DESIGN PROPOSAL OF CONFINED MASONRY BUILDINGS

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Abstract

The design procedure included in the Peru new Masonry Design Code called "Norma E.070" (SENCICO 2006) for confined masonry buildings is presented. This procedure is based on experimental tests performed in Peru and other countries, theoretical studies, and the performance of actual masonry buildings during past earthquakes. In this design procedure, two seismic design levels are considered. For moderate earthquakes the structure is designed to behave in the elastic range, while for severe earthquakes, the structure behaves nonlinearly and provisions are given to limit lateral drifts and prevent strength degradation, so the structure can be economically repaired.

Introduction

Small to medium height buildings in urban areas of Peru are mostly constructed using confined masonry. In these buildings, masonry walls are erected first and reinforced concrete confinements are cast afterwards (Figure 1). Vertical confinements are cast directly against the masonry walls and later, horizontal confinements, anchored on the previous ones, are cast monolithically with the slab. This construction sequence produces an integral system of all the involved elements, which behaves different than infill walls.

Recently, the new masonry design code in Peru has been approved (SENCICO 2006). The previous code (ININVI 1982) was based on allowable stresses. This paper presents the new code design approach applied to confined masonry buildings. It features strength design approach and seismic performance for two levels, moderate and severe earthquakes. A first

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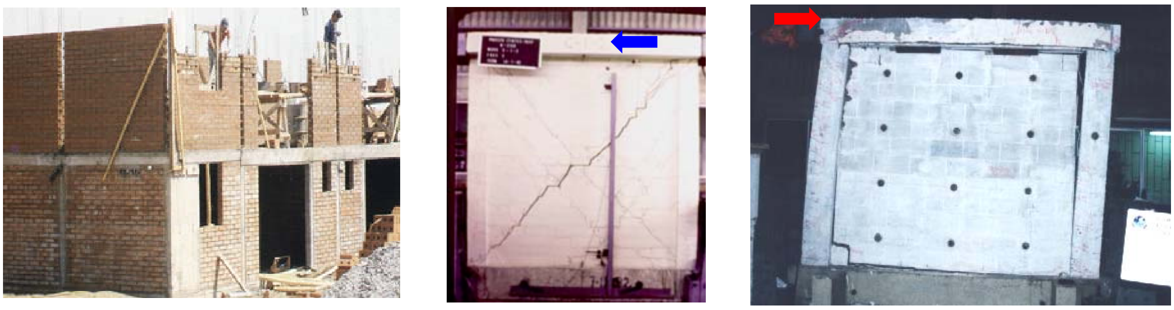
draft version of these provisions was presented by San Bartolomé and Torrealva (1990). More recently, an improved draft was published by San Bartolomé et. al. (2004).



Confined masonry   Infilled frame

Figure 1. Confined masonry construction exhibits different behavior than infilled frames.

General Requirements for Masonry Units

Regarding the masonry units, the new Peruvian Masonry Design Code establishes in Table 1, a classification for structural purposes, based on the maximum dimensional variation, the concavity or convexity, and the unit compressive strength. Only the values for bricks are included in Table 1. For bearing walls, solid units are required. Solid units are defined as those with a net cross-sectional area in every plane parallel to the bearing surface, equal to 70% or more of its gross cross-sectional area measured in the same plane.

Table 1. Masonry Unit Types For Structural Purposes

Type	**DIMENSIONAL VARIATION** (maximum in percentage)			**CONCAVITY or CONVEXITY** (maximum in mm)	**UNIT COMPRESSIVE STRENGTH** f_b' minimum in MPa over gross area
	Less than 100 mm	**Less than 150 mm**	**More than 150 mm**		
Brick I	± 8	± 6	± 4	10	4,9
Brick II	± 7	± 6	± 4	8	6,9
Brick III	± 5	± 4	± 3	6	9,3
Brick IV	± 4	± 3	± 2	4	12,7
Brick V	± 3	± 2	± 1	2	17,6

Methodology

The design procedure is based on numerous static and dynamic tests carried out at the Structures Laboratory of the Catholic University of Peru, theoretical analyses, and actual behavior of buildings during past earthquakes in Peru and other countries (San Bartolome, 1994). The procedure considers that: 1) the structure will behave elastically during moderate and frequent earthquakes; and, 2) a repairable ductile shear failure will occur in case of severe earthquakes.

