SEISMIC REINFORCEMENT OF EXISTING WALLS MADE OF HORIZONTALLY-HOLLOW BRICKS

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ABSTRACT

Horizontally-hollow brick walls are very popular in masonry buildings in Peru due to economic reasons and lack of control. However, their seismic behavior is very fragile and their resistance is relatively low, and therefore the Peruvian Masonry Code does not allow the use of such bricks for bearing structural walls. This paper deals with experimental research on existing walls made of horizontally-hollow bricks and a way to reinforce them, in order to enhance the seismic performance and avoid their collapse during severe earthquakes.

Two full scale confined masonry walls were built using horizontally-hollow bricks, wall W1 had a traditional construction, while for wall W2 after its construction, a welded mesh covered with cement mortar was attached to the wall to study the seismic behavior. Both walls were subjected to cyclic lateral load tests. The reinforced wall W2 had significant improvements in the behavior respect to wall W1. Larger values were obtained for the lateral rigidity (41%), for the load that produces tension cracks by flexure (13%), for the diagonal cracking load (34%) and for maximum lateral load (42%).

KEYWORDS: seismic retrofit, experimental techniques, horizontally hollow bricks

INTRODUCTION

Many people in Peru and other countries under development build houses by self-construction. Due to economic reasons, regardless the safety or regulations, this people usually choose the cheapest materials. In Peru, horizontally hollow clay bricks (called “pandereta”, see Figure 1) are used for bearing walls, although such brick units were conceived for use in non structural walls. They are forbidden for use in bearing or structural walls by the Peruvian Masonry Code (“Norma E.070” in Spanish); however, people use it, despite the vulnerability involved. This paper deals with a reinforcing methodology for existing structural walls made with horizontally hollow bricks. The 2007 Pisco earthquake in Peru (M8.0) showed the fragility of such walls and the need to prevent collapses of such walls in future events (San Bartolomé and Quiun, 2008).

Previous experimental research has shown successful results when wire meshes covered with mortar were used as reinforcements, in masonry walls made of hollow clay bricks (more than 40% of holes in the bed area) or concrete blocks without grout were used in the walls (San

Bartolomé et. al. 2008, 2012 and Quiun et. al. 2005). Those walls were tested under lateral cyclic load reaching important shear cracks; afterwards, they were repaired and reinforced with wire meshes, and tested again with similar load patterns, recovering the initial resistance. In this paper, we report the tests of two confined walls built similarly with horizontally-hollow clay bricks, W1 in the traditional way and W2 with external wire mesh reinforcement.

Figure 1: Horizontally hollow bricks used in self-constructions for bearing walls in Peru

MATERIAL PROPERTIES

The mechanical properties of the construction materials are shown hereafter. The bricks were industrial horizontally hollow clay units, 90x110x230mm size, with channels at the sides as observed in Figure 2. The dimension variation was 4.6%, warping was 0.3 mm, compressive strength f'_b was 5.6 MPa, absorption reached 46% and the natural suction was 46 gr/(200 cm^2-min). In order to reduce the suction prior to laying, the bricks were wetted for 30 minutes about 10 hours prior to be laid.

Figure 2: Horizontally hollow bricks and suction test

The mortar was mixed in volumetric proportion of 1:4, Portland cement - coarse sand. The horizontal and vertical joints were set to 15mm thickness. The concrete for confinement columns and collar beams was specified to have a 17.5 MPa compressive strength. The reinforcing steel bars had yield stress of 420 MPa. The steel wire mesh was composed of 4.5 mm diameter deformed bars with 150 mm spacing, meeting the ASTM A496-94 and ASTM A497-94 standards; the maximum strength tension was 550 MPa. The connectors of the mesh used in wall W2 were #8 wires every 450mm.

Small masonry specimens of prisms and wallets of these horizontally hollow bricks were constructed and tested. Four prisms of 110x230x600 mm were subjected to axial compression tests, leading to a compressive strength of 2.4 MPa. The failure in all cases was very fragile and explosive, which show us how dangerous is the use of such bricks in bearing walls as shown in Figure 3. Also, four wallets of 110x600x600 mm were built and tested under diagonal compression, reaching a shear strength of 0.93 MPa. The same kind of fragile and explosive failures were observed (Figure 3).

