2006, includes performance-based design considerations for seismic design of masonry walls [5]. It is the only Peruvian construction and design Code with such features. Finally, the challenges for proper application of the Code are discussed.

2. The early Peruvian masonry regulations of 1970 and 1977

The 1970 Peruvian code specifications for masonry structures were empirical [1]. The bearing walls needed a minimum thickness of 250 mm, without cover. The confining RC tie-columns were set to a maximum distance of 5 m, but cross-sectional dimensions and reinforcement requirements were not specified. The exterior walls and bearing walls had to have an RC tie-beam. The mortar compressive strength had to be larger than 6 MPa. The maximum height of buildings with bearing masonry walls was limited to four storeys, or up to 15 m overall height. The maximum height of rural constructions and houses was limited to two storeys or 6 m overall height.

In the 1970 Code, base shear seismic force was called H, depending on the importance factor U, a seismic coefficient C_1, and the building weight P (H = U C_1 P). For common building functions such as office, apartments or residential houses, factor U was set to 1.0 for the seismic region 1 (southern Coast of Peru); 0.8 for seismic region 2 (most of the Coast and highlands); and 0.6 for seismic region 3 (the low Amazon jungle). Seismic coefficient C_1 depended on the number of storeys in the building (0.16 for 1 or 2 stories; 0.14 for 3-storey building; and 0.12 for 4-story building). The base shear seismic force H had to be resisted by the RC tie-columns and masonry walls parallel to the direction of seismic analysis. However, masonry resistance was not defined, and a procedure for the stress calculations in the confined masonry system was missing.

In the 1977 Peruvian seismic code, Chapter 3 was devoted to masonry construction [3]. Allowable stress design was introduced to design brick masonry walls subjected to compression, shear and bending moments. The bearing walls could be constructed using either solid or hollow units. A solid unit should have a minimum 75% of the gross section in the bed area. The mortar could have lime content or not. The allowable stresses in masonry walls were as follows: compression by flexure (f_c) in walls and columns of 0.30 f'_m; tension by flexure (f_t) perpendicular to the layers of 1.0 kg/cm^2 (0.098 MPa), and parallel to the layers of 2.0 kg/cm^2 (0.196 MPa); shear for unconfined masonry of 0.15 $\sqrt{f'_m}$ but not greater than 1.1 kg/cm^2 (0.108 MPa); shear for confined masonry of 0.30 $\sqrt{f'_m}$ but not greater than 2.2 kg/cm^2 (0.216 MPa). The allowable axial compression stress (f_m) could be determined from Eq. (1) until the current 2006 Code.

$$f_m = 0.2 f'_m \left[1 - \left(\frac{h}{35t} \right)^2 \right] \qquad (1)$$

The confining RC tie-column and tie-beam sections were governed by shear, with a width equal or larger to the wall thickness. The reinforcing steel in RC tie-beams was obtained based on tensile stresses, by dividing the wall shear force V by one-half the yield stress and for RC tie-columns, by multiplying this value by 1.33. In the case of several wall panels, the total shear force was divided among the masonry panels to obtain the design force for the confining tie-columns and tie-beams. The minimum longitudinal reinforcement was set to 4-9.5 mm (3/8") diameter bars, and 6 mm (1/4") diameter ties were required at 300 mm spacing.

3. The first Laboratory tests and the Peruvian masonry code of 1982

Experimental research on masonry started in Peru in 1979 with the inauguration of the Structures Laboratory of the Catholic University of Peru (PUCP). Initially, small masonry specimens were used and the first tests on masonry specimens in this laboratory were conducted by PUCP professors Angel San Bartolomé and Mónica Svojsik [6, 7]. Laboratory tests included different types of clay bricks (hand-made, semi-industrial, and industrial) which were tested under axial compression (Fig. 1 left). Also, the effect of vertical load on the behavior of masonry walls was studied through tests on wallets under diagonal compression and axial compression (Fig. 1 centre), and also full-scale walls (Fig. 1 right). Clay brick walls and sand-lime brick walls were studied and a correlation was sought between the results of tests on full-size walls and small specimens.

