CYCLIC LOAD BEHAVIOR OF CONFINED MASONRY WALLS OF HORIZONTALLY-HOLLOW BRICKS RETROFITTED WITH WIRE MESHES

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Abstract

Structural masonry walls made of non-solid bricks are popular for housing construction in Peru due to economic reasons and lack of construction quality control. These non-solid bricks include both hollow bricks with more than 30% of holes in the bed area, and horizontally-hollow bricks (bricks with large horizontal holes) conceived for use in non-structural walls. The seismic behavior of masonry walls with such bricks is very poor and the seismic resistance is relatively low. For that reason, the Peruvian Masonry Code (Norma E.070 in Spanish) does not allow the use of such bricks for load-bearing structural walls.

This paper deals with experimental research on existing confined masonry walls made of horizontally-hollow bricks and a way to retrofit and reinforce them using welded wire mesh, in order to enhance the seismic performance and avoid their brittle collapse during earthquakes.

Firstly, two full scale confined masonry walls were built using horizontally-hollow bricks: wall W1 was constructed in a traditional manner, while for wall W2 a welded mesh was attached to both surfaces of the wall after its construction and later covered with cement mortar. To study the seismic behavior of these walls, cyclic lateral loads were applied in a displacement controlled test. In a second stage of the research, vertical load was added. Two walls, WV-1 and WV-2 were constructed, retrofitted with the wire mesh and covered with mortar. The test included vertical load before the cyclic loads were applied, similar to a 2-story (WV-1) and 3-story (WV-2) building.

The reinforced wall W2 showed significantly improved behavior compared to wall W1. Larger values were obtained for the lateral rigidity, the load that produces flexural tension cracks, the diagonal cracking load and the maximum lateral load (45%). The other retrofitted walls with applied vertical load, showed even larger maximum lateral load than the traditional wall W1 (86% for wall WV-1 and 110% for wall WV-2). These promising results indicate that a retrofitting procedure can be used to reduce the seismic vulnerability of many self-constructed (non-engineered) masonry buildings in Peru.

Keywords: masonry; horizontally-hollow bricks; cyclic load test; retrofit; wire mesh.

1. Introduction

The Peruvian Masonry Code (Norma E.070) [1] does not allow the use of non-solid bricks for structural masonry walls in seismic areas. This limitation includes bricks with holes in the bed area which exceed 30% of the gross area, and also horizontally-hollow bricks which were conceived for use in non-structural walls. The main reason for this specification is that walls built with such masonry units have experienced brittle failures followed by the collapse of the buildings [2]. This poor behavior was observed in the 2007 Pisco earthquake [3] (Fig. 1). Despite this fact, many people in Peru use such units for bearing structural walls, mainly in informal constructions, due to economic reasons (hollow units are cheaper than solid units), ignorance and lack of construction quality control. The purpose of this research was to study the seismic behavior of masonry walls built with the horizontally-hollow units by experimental tests, using wire mesh as external reinforcement attached to an existing wall. In this way, the goal is to prevent brittle failure and avoid the shear diagonal cracks in these walls, therefore, reducing their seismic vulnerability and improving their behavior under earthquakes.

Fig. 1 – Masonry buildings that collapsed in 2007 Pisco earthquake [3].

In previous research studies [4, 5], the effectiveness of welded wire mesh reinforcement was demonstrated in controlling the width of diagonal shear cracks and also increasing the capacity in stiffness and resistance of masonry walls made of hollow bricks (with holes in the bed area).

In the first part of this research [6], horizontally-hollow bricks were used in two full-scale masonry walls. Wall W1 was built in the traditional way, and wall W2 was reinforced after the construction was completed using a steel welded wire mesh, covered with cement-sand mortar. In both walls the seismic effect was simulated by cyclic lateral loads, controlled with lateral top displacements. The results obtained in these initial tests, showed significantly improved behavior of wall W2, compared to traditional wall W1 which had a poor behavior. The second part of the research is presented in this paper, in which two more masonry walls were constructed and tested using the same masonry units. Constant vertical loads were applied on the wall specimens WV-1 and WV-2, before the cyclic lateral loads. Both walls were reinforced prior to the test with wire mesh. In this way, gravity loads were applied in reinforced walls in order to represent simultaneous earthquake effects in bearing masonry walls made of horizontally hollow bricks. The objective was to study if the structural behavior of the retrofitted walls could be improved when subjected to both gravity and seismic loads.