Figure 3: Tests performed in prisms and wallets.

WALL PROPERTIES AND CHARACTERISTICS

The properties of wall W1 were calculated using the Peruvian Masonry Code ("Norma E.070 Albañileria" in Spanish) in order to predict the test results. The masonry elastic modulus used was obtained from the axial compression test of the prisms, E_m = 2530 MPa; the masonry shear modulus was taken as G_m=0.4E_m, that is G_m=1013 MPa; the concrete elastic modulus was taken as E_c=19440 MPa. With these values, the elastic modulus ratio between concrete and masonry is found to be n=E_c/E_m = 7.68.

An elastic model was used to evaluate the lateral rigidity (K) and the capacity of tension by flexure (F). Using the transformed section criteria, the concrete columns were transformed into equivalent masonry area using n=7.68. The elastic lateral rigidity K was obtained using a cantilever model with flexure and shear deformations, and the wall height from the foundation to the collar beam axis of 2.3m, giving K=99 kN/mm. The capacity of tension by flexure was evaluated considering the maximum tension stress at the concrete column (neglecting the wall weight), due to the flexural moment at the wall base caused by the lateral load at the collar beam. Under these assumptions, the capacity of tension by flexure was obtained as F=76.2 kN.

The wall shear force capacity Vm, was obtained using the Peruvian Masonry Code expression (Equation 1) in which for square walls, α=1; and for walls without axial load, Pg=0. The shear capacity obtained under these condition was Vm=130 kN.

$$Vm = 0.5 \propto v'm\ t\ L + 0.23\ Pg \qquad (1)$$

The wall flexural capacity Vf, was calculated neglecting the wall weight, and considering the yield of the vertical bars on the base of the columns in tension. The flexural moment caused by the lateral load (Vf h), was equated to the resisting moment caused by the yielding bars (As fy D), in which the effective distance "D" was taken as D=0.8L. With these assumptions, the flexural capacity is Vf=192 kN.

In wall W1 the flexural capacity value of Vf was found to be larger than the shear capacity value Vm. Therefore, the expected failure should be by shear; previously, cracks due to tension by flexure should develop in the concrete columns. This kind of failure occurs frequently in real confined masonry constructions subjected to severe earthquakes.

In wall W2 the flexural capacity is the same, as the wire mesh is not connected to the foundation. The shear capacity was obtained using Equation 1 with α=1, Pg=0, and a thickness "t" increased by the cover mortar in 50 mm, making t=160mm; therefore, Vm (W2) = 189 kN. The flexure and shear capacity obtained for W2 are quite similar, making the kind of failure unpredictable.

Both walls W1 and W2 had the same geometric characteristics, similar materials, reinforcement in the confinements, were built by the same masons (hand labour) and followed the same construction sequence. Figure 4 shows the wall characteristics and reinforcements. The columns had 130x200 mm dimension, reinforced with 4-12.7 mm (1/2") bars and stirrups of 6 mm (1/4"), 1@50mm, 4@100mm, and rest @200mm. The collar beam was 130x200mm dimension, reinforced with 4-9.5 mm (3/8") bars and stirrups of 6 mm (1/4"), 1@50mm, 4@100mm, and rest @200mm.