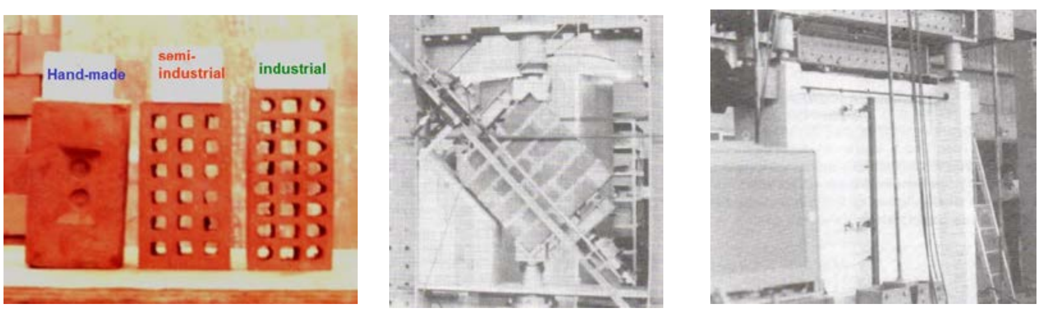

Fig. 1 – Early masonry tests in Perú by San Bartolomé [6].

PUCP professor and private researcher Hector Gallegos [8] also contributed with tests on masonry octagonal walls made of clay and silica lime units. The octagonal walls had different mortars with loads applied in three different directions with respect to the bed joints: 0°, 45° and 90° (Figure 2). Through these tests, the anisotropic nature of masonry was studied and the quality of the bond achieved by the mortar with lime was emphasized.

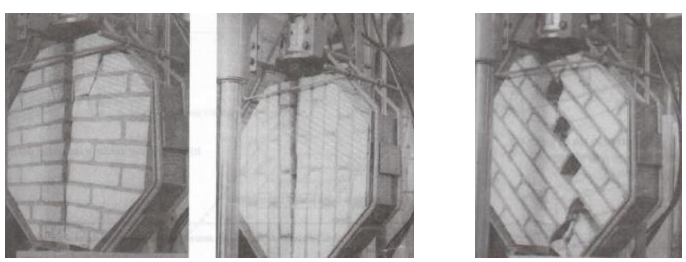

Fig. 2 – Octagonal masonry specimens tested by Gallegos [8].

The 1982 Peruvian Masonry code [4] ("Norma E.070-Albañilería" in Spanish) was prepared using the abovementioned test results. It was the first independent masonry code in Peru, and it followed the allowable stress design approach. The results of tests on small masonry systems, in which units were laid using mortar made of cement, lime and sand, led to the idea that such mortar mix increased the shear resistance of either confined or reinforced masonry. The 1982 masonry Code specified that the allowable masonry shear stress (V_m)

was based on a Coulomb expression, according to which masonry in lime mortar had 50% higher resistance. In the case of flexural tensile resistance (Ft), the masonry in lime mortar had 33% higher resistance.

Regarding the confined masonry, the 1982 Code specified that every wall carrying more than 10% of the total seismic force should be confined. The distance between the RC confining tie-columns should be less than twice the distance between horizontal tie-beams (or the foundation and a tie-beam above). Confined masonry walls were to be designed by following provisions for design of unreinforced masonry by allowable stress design approach. The confining RC elements had to be designed by following the shear friction concept. The minimum area of the cross-section, the area of longitudinal bars in tie-columns and tie-beams, depended on the wall shear force V. Also, the specifications were given for spacing of ties in tie-columns and tie-beams with regards to the length of the confined zone, and outside that zone, the ties could be provided at larger spacing (they were required for construction purpose only).