2. Material properties

Walls WV-1 and WV-2 were constructed using the same materials. The main properties of the materials are explained below.

2.1 Horizontally hollow clay brick

The masonry unit is known as “pandereta” in Peru. The unit dimensions are 90x110x220 mm, and features some ribs in the bed area (Fig. 2). The compressive strength is f’b=5.6 MPa over the gross area, the variation on dimensions is less than 1%, the maximum warping is only 0.1 mm in the bed area, a suction of 37 gr/(200 cm^2-min) requires a wetting prior to its placement to reduce the suction to values between 10-20 [1]. The same unit was used in the first part of the research for walls W1 and W2.

Fig. 2 – Horizontally-hollow brick unit “ladrillo pandereta”.

2.2 Mortar for joints and plaster

The mortar mix was prepared as cement:sand in 1:4 volume proportion. The thickness of the horizontal and vertical joints of the masonry walls was 15 mm. The wire mesh was covered by a thin sand mortar layer in the same mix proportion, with a thickness of 25 mm.

2.3 Cement and Reinforced Concrete

Portland cement type I was used for mortar and concrete. The concrete of the foundation beam had a nominal compressive strength of 20.6 MPa, and for the confining elements (tie-columns and tie-beam) it had 18.6 MPa. The steel bars for reinforced concrete were ASTM A615, grade 60, with a yield stress of 412 MPa.

2.4 Welded wire mesh

The mesh was composed of deformed bars with 6 mm diameter, at 150 mm spacing (Fig. 3). This mesh followed the requirements of standards ASTMA496/A 496M-05a and ASTMA497/A 497M-05A [7], with a yield stress of 490.3 MPa. Also, small wires #8 were used to connect the wire mesh at each side of the walls, at 450 mm spacing in the vertical and horizontal direction.

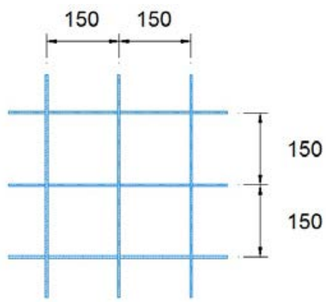

Fig. 3 – Wire mesh detail

2.5 Masonry prisms and wallets reinforced with wire mesh

Three masonry prisms were built and reinforced using the wire mesh, with 220x160x620 mm overall dimensions. They were tested under axial compression, using a load velocity of 50 kN/min. The masonry compressive strength was calculated as f'_m= 2.65 MPa, and the elastic modulus was Em=5835 MPa (Fig. 4, left). Also, three masonry wallets of 160x620x620 mm overall dimensions were constructed and reinforced with the wire mesh. They were tested under diagonal compression using a load velocity of 10 kN/min. The shear masonry resistance was calculated as v'_m=0.88 MPa, with a shear modulus of Gm=1275 MPa (Fig. 4, right).

Fig. 4 – Masonry tests on axial compression (left) and diagonal compression (right).

3. Wall specimen characteristics and reinforcement

The new walls WV-1 and WV-2 were constructed using the same geometry as previously tested walls W1 and W2 [6]. The same materials, same reinforcement in the confining elements, same workmanship, and same construction procedures were used in the new walls. After they were built, the wire mesh was placed on both sides of the masonry panel. In Figure 5 at the left, the wall elevation and section drawings are displayed, and in Figure 5 at the right, the external reinforcement of the wire mesh covered with mortar is shown.

The confining tie-columns for walls W1, W2, WV-1 and WV-2 had all rectangular cross section of 130x200 mm, reinforced with 4-12.7 mm diameter bars and 6 mm ties spaced 1 @ 50 mm, 4 @ 100 mm, rest @ 200 mm. The tie-beam also had rectangular cross section of 200x180 mm, reinforced with 4-9.5 mm diameter bars and 6 mm ties spaced 1 @ 50 mm, 4 @ 100 mm, rest @ 200 mm, as shown in Figure 5.