Figure 2 illustrates the design considerations. The structure is expected to behave elastically for angular distortions smaller than 1/800. Diagonal cracking in a masonry wall occurs at this point and the corresponding shear force is taken by the confinement elements, which should be designed for this purpose.

Laboratory tests have demonstrated that: 1) damage is economically repairable for inelastic angular distortions smaller than 1/200 (San Bartolomé, 1994); and, 2) there is no lateral strength reduction when the confinement elements are designed to sustain the load that causes the wall diagonal cracking (Vm). Also, the summation of the confined masonry wall strengths in each direction (ΣVm) should be at least equal to the base shear load (V).

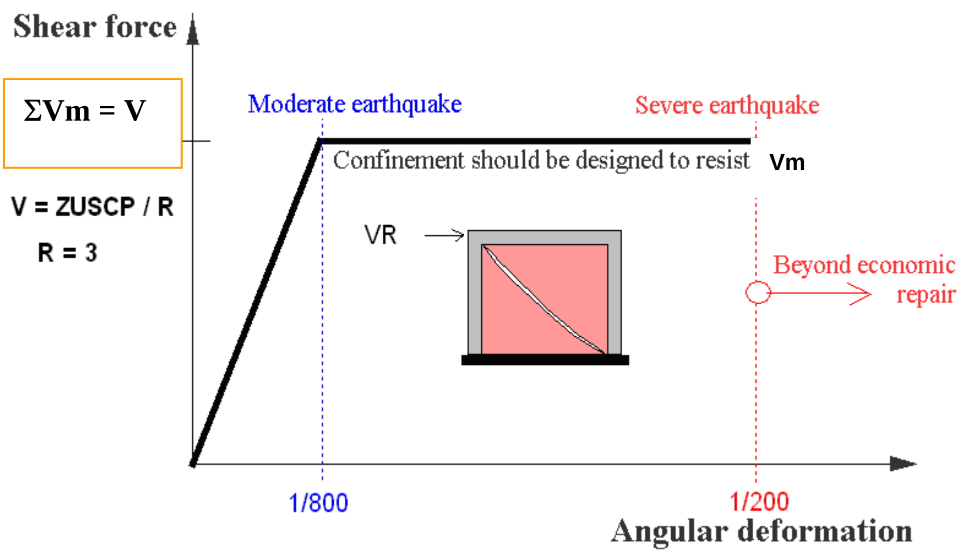


Figure 2. Objectives of the design procedure; V: Seismic design base shear; Z: Ground acceleration according to the zone; U: Building use factor; S: Soil factor; C: Building response coefficient; P: Building weight; R: Reduction factor = 3 for masonry (SENCICO 2003).

It is widely accepted that buildings made of confined masonry walls exhibit shear failure particularly in its lower stories, when subjected to severe earthquakes due to the predominance of the shear deformations over the flexural deformations. Although shear failure is considered brittle, confined masonry may exhibit ductile behavior provided that the confinement elements are properly designed, i.e. able to resist Vm.

Design Procedure

The proposed design procedure consists of five steps: 1) verification of the minimum wall density along the building main directions; 2) vertical load design; 3) elastic analysis for moderate earthquake loads; 4) verification of the elastic shear force against the shear strength Vm; and, 5) design for severe earthquake loads.

Verification of the minimum wall density

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

$$\frac{\Sigma\, Lt}{A_p} \geq \frac{Z U S N}{56} \qquad \textbf{[1]}$$

Where Z, U, and S are defined in the Peruvian Seismic Code (SENCICO 2003), N is the number of stories, L, the total confined masonry wall length, t, wall thickness, and A_p, the typical story area. If Eq.1 is not satisfied, some masonry walls may be replaced by reinforced concrete walls or the wall thickness should be increased. To use Eq.1 in the former case, the RC wall should be transformed to masonry using the transformed section principle.

In the 2001 Atico earthquake in south Peru, several failures occurred due to insufficient wall density in the direction parallel to the facade of the building (Figure 3).

Figure 3. Collapsed house in the Atico earthquake.

Design for vertical load

The axial stresses on the walls have to be calculated by any rational method. Experiments have shown that large axial stresses 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 (ASTM 2003). To reduce the wall stresses, two-direction slabs, which distribute the weight on two directions, may be considered. If the axial stress exceeds $0.05f'_m$, a minimum horizontal steel ratio equal to 0.001, as shown in Figure 4, is required. The steel diameter should not be larger than 6mm and it must be anchored in the vertical confinements.

