Seismic Reinforcement of Confined Masonry Walls made with Hollow Bricks using Wire Meshes

A. San Bartolomé, D. Quiun, K. Barr & C. Pineda
Pontifical Catholic University of Peru

SUMMARY:
Hollow bricks with more than 30% of holes in the bed area are forbidden in confined masonry walls according to the E.070 Peruvian Masonry Code, due to their poor seismic behavior, with brittle failures. However, due to economic reasons, these bricks are extensively used in Peru and other countries for bearing masonry walls. Previous experimental research has shown that the addition of horizontal steel reinforcement to masonry walls had a partial effectiveness in controlling the failure. This paper describes the experimental tests performed on hollow-brick confined masonry walls with the addition of wire meshes covered with mortar, which exhibited an effective improvement in their seismic behavior. Such reinforcement could also be applied to retrofit existing buildings with hollow-brick masonry walls before earthquakes occur.

Keywords: masonry, hollow bricks, confinements, wire mesh, seismic reinforcement

1. INTRODUCTION

The Peruvian Masonry Code (Norma E.070, SENCICO 2006) specifies that solid bricks should be used for confined bearing walls. Hollow units (more than 30% holes in the bed area) are forbidden in such walls because, under earthquake loads, they crush due to the diagonal cracks that open and close continuously, and cause a severe loss of resistance and rigidity of the walls (Fig. 1). This happens even for drifts below the limit specified by the Peruvian Seismic Code (Norma E.030, SENCICO 2003). However, many people use hollow bricks because they are cheaper than solid bricks, and thereby their buildings are vulnerable to earthquakes. It is necessary then to find ways to retrofit such buildings, especially when they have a low wall density, to prevent wall failures caused by diagonal shear cracking during severe earthquakes.

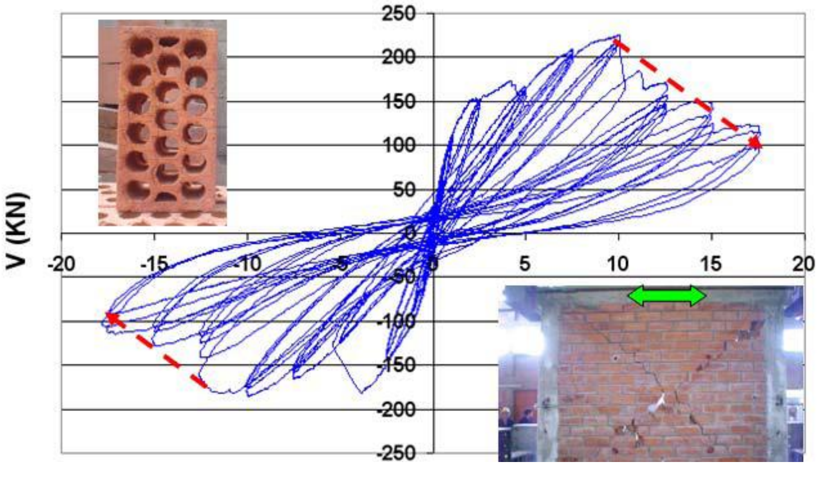

Figure 1. Poor behaviour of a previous test of masonry walls with hollow bricks (San Bartolome, et.al. 2009)

In a previous project (San Bartolome et.al., 2009), the mentioned problem was somehow controlled by the use of horizontal reinforcement within the wall. However, crushing of the hollow bricks still continued. This paper describes a research done using wire meshes covered with cement mortar over the masonry wall. Two full scale confined masonry walls were constructed using the same labor, the same materials and the same dimensions and reinforcement, walls W1 and W2, with the only difference that wall W2 had wire meshes on both sides. The walls were tested under cyclic lateral load with controlled displacement, simulating seismic loads, and satisfactory results were obtained.

2. MATERIAL PROPERTIES

The material properties used for the construction of the masonry walls W1 (normal) and W2 (with wire mesh) are as follows.

Industrially made clay bricks with 90x125x230 mm overall dimensions were used. In the bed area the bricks have 46% of holes. The compressive strength using the gross area was f'b=23 MPa. The dimensional variation was less than 1%. Warping was only 0.7 mm. The absorption was 11% and the initial suction was about three times larger than the Code specification. Therefore, the bricks were treated by watering them with a hose for half an hour, about 10 hours prior to construction. According to the Peruvian Masonry Code, the unit classified as "hollow" class V (highest).