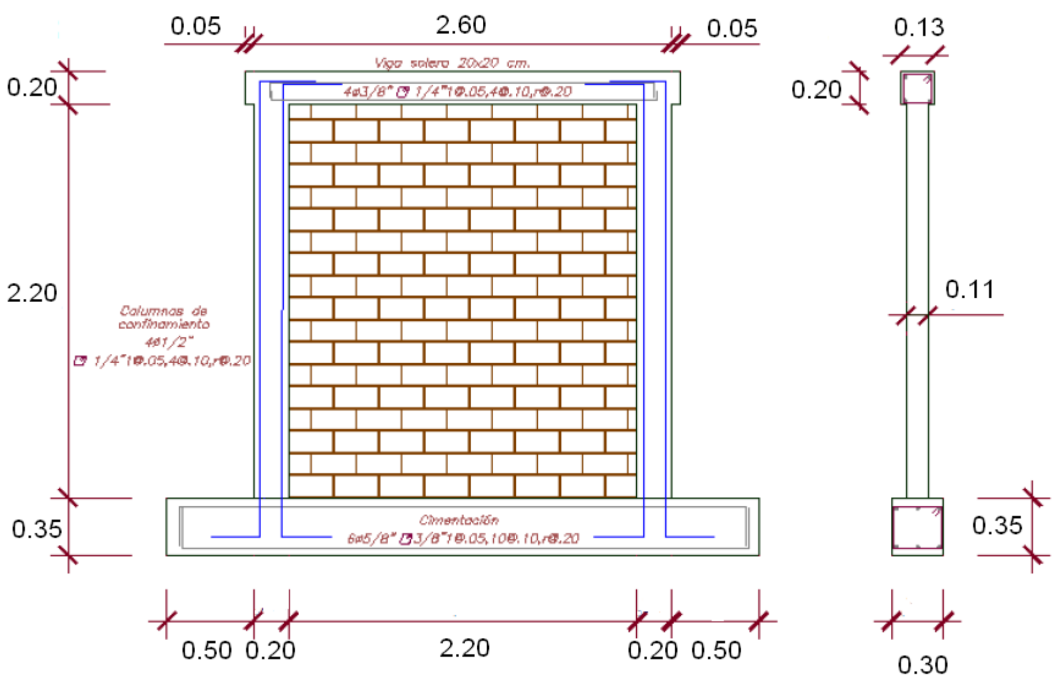

Figure 4: Walls W1 and W2 geometric characteristics with dimensions in meters.

In wall W2, a wire mesh was added over the masonry on both sides, connected by smaller wires #8 every 450mm, finally covered with a mortar of 25mm thickness, as shown in Figure 5. The wires of the mesh had no connection to the concrete columns or to the foundation. The purpose of the mesh was the reduction of the diagonal shear cracking in the masonry and the control of the crushing of the horizontally hollow clay bricks.

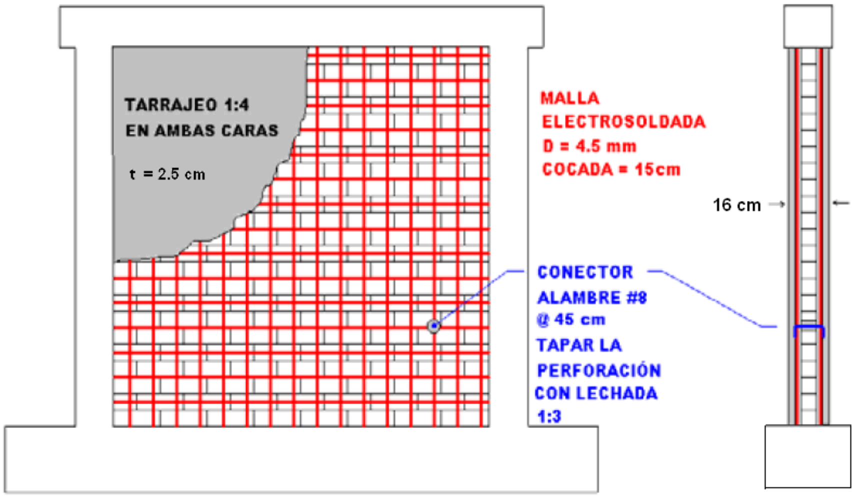

Figure 5: Wire mesh reinforcement used in wall W2.

CONSTRUCTION OF THE WALLS AND REINFORCEMENT OF WALL W2

Besides using the same materials, hand labor and confinement reinforcements, the construction specifications for both walls were as follow. The bricks were laid so that the wall thickness was the smaller side of the bricks. The mortar mix proportion was 1:4, cement - sand. The horizontal and vertical joints were of 15mm thickness. The bricks were wetted for 30 minutes, 10 hours prior to the construction. The connection between columns and walls was toothed; the horizontal holes of the bricks were covered with paper, leaving 20mm for the concrete of the columns to enter as shown in Figure 6. The masonry wall was erected in two days, then the concrete of the columns was poured, and finally the collar beam was poured. The concrete was water cured once a day for one week. After 28 days of construction the walls were ready for testing.

Figure 6: Construction of walls W1 and W2.