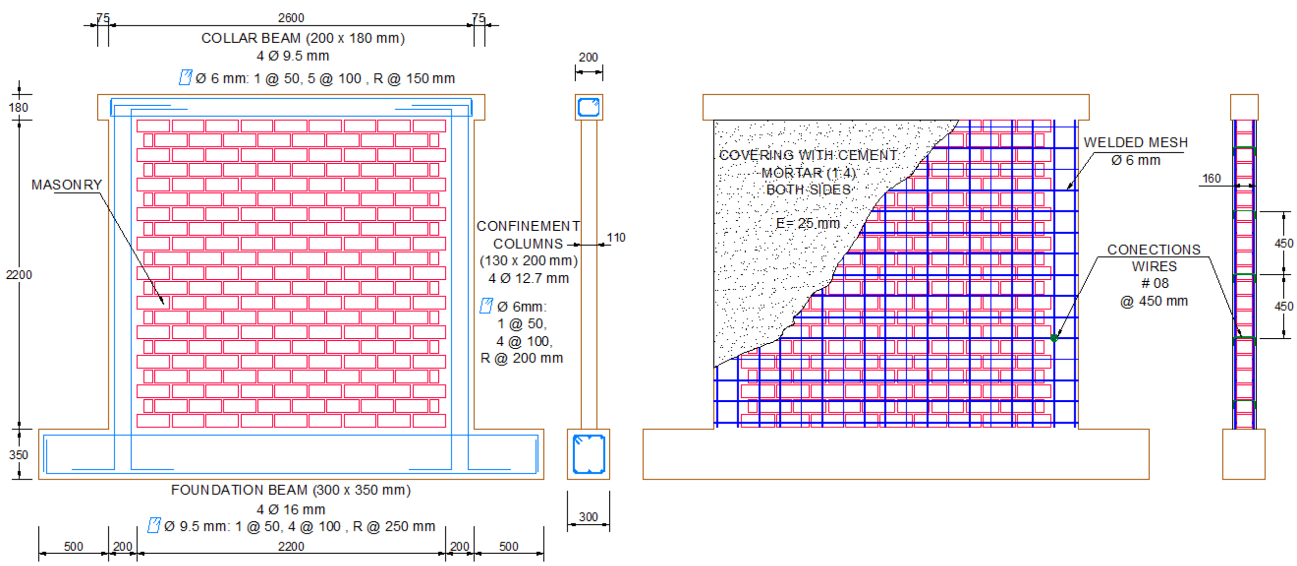

Fig. 5 – Geometry and reinforcement for confined masonry walls (left), and welded wire mesh (right).

The welded wire mesh used as external reinforcement to these vulnerable masonry walls was designed as follows. It was considered that the steel wires had to resist the load that produces diagonal shear cracking of the masonry, Vm. For the reinforced walls, Vm was calculated from Eq. (1) given in the Peruvian Masonry Code [1] for clay masonry walls. Parameter α takes into account the wall in-plane slenderness, which is equal to unity in square walls as this case; the shear resistance v'_m was 0.88 MPa, from the wallets diagonal shear test; wall thickness t includes the mortar cover of 25 mm on each side, that is, wall thickness plus 50 mm; wall length L of

2600 mm includes the length of confining tie-columns. The amount of vertical load, Pg, was set as 108 kN for wall WV-1 and 157 kN for wall WV-2, simulating gravity loads of two-story and three-story masonry buildings.

$$Vm = (0,5)\,(\alpha)\,(v'_m)\,(t)\,(L) + 0,23\,(Pg) \quad (1)$$

After replacing the abovementioned values, it was obtained that Vm= 209 kN for wall WV-1, and Vm= 220 kN for wall WV-2. With these Vm values, the steel area As for the wire mesh was obtained using Eq. (2). The wire spacing, s, was 150 mm; the yield stress fy was 490 MPa, and wall length, L, was 2600 mm.

$$As = (Vm)\,(s) / [(fy)\,(L)] \quad (2)$$

The reinforcement steel required was As1=28.7 mm^2 for Wall WV-1 and As2=30.2 mm^2 for Wall WV-2. In both walls the mesh provided was 6 mm in diameter, having As =56 mm^2 which satisfies both requirements.