The mortar was measured by volume, using a 1:4 cement-coarse sand proportion. The amount of water was added by the mason in order to get a workable mixture. The horizontal and vertical joints of the masonry walls had 15 mm width. Type I Portland cement was used for the mortar joints and the concrete confining elements.

The strength of the concrete used for the confinement elements was f'c =17.5 MPa. The steel reinforcement was ASTM A615 grade 60 with yield stress of 420 MPa. The wire mesh used in wall W2 had 4.5mm diameter welded deformed bars, spaced every 150mm, with strength of 611 MPa.

Three masonry prisms (125x230x600 mm) were tested under axial compression, giving an axial resistance of masonry f'm =7.8 MPa. The failure was brittle with some crushed bricks (Fig. 2). Also, small walls (125x600x600 mm) were tested under diagonal compression, providing a shear resistance v'm = 0.72 MPa. The failure was brittle, mostly along the joints and cutting or crushing some of the hollow bricks (Fig. 2).

Figure 2. Hollow brick (left) and typical failure of prisms (centre) and small walls (right).

3. WALL CHARACTERISTICS AND REINFORCEMENT

The confined masonry walls W1 and W2 (Fig. 3) had the same geometric characteristics, the same type of materials and reinforcement in the confinements, the same workmanship, and the same construction procedure. The only difference was that for wall W2, in each side a wire mesh was installed over the masonry. The wire meshes had no connection to the foundation nor to the confinements, because its purpose was the control of the diagonal cracks produced by the seismic lateral loads, and by extension, control the crushing of the hollow bricks. The meshes were connected to the walls with smaller #8 wires, every 450 mm. They were covered with cement:sand mortar 1:4 in volume, 30mm thickness on each side, giving the wall a total thickness of 185 mm.

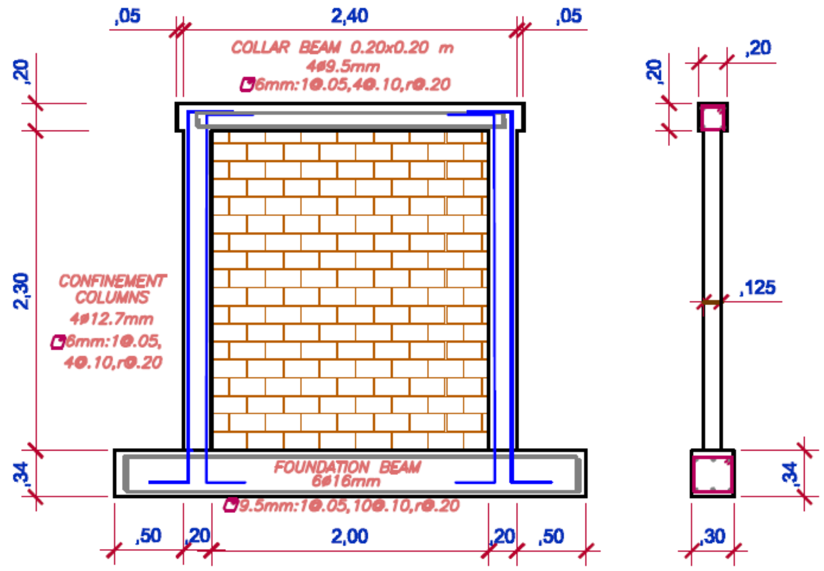

Figure 3. Wall W1 overall dimensions in meters.

3.1 Confinements design

According to the Peruvian Masonry Code, masonry walls should have a shear failure under a severe earthquake, and eqn.1 gives the shear wall capacity for diagonal cracking, Vm. The confinement elements have to be designed to resist this load.

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

Slenderness Coefficient α = V L / M = V L/ V h = 1 for square walls,
v´m = 0.72 MPa (see diagonal compression tests in paragraph 2),
t = 125 mm is the wall thickness,
L = 2.4 m is the wall total length,
P = 0 , as no axial load was applied in the tests.