Figure 4. Horizontal reinforcement in the layer anchored to the vertical confinement.

Two walls tested under cyclic lateral load with axial stresses equal to $0.09\ f'_m$ are shown in figure 5. Wall MV1 has not horizontal reinforcement whereas wall MV2 has a horizontal steel ratio equal to 0.001. The envelopes of lateral load-displacement are shown in figure 6.

Figure 5. Confined walls tested under cyclic load and constant axial load; left: wall MV1 without horizontal reinforcement, and right: wall MV2 with horizontal reinforcement.

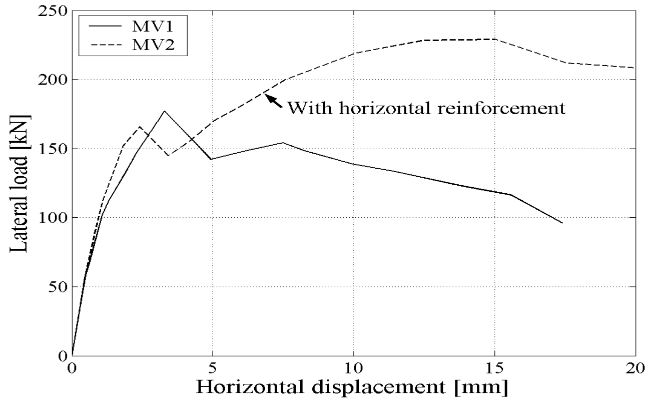


Figure 6. Envelopes of cyclic load tests of confined masonry walls shown in figure 5.

It is clear that MV2's lateral strength is higher in the inelastic range, due to the horizontal reinforcement, which limits the masonry damage and controls the strength degradation.

Elastic analysis for moderate earthquake

The moderate earthquake is defined in the Masonry Code as the one that produces half of the seismic forces of the severe earthquake. In the structural modeling, the effects of the slab rigid diaphragm, parapets integral with the structure, and walls perpendicular to the analyzed direction should be considered. Because the confined masonry walls consist of two different materials, the transformed section criterion may be used to homogenize the structure. In order to simplify the modeling, it is recommendable to separate the window parapets as shown in Figure 7. This prevents the wall stiffening due to the reduction of the unsupported height thus reducing the possibility of shear force concentration as shown in Figure 8, and torsion effects. The shear forces obtained from the elastic analysis (V_e) should not exceed 0.55Vm to assure the wall elastic behavior in moderate earthquakes.

Figure 7. Independent parapets.

Figure 8. Shear failure due to wall stiffening.

Evaluation of the diagonal cracking shear load (Vm)

The code equations to evaluate the diagonal cracking shear load for confined masonry walls were established based on the results of many experiments on full size and small walls. Equation 2 holds for clay and concrete units, and equation 3 holds for silica lime units. For both cases, the aspect ratio α is defined in equation 4. The cracking shear force Vm should be calculated for all the walls of the buildings at every floor.

Clay and concrete units: $$Vm = 0.5\, v'_m\, \alpha\, t\, L + 0.23\, P_g \qquad \textbf{[2]}$$

Silica Lime units: $$Vm = 0.35\, v'_m\, \alpha\, t\, L + 0.23\, P_g \qquad \textbf{[3]}$$

$$1/3 \le \alpha = \frac{Ve\, L}{Me} \le 1 \qquad \textbf{[4]}$$

The variables in equations 2, 3 and 4 are as follows: *v´m* is the diagonal shear strength of small square walls (ASTM 2002); P_g is the wall axial load; *Ve* and *Me* are the shear force and bending moment obtained from the elastic analysis, respectively.

The in-plane aspect ratio height-to-length (h/L=1/α) of the wall has shown its influence in the shear strength Vm, in several tests performed on walls under cyclic loading (Figure 9). This effect was also observed on a 3-story specimen with slender walls (Figure 10) tested on a shaking table (San Bartolomé, Quiun and Torrealva, 1992).