In the already constructed wall W2, the wire mesh was installed on both sides (Figure 7). The bricks were perforated for the connection wires #8 every 450mm, which is 3 times the wire spacing. The connector wires were bent 90° and tied to the meshes with smaller wires. The perforations were then filled up with grout of cement-fine sand in 1:3 by volume. The final cover was completed in two steps, with mortar of cement-sand in 1:4 by volume (Figure 8).

Figure 7: Wire meshes added to W2, detail of connection wire and filling the perforations

Figure 8: Cover of wire meshes in W2, done in two steps

CYCLIC LATERAL LOAD TEST

The cyclic lateral load test was performed by setting the top displacement D1, in increasing amplitude in 10 steps, as indicated in Table 1. The test for wall W1 only reached step 8. Finally, a harmonic lateral displacement was applied.

The set of instruments LVDTs used in the wall is shown in Figure 9: D1 was used for the test control of horizontal displacement, D2 recorded the diagonal crack width in the middle of the masonry wall, D3 and D4 were used to monitor if the columns and masonry wall get separated, and D5 and D6 recorded the vertical column low ends displacements.

Table 1: Steps of Cyclic Load Test

Step	1	2	3	4	5	6	7	8	9	10
D1 (mm)	0.5	1.5	2.5	5.0	7.5	10.0	12.5	15.0	17.5	20.0
Number of cycles	1	2	3	3	3	3	3	3	3	3

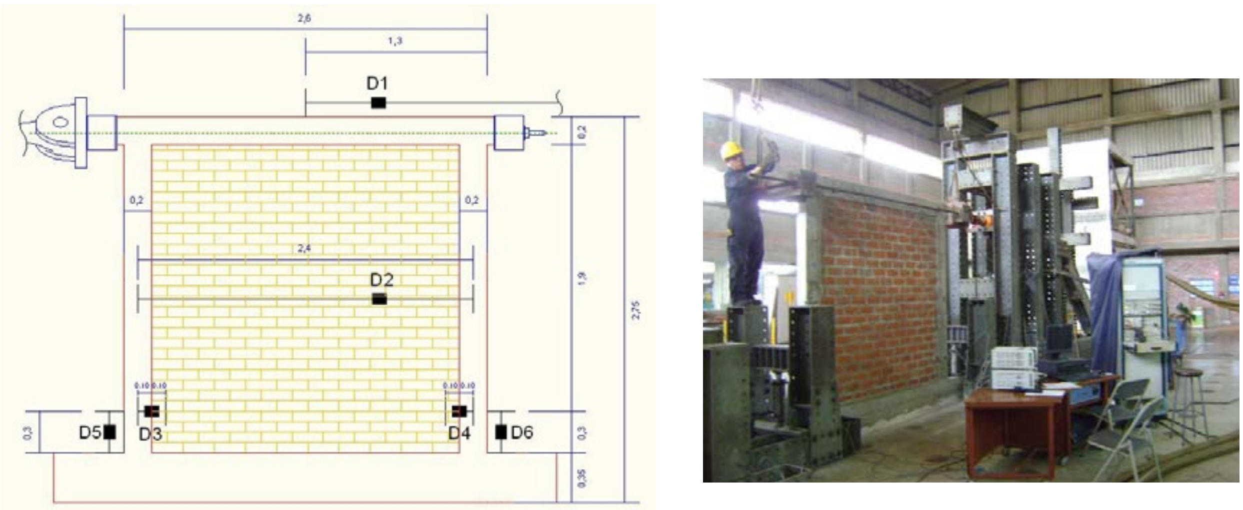

Figure 9: Instruments used in walls W1 and W2

In wall W1, the columns started cracking in step 2 and the full diagonal cracks appeared in step 4, with a lateral load of 148 kN and a crack width of 3mm. Some bricks located in the upper part of the wall near the columns began to crush in step 6 and were quite damaged at the end of last step 8. The test in wall W1 was then stopped due to the load degradation registered.