Using eqn. 1, a value of Vm =106 kN was obtained. This led to 125x200 mm confinement concrete columns reinforced with 4-12.7 mm bars, and 6mm ties spaced 1 at 50mm, 4 at 100mm and the rest at 200 mm. The collar beam was square section 200x200 mm reinforced with 4-9.5 mm bars, and 6 mm ties spaced 1 at 50mm, 4 at 100mm and the rest at 200 mm.

3.2 Design of the wire mesh

The shear capacity of the reinforced wall W2 was estimated using the same eqn. 1, with a wider 185 mm thickness, increased due to the 2x30=60 mm cover which was necessary to properly connect the wire mesh reinforcement to the masonry wall. The calculation using eqn. 1 leads to Vm (W2) = 160 kN. However, this value corresponds to the simple masonry shear resistance v'm, and in a future research the mortar-covered mesh reinforced masonry shear should be used.

The required reinforcement is given by eqn. 2:

A_s (required) = Vm s / fy L (2)

Where s = spacing of the bars, fy = yield stress of the bars, L =2.4 m.

Calculation by eqn. 2 gives A_s (required) = 24 mm^2, and the provided mesh of 4.5mm diameter in both sides gives A_s (provided) = 32 mm^2.

4. WALL W1 LOAD CAPACITY PROPERTIES AND FAILURE LOAD PREDICTION

The elastic shear and flexural lateral load capacity of the confined masonry wall was evaluated theoretically using the transformed section criteria. Then, the maximum lateral load capacity in case of shear failure and flexural failure was also evaluated.

4.1. Elastic Properties and Transformed Section

The elastic and shear modulus of the masonry was obtained using the masonry Code expressions Em=500 f'm, and Gm=0.4 Em. The calculations give values of Em=3.9 GPa and Gm=1.56 GPa. The concrete modulus was calculated to be 20 GPa. Then, the elastic modulus ratio was n=Ec / Em = 5.15, and was used to obtain the transformed section properties in the elastic range: Axial area A=0.506 m^2, Shear area Ao = 0.3m^2 and Inertia I=0.394 m^4.

4.2. Elastic Initial Stiffness (Ko)

Wall W1 was treated as a cantilever with a height of 2.4m in order to obtain the elastic lateral displacement due to a unit lateral load. Then the elastic initial stiffness was calculated as the inverse of such displacement with a value Ko=120 kN/mm.

4.3. Elastic Flexural Load Capacity (F)

The elastic load capacity F was the load required to produce the flexural tension of the concrete elements at the base of the wall. A value of concrete limit tension of 2.6 MPa was used and a value of lateral load F=69 kN was obtained.

4.3. Maximum Shear Load Capacity (Vm) and Maximum Flexural Capacity (Vf)

The maximum shear load capacity was already calculated to be Vm=106 kN using the Masonry Code expression given in eqn.1. The maximum flexural capacity is called Vf, and can be obtained assuming the yield of the reinforcing bars in the concrete columns and using eqn. 3.

M (base) = Vf h = As fy d (3)

As is the sum of areas of the four bars (12.7 mm diameter) in one column, which yields 4x129=516 mm^2; fy is the steel yield stress taken as 420 MPa, and d is the distance between column axes. Replacing values in eqn. 3 gives a flexural load capacity of Vf = 199 kN.

4.4 Expected Failure

Given the load capacity values of Vf= 199 kN (in flexure) and Vm=108 kN (in shear), a shear failure is expected in wall W1. The tension failure in the concrete columns by flexural moment should occur before, with a load of 69 kN.

Wall W2, reinforced with wire mesh covered with cement-sand mortar has the same flexural capacity Vf as wall W1, because the vertical bars of the mesh were not connected to the foundation. The shear capacity for wall W2 is larger than W1 because W2 is thicker. Moreover, the mortar used to cover the wire meshes may increase the shear resistance v'm. The conditions mentioned above indicate that the type of failure which wall W2 could have is uncertain.