4. Wall theoretical capacity and predicted mechanism failure

The wall´s theoretical capacity in lateral force was calculated using the procedures indicated in the Peruvian Masonry Code. The elastic modulus and shear modulus obtained in the small specimens' tests were used in the calculations, Em=5835 MPa and Gm=1275 MPa. The elastic modulus of the concrete Ec, was obtained using the Peruvian Concrete Code Eq. (3) [8]. The ratio of elastic modulus for the transformed section criteria in elastic stage was then determined by Eq. (4).

$$Ec = 4700\sqrt{f'c} = 20\,270 \text{ MPa} \quad (3)$$

$$n = Ec / Em = 20270 / 5835 = 3.47 \quad (4)$$

4.1 Properties of the transformed section

In the elastic range before the cracks develop, the initial lateral stiffness and the tension by flexure can be obtained in the confined masonry walls using the transformed section criteria. The concrete tie-column area was replaced by an equivalent masonry area, using an enlarged width of the columns by the factor n=3.47, and keeping the length constant. The transformed cross section properties were calculated as follow:

- A = 2 (3.47) (130 mm) (200 mm) + (2 200 mm) (160 mm) = 532 648 mm^2 (axial area)
- Ac = 2 (130 mm) (200 mm) + (2 200 mm) (160 mm) = 404 000 mm^2 (shear area)
- f = A / Ac = 1.32 (shape factor)
- I = 403 x 10^9 mm^4 (centroidal moment of inertia)

4.2 Initial lateral stiffness (K_0)

The model of the confined masonry wall subjected to lateral force resembles a cantilever beam with flexural and shear deformations [9]. Eq. (5) was used to calculate the lateral stiffness in the elastic range K_0, with wall height h=2290 mm and using the previous properties of the cross section and elastic modulus of masonry.

$$Ko = \frac{Em}{\frac{h^3}{3I} + \frac{(f)(h)(Em)}{(Gm)(A)}} \quad (5)$$

After replacing values, the initial stiffness was found as K_0=162 kN/mm.

4.3 Lateral force to produce cracking due to tension by flexure (F)

The lateral force resistance F, associated with the appearance of the first cracks due to tension by flexure at the base of the walls can be determined by equating the cracking tension of the concrete to the normal stress produced by the combination of the vertical load Pver, and moment F h. If the tensile strength of concrete is 0.67 √f'c [8], and the self weight of the wall is neglected, then the lateral force can be obtained using Eq. (6).

$$F = \frac{\left[\frac{0.67\sqrt{f'c}}{n} + \frac{Pver}{A}\right](I)}{h\ y} \tag{6}$$

The known values are n=3.47; f'c=18.6 MPa; h=2290 mm; A = 532648 mm²; I = 403 $x10^9$ mm^4; Pver = 108 kN for WV-1; Pver = 157 kN for WV-2; y = L/2 = 1300 mm (distance from the centroid to the farther fiber in tension). After replacing in Eq. (6), the force was found as F=140 kN and F=153 kN for walls WV-1 and WV-2.

4.4 Lateral force to produce diagonal shear cracking (Vm)

The lateral force that produces the diagonal shear cracking was determined using Eq. (1), and the following forces were obtained: Vm=209 kN for WV-1 and Vm=220 kN for WV-2.

4.5 Lateral force to produce steel yielding in flexure (Vf)

The lateral force that produces the yielding of the column bars in tension was calculated using Eq. (7), in which the self weight of the wall was neglected.

$$Vf = (As)\ (fy)\ (d)\ /\ h + (Pver)\ (L)\ /\ (2h) \tag{7}$$

The known values in Eq. (7) are: As = 508 mm² (4-12.7 mm diameter); fy = 412 MPa; h = 2290 mm; d = 0,80 L = 2080 mm (effective depth); Pver = 108 kN for WV-1; Pver = 157 kN for WV-2. Replacing values, the lateral forces Vf =251 kN for WV-1 and Vf = 279 kN for WV-2 were obtained.