4. The Laboratory tests between 1982 and 2000

Many experimental research studies and laboratory tests were performed after the 1982 Masonry Code was issued, and a Code revision was proposed with a Technical Committee in 2000. Professor San Bartolomé continued to study different aspects of both confined and reinforced masonry construction. Regarding the confined masonry walls, important research studies were: 1) the effect of the horizontal reinforcement within the mortar bed joint in full-scale walls subjected to lateral cyclic loads; 2) the effect of vertical load and horizontal reinforcement in full-scale walls under lateral cyclic load (Figure 3), and 3) the effect of in-plane slenderness in half-scale walls under lateral cyclic load [6]. In general, the walls experienced shear failure characterized by the diagonal cracking. After gathering the results of these and other theoretical research studies, a new approach for the design of confined masonry walls was presented in 1990, based on the ultimate shear strength of the wall [9].

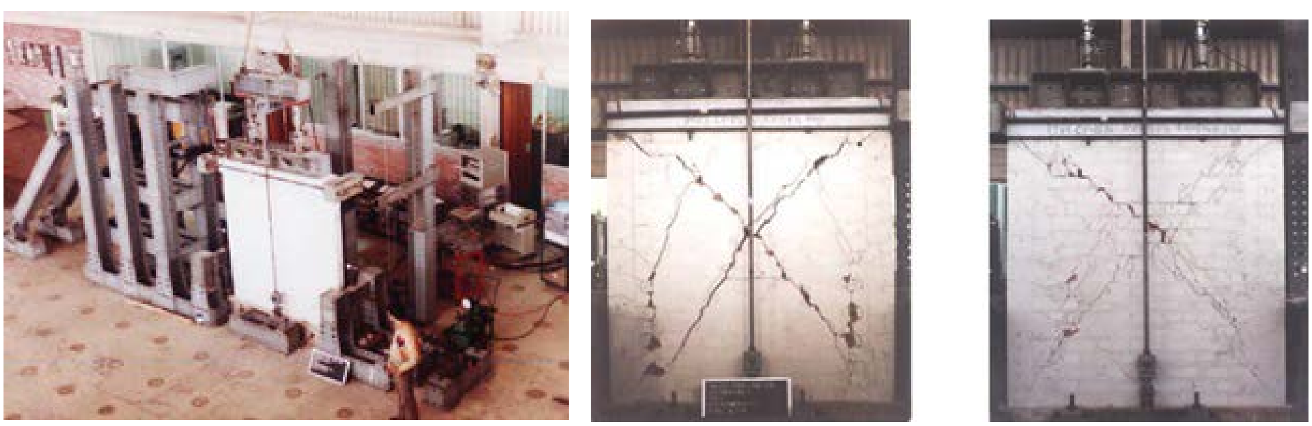

Fig. 3 – Tests on full-scale confined masonry walls with variable vertical load by San Bartolome [6].

The idea of a brittle shear failure instead of a ductile flexural failure for masonry walls generated intense discussions between Peruvian academics. Continuing the experimental evidence, a shaking-table test was performed on a half-scale three-storey confined masonry specimen consisting of two parallel walls [10]. The slender walls started to demonstrate a flexural failure, since tension cracks developed in the RC tie-columns. However, the shear failure subsequently occurred as diagonal cracks appeared in both walls (Figure 4).

Subsequent important research studies included: 1) a comparison between a two-story coupled wall system with shallow beams, another specimen with larger beam size, and a cantilever two-story wall subjected to cyclic lateral loads, 2) the effect of the number of wall panels in a half-scale confined wall (Figure 5); 3) a comparison between a confined masonry wall with toothed connection to the tie-columns and another with vertical connection and wires connecting the tie-columns and masonry wall. The latter research study was planned after the good behavior was observed in masonry constructions with such vertical connections in the 1985 Chile earthquake.

Fig. 4 – Shaking table test on 3-D masonry model [10].