In wall W2, no cracks were observed in steps 1 or 2. In step 3, the columns started cracking by flexure. In steps 4 and 5, some horizontal cracks appeared in the bottom of the columns that extended diagonally into the wall. In steps 6 and 7, vertical cracks appeared at the bottom part of the wall-column connection; these cracks in step 8 reached the wall mid height, while the lateral load was 210 kN. In step 9, some sliding was noticed between the masonry wall and the foundation beam, also the columns bottom started crushing. In step 10 (last one), the masonry sliding reached 15mm, the vertical cracks reached 15mm width and the covering of the wire mesh started to spall apart. Figure 10 show both walls at the end of this part of the test.

After the cyclic load test, a second part of the test consisted in a harmonic load with an amplitude of 15mm and a frequency of 2 Hz. The damage in both walls increased, with more crushing of the bricks in wall W1 and a larger crushing of the column bases of wall W2. In wall W1 the failure was unsymmetrical, the diagonal cracks did not intersect, and therefore the central region of the masonry wall remained undamaged. At the end of the harmonic tests, the final failure was by sliding for both walls. The sliding was located at the upper layer one before the last for W1 and at the wall base for W2.

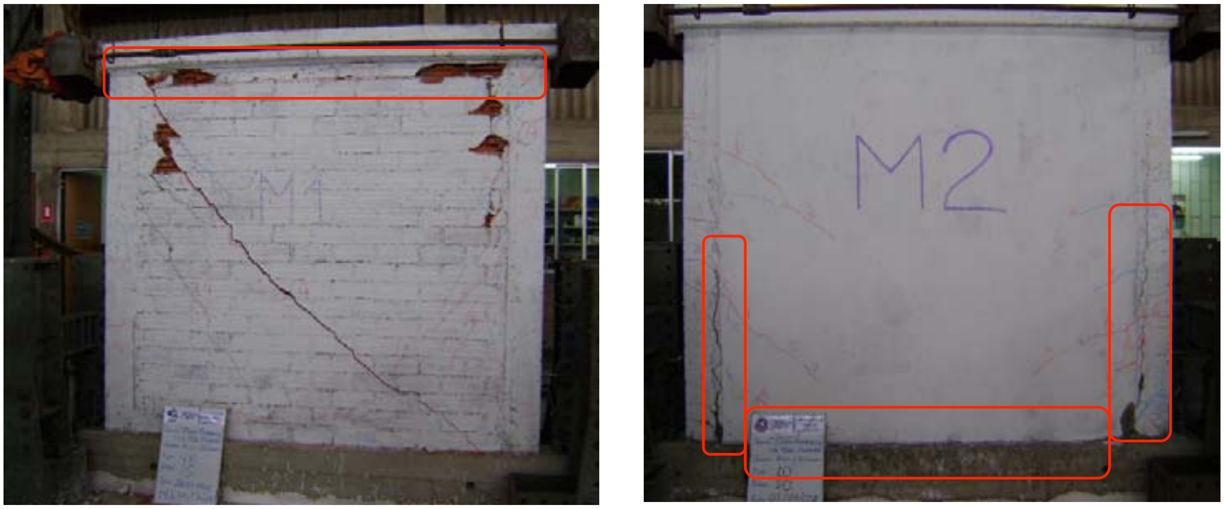

Figure 10: Walls W1 (left) and W2 (right), the main cracks are marked.

After the end of the test, for wall W2 an inspection was conducted by exploring the connection between the masonry and the right column. The vertical crack that appeared was located in the cover of the longitudinal internal bar (Figure 11). Therefore, when this bar lost its bond with the concrete, it was not able to resist the lateral load effects. The damage also included the concrete crushing at the base of the columns and the already mentioned sliding at the base, all of which contributed to the degradation of shear force capacity of this wall.

Figure 11: Wall W2 inspection after the tests

TEST RESULTS

The hysteretic loops of lateral force vs. lateral displacement for both walls are shown in Figure 12. Both graphs are non symmetrical, because in wall W1 the shear failure was unsymmetrical while in wall W2 an internal void in one of the columns was found; besides, the vertical cracks that developed at the column-wall connections had different lengths. Moreover, in wall W2 it can be noted that after step 7 (D1=12.5mm) large permanent displacements with null load happened due to the sliding of the wall along its foundation.