4.6 Expected failure mechanism

Given the calculated values of the forces F, Vf and Vm, it can be observed that Vf is larger than Vm in both walls. Therefore, the expected failure mechanism is by diagonal shear cracking. Also, prior to such failure, the lower F values indicate that both walls should show tension cracks due to flexure. These kinds of failures are often seen in confined masonry walls subjected to intense earthquakes.

5. Construction of walls WV-1 and WV-2

The two confined masonry walls were built in the traditional way, similar to the previous walls [6]. Afterwards, the welded wire mesh was added on both sides and covered with a cement-sand mortar. For both walls the same materials, same workmanship, and same kind of reinforcement in the confinement were used. The horizontal and vertical joints were 15 mm thick, using cement:sand mortar in a volume proportion of 1:4. The brick units were wetted for half an hour approximately 10 hours prior to their placement. The connection of the reinforced concrete tie-columns to the masonry wall was toothed. To prevent the concrete of the columns to enter the horizontal holes of the border bricks, pieces of paper were used to fill such holes. The masonry wall was constructed in two days, half of the height each day. After that, the column's formwork was set and the concrete was poured. The concrete used in the confinement elements had a resistance of 18.6 MPa, according to the samples taken during the construction. No holes were found in the tie-columns after the construction. Finally, the concrete of the tie-beam was poured and the specimens were let to dry for 28 days. Figure 6 illustrates construction process.

Fig. 6 – Construction of the masonry walls (left), tie-columns (center) and tie-beam (right)

In the already built walls, the welded wire mesh was placed on both sides (Fig. 7, left). Some bricks were perforated with a 6 mm diameter steel bar and a hammer, every 450 mm, to allow a connecting wire to cross the masonry wall (Fig. 7, center). These connecting wires were bent in a 90° hook and tied to the wire mesh with smaller ties. Later, the perforations were filled with a cement grout in a proportion of cement and sand of 1:3. Finally, both sides of the wall were covered with cement-sand mortar in a proportion of 1:4 (Fig. 7, right).

Fig. 7 – Placement of the welded wire mesh (left), connecting wires (center), mortar cover (right)

6. Testing with vertical and cyclic lateral loads

The topmost horizontal displacement, D1, was used as control, measured with an LVDT. First, the vertical load Pver, was applied (Pver = 108 kN for WV-1 and Pver = 157 kN for WV-2), with a speed of 20 kN/min. At this time, vertical displacements were recorded for each wall. Then, the lateral cyclic load was applied in several steps keeping constant the vertical load. Table 1 summarizes the number of steps and cycles within each step.

Table 1 – Steps of Cyclic Load Test

Step	**1**	**2**	**3**	**4**	**5**	**6**
Number of cycles	1	2	3	3	3	3
D1 (mm)	1.0	2.5	5.0	10.0	15.0	20.0
Drift (%)	0.04	0.11	0.22	0.43	0.65	0.87

The set of instruments for displacements recording used in the walls is shown in Figure 8: D1 is the top horizontal displacement, D2 is the vertical displacement in the central part of the wall, D3 and D4 are the displacements at the diagonal central part of the wall, D5 and D6 are the vertical displacements at the bottom of the columns, D7 and D8 are the horizontal displacements at the connection between the concrete columns and the masonry wall.

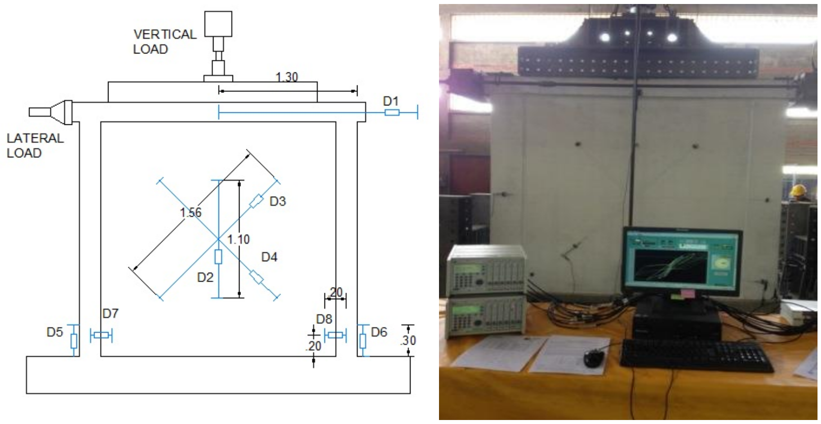

Fig. 8 – Instrument in the walls (left) and recording equipment (right)