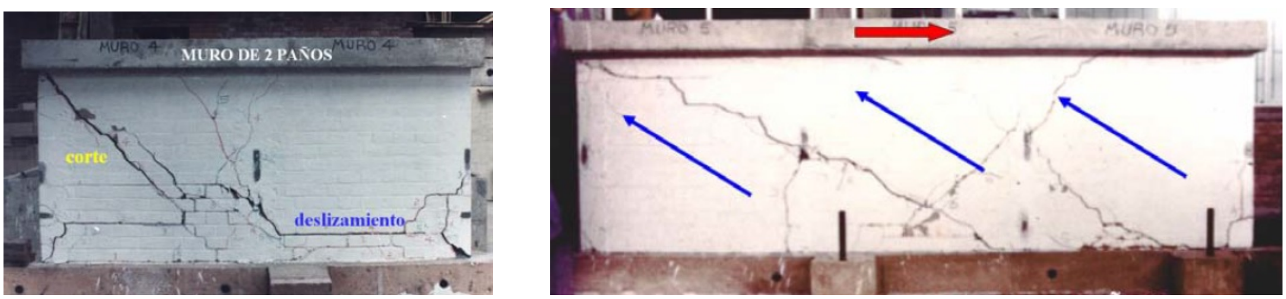

Fig. 5 – Effect of the number of wall panels in a ½ scale masonry model [6].

A joint project between two universities: Catholic University of Peru (PUCP) and National University of Engineering thorough the Japan Peru Center for Seismic Research and Disaster Mitigation (UNI-CISMID), was performed to study the testing procedure for two-story masonry models. A dynamic test was conducted on shaking table in PUCP, while a static monotonic lateral load test was performed in UNI-CISMID. As a result of these two tests, a shear failure was obtained at the first storey level (which could be predicted theoretically), as shown in Fig. 6 (left). The load-displacement envelopes for both models were very similar (Fig. 6 right).

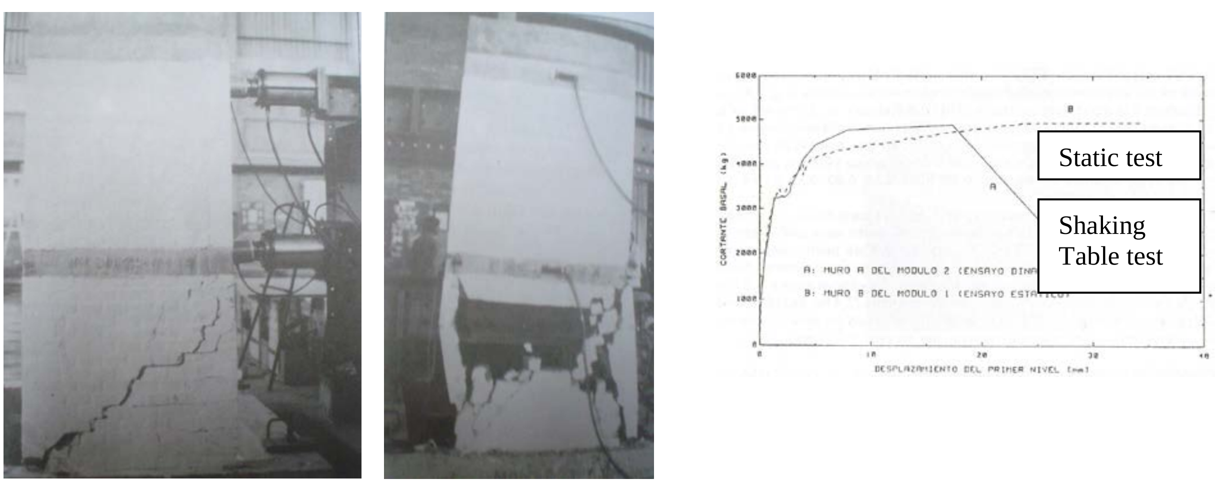

Fig. 6 – Effect of the testing procedures in the seismic wall behavior [6].

All the experimental evidence confirmed the theory that the shear failure was inherent in low-rise confined masonry walls. The way to deal with a brittle shear failure in confined masonry walls consisted in designing the RC tie-columns at wall ends to resist the diagonal cracking force. In a severe earthquake, the first story walls would experience a shear failure, but the collapse would be controlled by the shear capacity of the confining RC tie-columns.