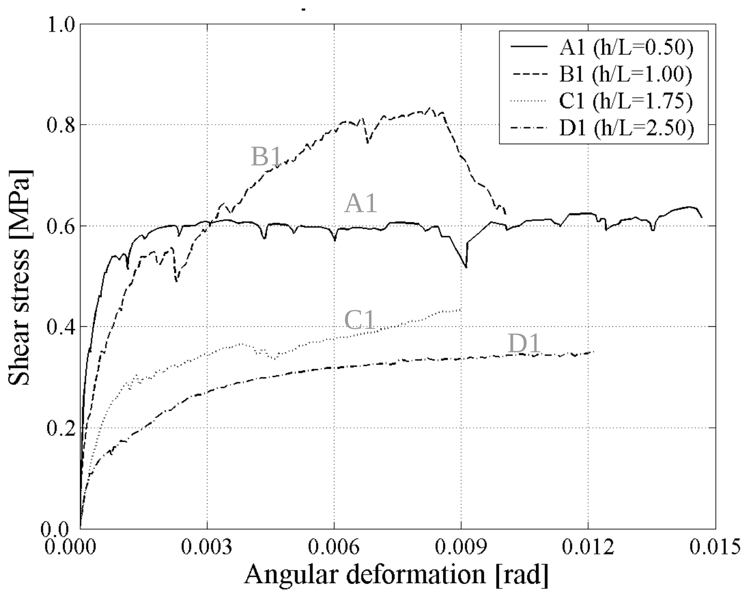


Figure 9. Influence of wall aspect ratio in Vm. (San Bartolomé, 1994)

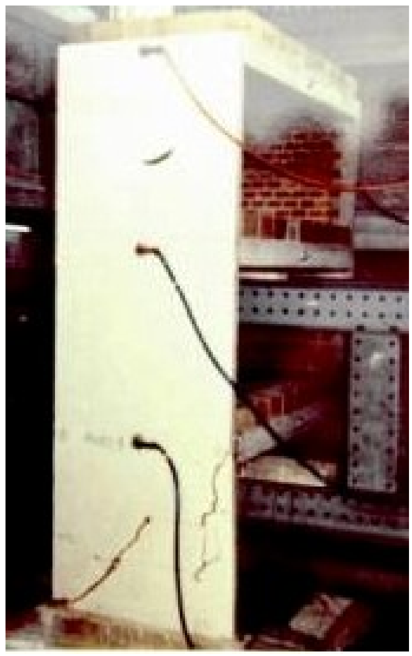


Figure 10. 3-story specimen tested on a shaking table.

Design for severe earthquake

This step consists of several sub-stages:

Verification of the building global strength:

Considering the Vm values already calculated, the summation of the shear strength of the first story (ΣVm_1) is determined. This should be larger than the seismic design shear load V. If the strength is insufficient, some masonry walls may be replaced by reinforced concrete walls or the wall thickness may be increased. If ΣVm_1 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.

Evaluation of the amplification factors and verification of the diagonal cracking of the walls in the stories above the first floor:

It is assumed that during a severe earthquake, each wall of the first floor crack when the shear force reaches its strength Vm_1. In order to obtain the ultimate bending moment and shear forces in the upper floors (M_u V_u), the calculated elastic internal forces (M_e, V_e) should be amplified by Vm_1/V_{e1}, where V_{e1} is the elastic shear force at the first story. The amplification factor should be calculated for each wall and does not need to be higher than R. If the ultimate shear force at i-th story wall, V_{ui} (i>1), is larger than Vm_i, the wall at this level will also crack and its confinements should be designed accordingly.

Evaluation of the internal forces of the first floor vertical confinements:

The first floor elements should be given special attention because they are subjected to the larger loads and generally present shear failure. The vertical confinement internal forces may be calculated for simple cases, such as one bay cantilever walls, using equilibrium equations as shown in Figure 11. There is no bending moments, because the column has not flexural deformation. For more complex cases, such as several span walls connected through reinforced concrete beams or with transverse walls, the formulas presented in Table 2 may be used.

These formulas were obtained from the analysis of models as shown in Figure 12. They pay special consideration to the columns on the wall sides to prevent the sliding of the cracked masonry wall.