Vertical load of 108 kN was applied to wall WV-1 and no cracks developed. In step 2, for a lateral load of 167 kN, the crack at the bottom of the column had a width of 0.05 mm. In steps 3 and 4, the cracks developed toward the wall center, having widths of 0.5 mm. The diagonal crack at the wall center appeared in step 5 with a maximum lateral load of 365 kN and a crack width of 0.7 mm. Finally, in step 6 the cover of the concrete columns spalled and cracked, and a shear friction failure occurred characterized by sliding at the wall base (Fig. 10, left).

Vertical load of 157 kN was applied to wall WV-2 without any cracks. In step 2, for a lateral load of 182 kN the crack at the bottom of the column had a width of 0.05 mm. In steps 3 and 4, the cracks developed toward the wall center. The diagonal crack at the wall center appeared in step 5 with a maximum lateral load of 414 kN and a crack width of 1.3 mm, and crushing of the concrete at the column base took place (Fig. 10, right). Finally, in step 6 the concrete columns base continued to crush until the shear friction failure occurred with sliding displacements of 15 mm at the wall base at the end of the test (Fig. 11).

Fig. 9 – Wall WV-1 (left) and wall WV-2 (right), step 5.

Fig. 10 – Crushing of the bottom of the columns for wall WV-1 (left) and wall WV-2 (right).

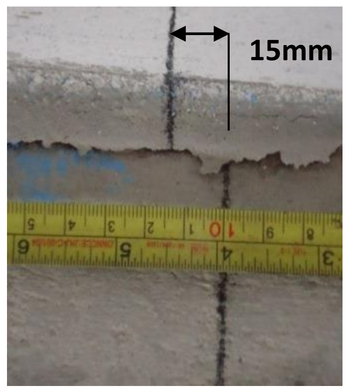

Fig. 11 – Sliding failure for wall WV-2

7. Cyclic test results for masonry walls

Figure 12 shows the hysteretic load-displacement relationship for walls WV-1 (left) and WV-2 (right). The lateral stiffness decay may be observed at the higher displacements. In both walls the load capacity is strongly reduced for D1 of 15 mm, the reason is the crushing of the concrete at the bottom of the column.