5. The Masonry Code of 2006

5.1 The technical masonry committee

By the end of 1999, a technical committee was called to produce a new masonry Code (to cover both confined and reinforced masonry). The draft document was prepared by Angel San Bartolomé, and the committee chairman was Carlos Casabonne. The committee finalized the masonry Code update in 2004, but the Code became official in 2006, as a part of the National Building Code. This Code entirely replaced the 1982 Masonry Code. The current Peruvian Masonry Code includes the allowable stress design, but also a new approach of performance-based design of structures for moderate and severe earthquakes. A summary of the specifications and the Code criteria were presented at the masonry conferences in 2007 and 2008 [11, 12]. Since then, a new Committee has started the work on preparing a further Code update.

5.2 Review of general updates

With regards to the component materials for masonry walls, solid bricks were defined as those with a mortar-bedded section area of at least 70% of the gross area (the previous Code required 75%). Hollow bricks with vertical holes and hollow bricks with horizontal holes are not allowed for structural walls in high hazard seismic zones (coast and highlands); those bricks may only be used in the Amazon area that has very low seismic hazard. Also, the Code contains an updated procedure for construction and testing of small masonry specimens. It includes a chapter on structural configuration, and another one on minimum structural requirements, as well as the analysis and design of confined and reinforced masonry walls.

The allowable stress design of masonry has been preserved for the following cases: 1) design for axial stress, as it was mentioned when Eq. (1) was discussed; 2) design for bearing stress due to concentrated gravity loads, and 3) out-of-plane loads for structural walls and nonstructural walls under seismic forces.

5.3 Confined masonry wall design

The design procedure for in-plane seismic forces is based on numerous experimental tests carried out at the Structures Laboratory of the PUCP, theoretical analyses, and the observed behavior of buildings during earthquakes in Peru and other countries. The procedure considers a performance-based design for the following two stages: 1) the walls will show elastic behaviour during moderate and frequent earthquakes; and 2) a repairable ductile shear failure will occur for severe earthquakes (without a loss of lateral load capacity).

The structural walls are expected to behave in elastic manner when angular distortions (lateral drifts) are less than 1/800. Diagonal cracking in a masonry wall occurs at this drift level and the corresponding shear force is taken by the RC confining elements, which should be designed for this purpose. Laboratory tests have demonstrated that: 1) damage is economically repairable for lateral drifts smaller than 1/200; and 2) there is no lateral strength reduction when the confining elements are designed to resist the load that causes the diagonal cracking of the wall (V_m). Also, the summation of the confined masonry wall strengths in each direction of the building (ΣV_m) should be at least equal to the base seismic shear load (V). Fig. 7 illustrates the design considerations, where V is seismic design base shear, Z is ground acceleration according to the zone; U is importance factor; S is soil factor; C is building response coefficient; P is building weight; R is reduction factor (equal to 3 for any type of masonry).

It is widely accepted that buildings made of confined masonry walls will exhibit shear failure, particularly at lower stories, when subjected to severe earthquakes, due to the predominant shear deformations over the flexural deformations. This behavior has been observed in real earthquakes as well as in experimental studies, both in single walls subjected to cyclic lateral loads and a 3-storey half-scale building tested on a shaking table [10]. Although in general shear failure is considered brittle, confined masonry may exhibit ductile behavior provided that the confining elements are properly designed and able to resist the shear strength V_m.