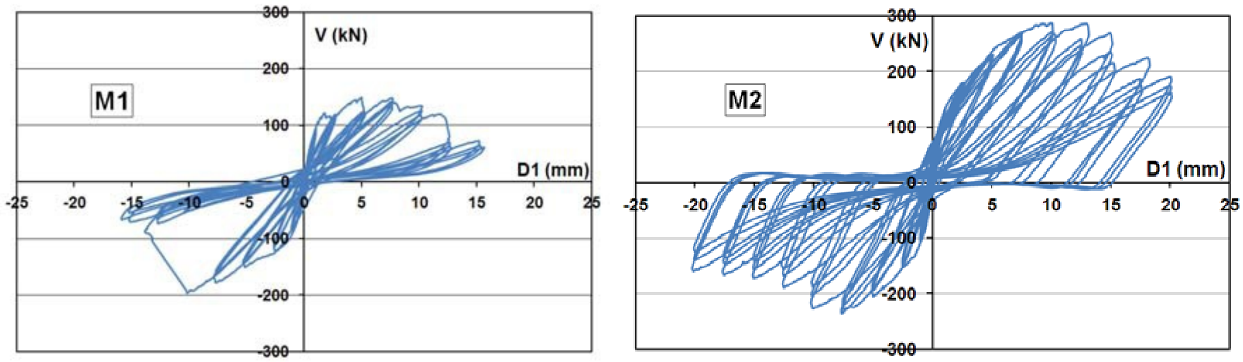

Figure 12: Wall W1 (left) and W2 (right) hysteresis loops

The crushing of the horizontally hollow bricks in wall W1 was very clear from step 6 and further. This corresponds to a drift of 0.0043, less than the Peruvian Seismic Code ("Norma E.030" in Spanish) allowable drift, which is 0.0050 for masonry structures. The brick crushing also produced a severe loss of lateral load capacity. Therefore, it is quite obvious that the horizontally hollow bricks should not be used in bearing structural walls. Besides, the test conditions had no vertical load, the bending moments were small, and the loads were applied slowly. In real earthquake conditions, the brick crushing should occur for smaller drifts.

In wall W2, the capacity degradation also occurred in step 6, before the drift reached the allowable Code limit. However, we presume that this problem could be avoided if the vertical cracks at the column-wall connections are controlled, maybe by extending the horizontal bars of the wire mesh into the columns or by adding extra dowel bars.

For the evaluation of the initial rigidity K, the values of shear force V and lateral top displacement D1 of the first cycle of step 1 were used in which both walls have elastic behavior. The theoretical rigidity K for W1 was found as 9 kN/mm, while the experimental value was 108 kN/mm, a difference of only 8%. For W2 the experimental rigidity was 155 kN/mm, which is 41% larger than W1 rigidity. This increase is similar to the increase in the wall thickness for W2, from 110 to 160 mm (45%), due to the mortar cover of the meshes on both surfaces.

Regarding the tension by flexure, the first cracks appeared in wall W1 columns for step 2 of the test, with a lateral load of 93.5 kN, 23% larger than the theoretical value of 76.2 kN. For wall W2, this kind of failure occurred at step 3 with an associated lateral load of 105.8 kN.

Referring to the diagonal shear cracking, in step 4 for wall W1 this crack occurred for a lateral load of 145 kN, only 12% larger than the theoretical value of 130 kN. For wall W2, the diagonal cracks occurred for a lateral load of 194 kN, also in step 4. This value is 34% larger than the one obtained for W1, and only 2% different than the theoretical value of 192 kN.

The maximum load in the reinforced wall W2 was 42% larger than the traditional wall W1. However, this increased resistance cannot lead to major conclusions, due to the great differences of the positive and negative branches of the envelope for both walls. This behavior is attributed to the asymmetrical failure in wall W1 and the voids in one of the columns of wall W2.

CONCLUSIONS

The conclusions are limited to the few tests performed; however, it can be said that the main objective of the reinforcement (wire mesh covered with mortar) was accomplished, because the crushing of the horizontally hollow bricks was avoided. For informal or self constructions in which the bearing masonry walls are made with this kind of fragile units, the reinforcement described here can be applied as a prevention measure. This project also showed some experimental results that can be used for future researches in which the need of wire mesh reinforcements are applied over weak masonry walls.

ACKNOWLEDGEMENTS

The authors want to thank the staff at the Structures Laboratory of the Catholic University of Peru.

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