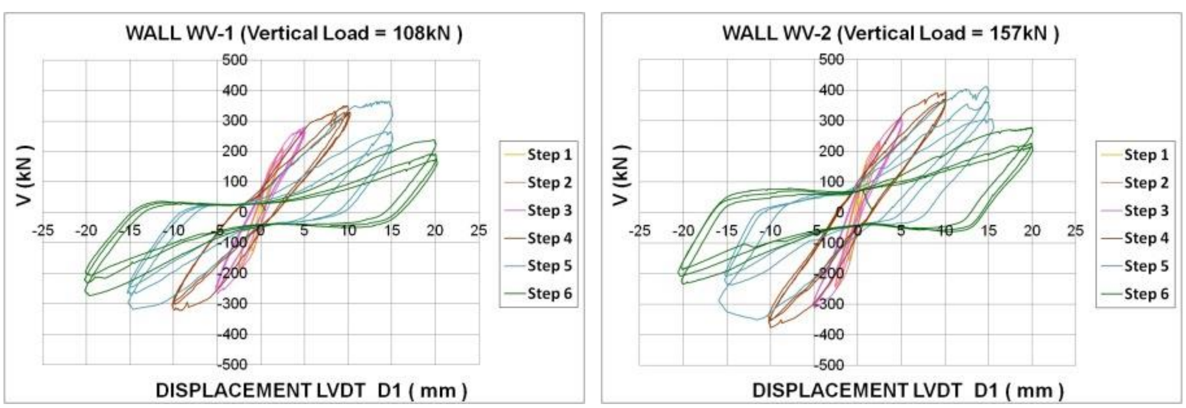

Fig. 12 – Lateral load-displacement relationships for walls WV-1 and WV-2

The Peruvian seismic Code [10] establishes a maximum allowable drift of 0.005 for masonry structures. In the cyclic load tests, this drift is reached in the first cycle of step 5 with a lateral displacement of 11.9 mm. After that, the load capacity still increased a little until the drift reached 0.0058. Then, the concrete crushing started and the lateral stiffness dropped significantly. Comparison of the backbone curves for reinforced walls with vertical loads WV-1 and WV-2 with the traditional wall W1 is shown in Figure 13. The load capacity for drifts larger than the 0.005 drift value prescribed by the code, is significantly higher in the reinforced walls than in the traditional wall in which the stiffness decreased earlier for drifts of 0.0023.

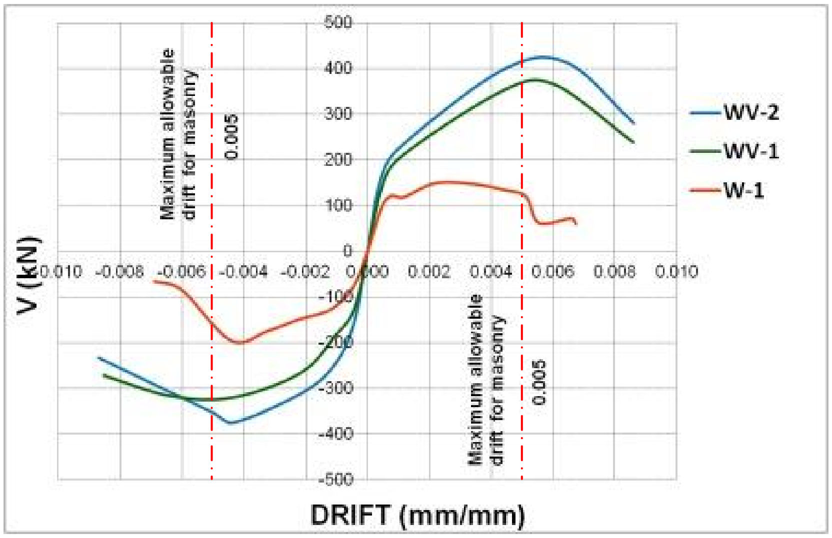

Fig. 13 – Comparison of traditional and reinforced walls with Code drift.

The initial stiffness K was determined using the shear force V and top displacement D1 in the first cycle of step 1, with elastic behavior for both walls. The experimental stiffness values for walls WV-1 and WV-2 were 173 kN/m, and 214 kN/m, respectively. In the previous test of traditional wall W1 the initial stiffness was only 108 kN/m, and for the reinforced wall W2 it was 152 kN/m. The larger values in the walls WV-1 and WV-2 are attributed to the effect of vertical load, with an increase of 14% and 40% more with respect to wall W2.

The cracks due to tension by flexure appeared in step 2 of the tests. For wall WV-1 the load that produced the first tension crack was 167 kN; this is 19% higher than the theoretical value which was 140 kN. For wall WV-2 the load at the first crack was 181 kN.

The diagonal shear crack appeared in step 5 of the test. For wall WV-1 the load that produced the first diagonal crack was 321 kN, very different from the theoretical value which was 209 kN. Similarly, for wall WV-2 the load that produced the diagonal crack was 414 kN, that is significantly larger the theoretical value which was 220 kN. It is observed that Code Equation (1) gives lower values, as it was established for masonry walls made of solid units and without any mesh reinforcement. Nevertheless, for the purpose of this research, Eq. (1) is the only way available to find an approximate estimation to the shear cracking load.

The crack width for each step of the tests was found by LVDT´s records of D3 and D4. In step 5 the maximum crack width was 0.7 mm for wall WV-1 with maximum lateral load of 321 kN, and 1.3 mm for wall WV-2 with maximum lateral load of 414 kN (Figure 14).