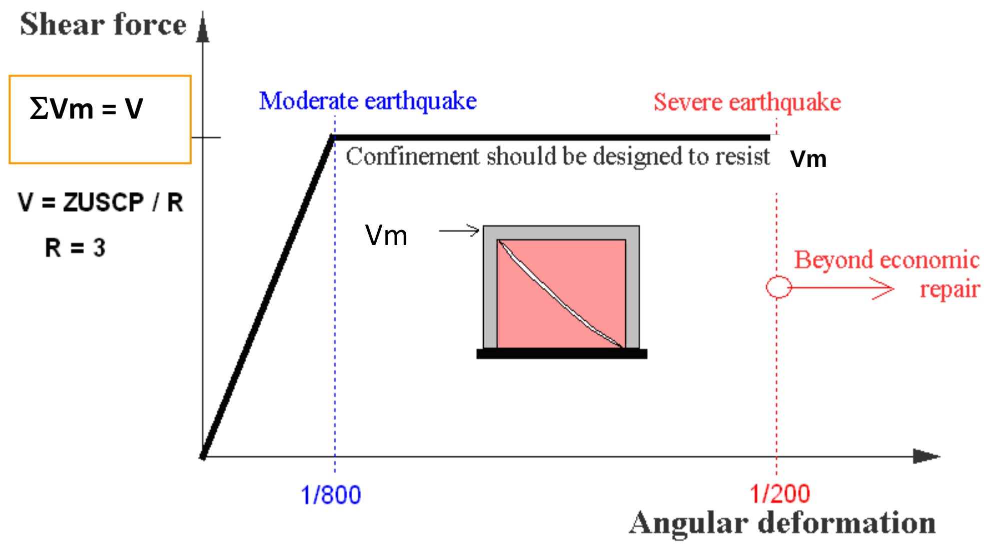

Fig. 7 – Objectives of the design procedure [11, 12].

The design procedure for masonry buildings may be summarized in five steps: 1) verification of the minimum wall density along the building main directions; 2) design for vertical loads; 3) elastic analysis for moderate earthquake loads; 4) verification of the elastic shear force against the shear strength V_m, and 5) design for severe earthquake loads. Steps 1 through 4 are similar for confined and reinforced masonry walls.

In order to avoid a brittle failure due to insufficient lateral strength or excessive ductility demand, a minimum wall density (step 1) should be provided in each of the building main directions as specified in equation (2).

$$\frac{\Sigma\, Lt}{A_p} \geq \frac{ZUSN}{56} \tag{2}$$

where *Z*, *U*, and *S* are defined in the Peruvian Seismic Code, *N* is the number of stories, *L*, the total confined masonry wall length, *t*, wall thickness, and A_p, the typical story area.

Step 2 has the purpose of ensuring an adequate ductility under axial stresses. Experimental studies have shown that large axial stresses can significantly decrease the wall ductility. Therefore, it is specified that the axial stresses do not exceed $0.15f'_m$, where f'_m is the masonry compression strength. If the axial stress exceeds $0.05f'_m$, a minimum horizontal steel ratio of 0.001 is required.

The seismic analysis is performed in step 3. The moderate earthquake is defined in the Peruvian Masonry Code as the one that produces half of the seismic forces relative to a severe earthquake. In each wall and at every storey level, the shear forces obtained from the elastic analysis (V_e) should not exceed 0.55 V_m, to ensure the elastic behavior of a wall in a moderate earthquake.

The code equations for evaluating the shear strength (V_m) for masonry walls were established based on the results of many experimental studies on full-size walls and scaled wall specimens. Equation (3) is applicable to walls with clay and concrete units, and equation (4) applies to walls with silica lime units. For both cases, the aspect ratio α is defined in equation (5).

Clay and concrete units: $$V_m = 0.5\,v'_m\,\alpha\,t\,L + 0.23\,P_g \quad (3)$$

Silica lime units: $$V_m = 0.35\,v'_m\,\alpha\,t\,L + 0.23\,P_g \quad (4)$$

$$1/3 \le \alpha = \frac{V_e\,L}{M_e} \le 1 \quad (5)$$

The variables in equations (3), (4) and (5) are as follows: v'_m is the diagonal shear strength of small square wall specimens; P_g is the wall axial load; V_e and M_e are the shear force and bending moment obtained from the elastic analysis, respectively. Several tests on walls subjected to cyclic lateral loads have shown the effect of the wall height-to-length ($h/L=1/\alpha$) aspect ratio on its shear strength, V_m.