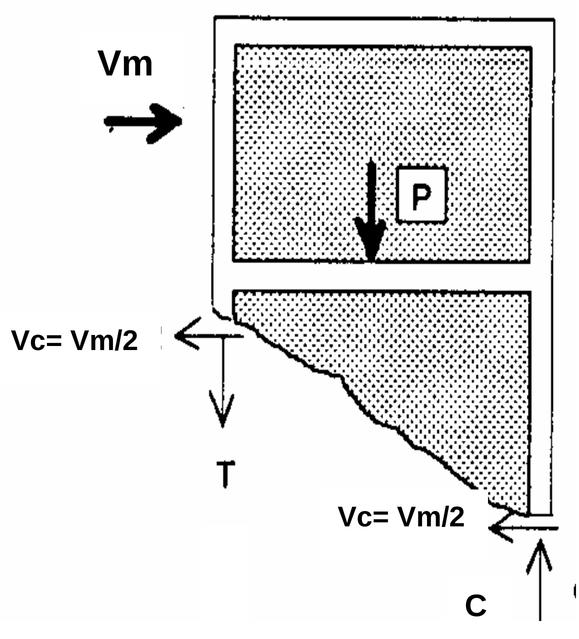


Figure 11. Vertical confinement internal forces of a one-bay wall

Table 2. Design internal forces at first story vertical confinements

VERTICAL CONFINEMENT	Shear force, V_c	Tension, T	Compression, C
Interior	$\frac{Vm_1 L_m}{L(N_c+1)}$	$Vm_1 \frac{h}{L} - P_c$	$P_c - \frac{Vm_1 h}{2L}$
Exterior	$1.5 \frac{Vm_1 L_m}{L(N_c+1)}$	$F - P_c$	$P_c + F$

L_m: Longest wall span ≥ 0.5L. For one span walls, L_m = L
L: Total wall length including vertical confinements
N_c: Number of vertical confinements. For one span wall, N_c = 2
P_c: Vertical column load (including the load from the transverse walls)
F: Axial load due to bending moment = M / L = (M_{u1} – 0.5×Vm_1×h_1) / L
h_1: First story height

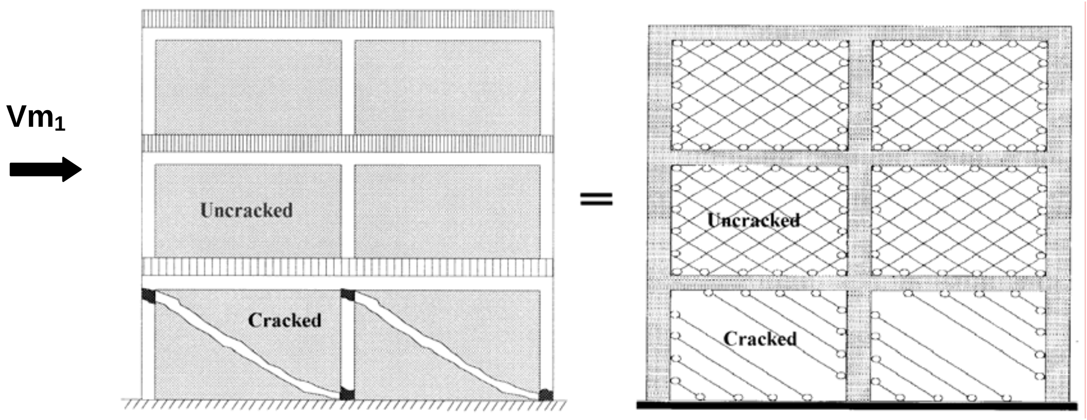


Figure 12. Model used to calculate the forces at the wall confinements in complex cases.

If sliding is avoided, the cracked walls inside the confinements provide lateral load resistance as shown in Figure 13. This fact is considered in the evaluation of V_c in Table 2.

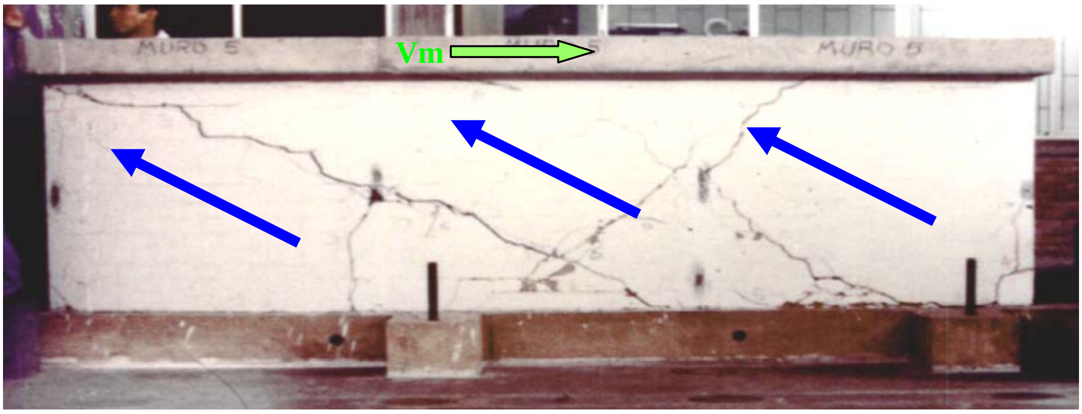


Figure 13. The cracked masonry wall contributes to the lateral strength.