Referring to the maximum lateral load attained, in the reinforced walls with vertical load WV-1 and WV-2, the maximum load was larger than the one obtained for previously tested walls. Comparing to wall W1 without reinforcement and without vertical load, the increase of load capacity was 86% for WV-1 and 110% for WV-2. Figure 15 shows the load-displacement envelope for the four walls. The load capacity increases with vertical load, and the drop in the lateral load occurs for drifts larger than the Code value of 0.005, which corresponds to a lateral displacement of 12.5 mm.

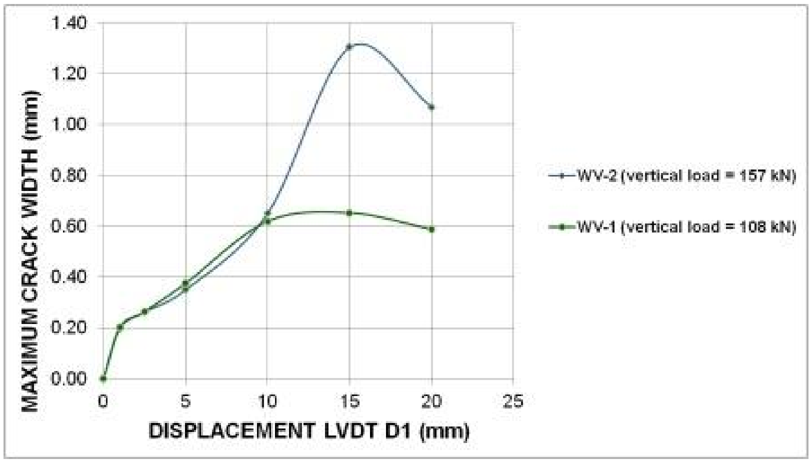

Fig. 14 – Crack width variation for tested walls

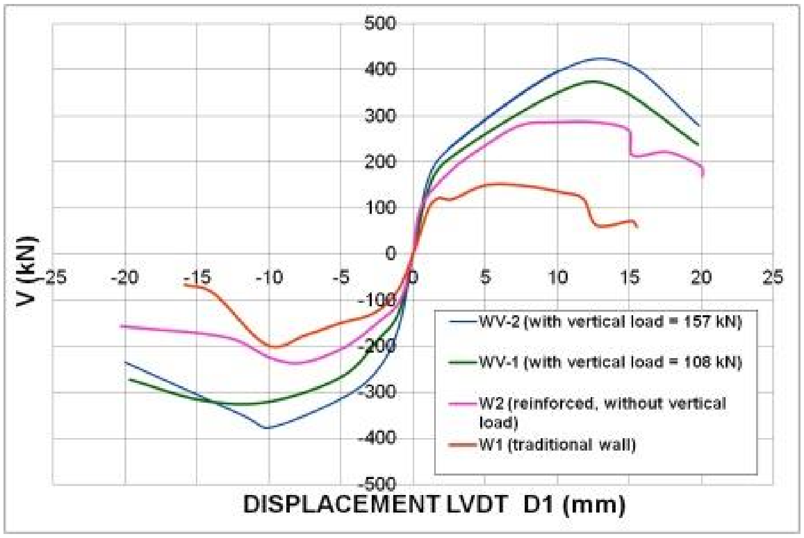

Fig. 15 – Envelope of lateral load-displacements for the four walls

8. Conclusions

The number of tests performed in this research is limited to arrive to numeric conclusions. However, it was observed from the tests results that the reinforcement provided by the welded steel mesh is an effective approach for increasing the lateral load resistance, compared to a traditional wall made of horizontally-hollow bricks. The welded mesh in the reinforced walls W2, WV-1 and WV-2 was able to control the shear cracks in the masonry walls. The stresses increased in the tie-columns, until they failed by crushing, and afterwards the sliding failure occurred in the walls. The results are promising, showing a practical way of how to reinforce masonry walls that otherwise are brittle and weak due to the misuse of horizontally-hollow bricks. Such masonry walls are common in Peru, so this approach for reinforcing can reduce their seismic vulnerability.

9. Acknowledgements

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