The fifth and last step for the building analysis and design procedure is the design for severe earthquakes. Firstly, the building global strength has to be checked. Considering the already calculated V_m values, the summation of the shear strength at the first story level (ΣV_{m1}) can be determined. This value should be larger than the seismic design shear load V. If the strength is insufficient, some of the masonry walls may be replaced by reinforced concrete walls or the wall thickness may need to be increased. If ΣV_{m1} is larger than R times the base shear V, then the structure will behave elastically and there is no need for further verification - only minimum reinforcement is required for out-of-plane loading.

The in-plane design of confined masonry walls continues with an evaluation of the amplification factors and the verification of the diagonal cracking strength of the walls above the first floor level. It is assumed that during a severe earthquake, each wall at the first floor level will experience diagonal cracking failure when the shear force reaches the strength V_{m1}. In order to obtain the ultimate bending moments and shear forces in the upper floors (M_u and V_u), the calculated elastic internal forces (M_e and V_e) should be amplified by the ratio V_{m1}/V_{e1}, where V_{e1} is the elastic shear force at the first story level. The amplification factor should be calculated for each wall and should be between 2 and 3, the lower value is the ratio of forces from severe to moderate earthquake, and the larger value equals the reduction factor R. When the ultimate shear force in i-th story wall, V_{ui} (i>1), is larger than V_{mi}, the wall at this level will also experience diagonal cracking failure and its confining elements should be designed accordingly. Next, the evaluation of the internal forces in the first floor vertical confining elements has to be done. The internal forces in vertical confining elements (tie-column) may be calculated for simple cases, e.g. single-bay cantilever walls, using equilibrium equations. There is no bending moment, because the tie-column do not experience flexural deformations. For more complex cases, such as several span walls connected through reinforced concrete beams or with transverse walls, the Code includes formulas for ultimate internal shear force (V), tensile force (T) and compression force (C), for interior and exterior tie-columns. The tie-columns at the wall ends are designed to prevent the sliding in the cracked masonry wall. The cross-sectional design needs to be performed using the ultimate internal forces V, T, and C, according to concrete design standards. The tie-column cross section is subjected to a combined shear-friction and tension mechanisms. The concrete core section (inside the ties) and the shear reinforcement are dimensioned to prevent concrete crushing (Fig. 8).

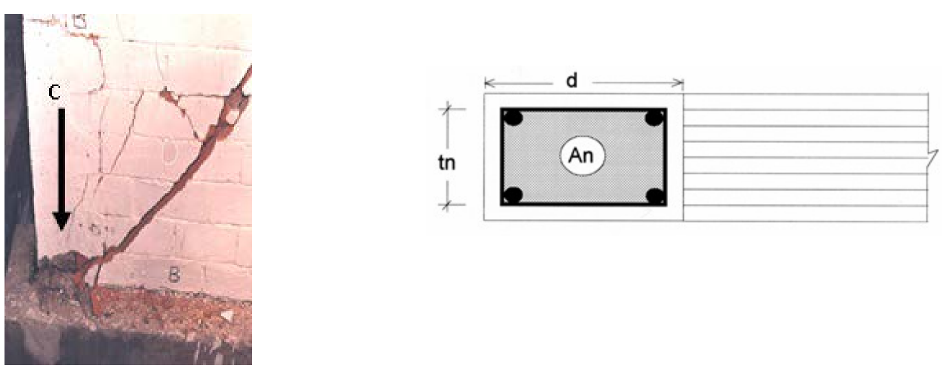

Fig. 8 – Vertical confinement (tie-column) design forces and cross section [12].