Design of the first story confinements:

The vertical confinements are designed with the ultimate internal forces shown in Table 2, according to concrete design standards. The vertical confinement is subjected to a combined shear-friction and tension mechanisms. A minimum longitudinal reinforcement equal to 4 bars (8mm diameter) is specified. The concrete core section (inside the stirrups) and the shear reinforcement are dimensioned to prevent concrete crushing (Figures 14 and 15). The total confinement cross section area should not be less than *150 t* in mm^2.

The horizontal confinements should be able to transfer the seismic loads from the slab to the masonry wall. For this purpose, they are designed for a tension T_s given in equation 5. Only minimum stirrups have to be provided, as the horizontal confinements do not have significant shear loads, because the shear area above the cracked first floor is large (see Figure 11).

$$Ts = 0.5 \cdot Vm_1 \cdot {}^{Lm}\!/_{L} \qquad \textbf{[5]}$$

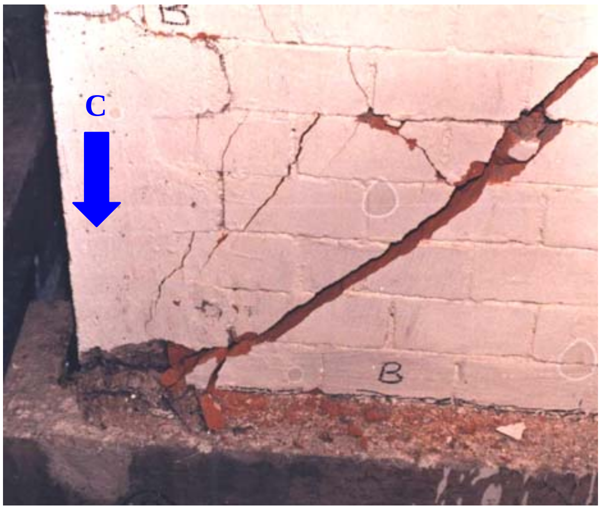


Figure 14. Crushed concrete column


Figure 15. Concrete cover spalling and undamaged concrete core.

The minimum specified stirrups for both vertical and horizontal confinements is bars of 6mm diameter, spaced at 100mm on the element ends, length equal to 1.5 the element depth or 450mm, and 200mm in the rest of the element.

Design of the confinements of the stories above the first floor:

In case V_{ui} is smaller than Vm_i, the masonry wall resists the seismic forces without cracking. In such case, the vertical confinements should not be designed considering the shear-friction effect. Instead, only the external confinements are designed for the tension, T, and compression, C, produced by the flexural moment $M_{ui} = M_{ei} \times Vm_1 / V_{e1}$. The internal columns do not need to be designed for in-plane actions, because they are integrated to the uncracked masonry wall. However, they should be able to support the wall under out-of-plane seismic actions. The maximum spacing between columns should not be larger than twice the distance between horizontal confinements. The horizontal confinement should be designed by tension, produced by the transmission of seismic forces to the walls.

Conclusions

The Peruvian Code design method has been successfully verified with static and dynamic tests performed on confined masonry walls at natural and reduced scales. The design procedure considers that the structure will behave elastically for moderate earthquakes and nonlinearly for severe earthquakes. In the presented approach, the shear failure of masonry walls is considered acceptable provided that: 1) the inelastic lateral displacements are limited by a sufficient wall density; 2) the confinements are designed to carry the seismic load after the wall cracks; and, 3) the bricks must be solid. In order to increase the wall ductility, horizontal continuous reinforcement may be placed in the mortar joints. Experiments have shown that the optimum horizontal reinforcement ratio is 0.001. Increasing this value twice may improve the ductility but keeps the wall strength almost unchanged.

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