The horizontal confining elements (tie-beams) should be able to transfer the seismic loads from the slab to the masonry wall. Only minimum ties have to be provided, because the horizontal confining elements are not subjected to significant shear loads, because the shear area above the cracked first floor is large. Finally, the confining elements at the upper stories (above the first floor) need to be designed. When V_{ui} is smaller than V_{mi}, the masonry wall resists the seismic forces without cracking. In such a case, vertical confining elements should not be designed considering the shear-friction effect. Instead, only the external confining elements are designed for the tension, T, and compression, C, due to the bending moment $M_{ui}= M_{ei}\times V_{m1}/ V_{e1}$. The internal tie-columns do not need to be designed for in-plane actions, because they are integrated into an uncracked masonry wall. However, they should be able to support the wall under out-of-plane seismic actions. The maximum spacing between tie-columns should not be larger than twice the distance between horizontal confining elements (tie-beams). The horizontal confinement (tie-beam) should be designed for tension, produced by the transfer of seismic forces to the walls.

A pseudo dynamic testing was performed in CISMID-UNI on a full scale two-story masonry house (Fig. 9), designed according to the 2006 Code, using low quality bricks but with adequate workmanship [13]. It was found that the structure had a good seismic behavior although some torsional effects appeared. At a drift of 1/200 which is the Code limit for masonry constructions the buildings was in a repairable stage. Finally, the testing was completed after reaching a lateral drift of 1/65.

Fig. 9 – Test on a full scale two-story masonry house at UNI-CISMID.

6. Challenges for the proper application of the masonry Code.

Although the Peruvian Masonry Code is solidly based on experimental research and theoretical background, a series of challenges appear that limit its proper application to the general community. Peru is a country under development, where many people in rural areas and marginal urban areas have limited economic resources. The national government and local municipal authorities are not capable to widely enforce the use of the Code considerations in the country, due to limited human and technical resources. The consequence of this combination regarding masonry constructions is a lot of informal construction, with low quality materials, a poor construction quality, absence of architects and engineers in the design and construction, buildings with inadequate seismic configurations, lack of lateral rigidity, low seismic resistance, among other deficiencies. In the 2007 Pisco earthquake (M8.0), about 220 km south of Lima, severe damage was reported in many masonry buildings [14, 15].

In terms of the masonry units, clay bricks used informally for construction of structural walls are hollow bricks with vertical holes or the hollow bricks with horizontal holes, which are forbidden for use in seismic areas by the Code (Fig. 10). Tie-columns and masonry walls should have either a toothed connection or a vertical connection with steel wires entering the masonry and anchored into the tie-columns. Another important issue is that the correct construction sequence for confined masonry walls has not been followed in some cases. The pouring of concrete into tie-columns should be performed after the masonry panel is constructed. When the wall construction is performed in any other way, the bond between tie-columns and masonry walls is very poor, and the vulnerability to out-of-plane forces is very high, thus failure by overturning may occur (Fig. 11). Poor structural configuration (e.g. soft storey, torsional effects, low wall density), the lack of rigid diaphragms, the absence of tie-beams, too long distance between confining tie-columns among several other defects, were found all around the affected area after the 2007 Pisco earthquake. Similar defects in the masonry construction and its structural characteristics have been observed in other urban and rural areas of Peru. Therefore, a great effort must be made by the government authorities to enforce the use of good construction practices, by the civil and structural engineers to use the Code properly, by the academics to disseminate the lessons from previous earthquakes and teach how to design and build masonry walls to ensure earthquake-resistant construction.

Fig. 10 – Hollow bricks with horizontal holes used informally in structural walls

Fig. 11 – The 2007 Pisco earthquake revealed incorrect construction sequence in confined masonry

7. Conclusions

The masonry code in Peru has been revised and updated since 1970, with different versions issued in 1977, 1982, and 2006. Many years of experimental studies on masonry specimens built using local materials and numerous analytical studies are the basis for the current Code which was issued in 2006. Masonry code is the only code in Peru with a performance-based design for masonry structures subjected to moderate and severe earthquakes. However, its use is quite limited, due to the lack of control by the authorities and the limitation of economic resources by the house-owners. Hence, low quality materials and poor workmanship are common, making many masonry constructions vulnerable for the effects of moderate and severe earthquakes.

8. Acknowledgements

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