5. WALL CONSTRUCTION AND REINFORCEMENT OF WALL W2.

The construction of both walls was done using similar materials, the same workmanship and similar reinforcing bars in the confinement elements. Other characteristics were that the brick layout was set using the smaller unit thickness as wall thickness, and toothed connections were used between the wall and the columns, as it is usual in Peru. In order to reduce their natural initial suction, the clay bricks were watered with a hose for 30 minutes about 10 hours prior to use. This procedure is recommended by the Peruvian Masonry Code to achieve an optimal suction in clay bricks at the time of placement. The confinement concrete cover was 30 mm. The concrete was prepared in a mixing machine, had a 100 mm slump and was compacted with a vibrator. The masonry wall was built in two days, followed by the pouring of the concrete of the columns. Finally, the collar beam was poured. Figure 4 displays part of the masonry wall construction.

Figure 4. Construction of masonry walls W1 and W2.

If the masonry wall were a new construction, the wire connectors of the meshes could be positioned in the mortar joints during the brick layout. However, in this case an existing wall with need of reinforcement was simulated. Therefore, after construction of wall W2, a series of perforations were performed within the brick units every 450mm (three times the spacing of the mesh wires). The wire meshes were placed on both sides of wall W2, but only over the masonry and not over the concrete columns or the collar beam (Fig. 5, left). Then, small connecting wires #8 were installed through these perforations, which were bent 90° and tied to the mesh nodes (Fig. 5, centre). Afterwards, the perforations were filled up with a liquid mortar of cement-sand with 1:3 parts in volume. Finally, both sides of wall W2 were covered with a mortar of cement-sand with 1:4 parts in volume (Fig. 5 right).

Figure 5. Wall W2: wire mesh (left), connector (centre), and final cover (right).

6. DESCRIPTION OF THE CYCLIC LATERAL LOAD TESTS.

6.1 Steps of the test and instrumentation

Both walls were subjected to an experimental cyclic lateral load test where the topmost horizontal displacement was controlled. No vertical load was applied. A 500 kN load capacity actuator and 6 LVDT displacement transducers were used, in the positions shown in Figure 6. The purpose of the transducers is as follows: D1 controls the lateral amplitude of displacements; D2 lets us measure the diagonal cracking accumulated on the central part of the wall; D3 and D4 let us measure the cracks that appear on the base of the columns; finally, D5 and D6 measure the separation between the columns and the masonry wall if any appears.

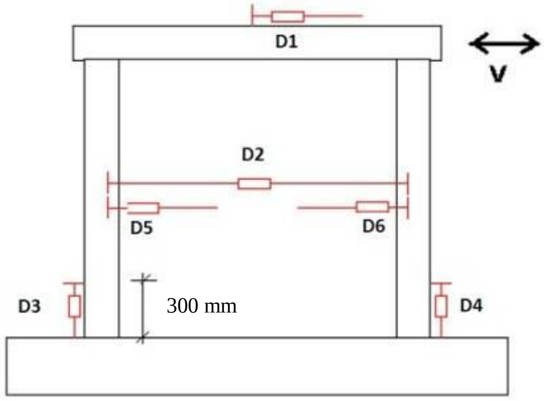

Figure 6. Test setup and instruments on the walls

Table 1 gives the steps used in the experiment for both walls. The number of cycles per step varied in order to get a stable hysteretic loop V-D1. The test speed was about one cycle every 4 minutes, enough to record at least 100 points of V, D1 in each cycle.

Table 1. Steps of the Cyclic Lateral Load Test

STEP	1	2	3	4	5	6	7	8	9	10
D1 (mm)	0.5	1.0	2.5	5.0	7.5	10.0	12.5	15.0	17.5	20.0
Cycles in W1	1	2	3	3	3	3	3	4	3	4
Cycles in W2	1	1	2	2	2	2	3	3	3	3

After the 10 steps were completed, both walls were subjected to a harmonic load with displacement amplitude of 15mm and a 2 Hz frequency. The purpose of this harmonic step was to observe quickly the predominant failure type of the walls and their degradation, which is hard to see in slow tests.

6.2 Behavior of the walls

As it was previously predicted, tension flexural cracks appeared at the column bases in wall W1. Also as it was predicted earlier, wall W1 had a shear failure with diagonal cracks and crushing of the hollow bricks (Figure 7). This brittle behaviour is common for this kind of walls (San Bartolome et. al., 2011) which is why the Peruvian Masonry Code forbids the use of hollow bricks in structural masonry walls.

Wall W2 had a flexural failure, with quite thin shear cracks in the masonry controlled by the wire mesh reinforcement. The final failure was a vertical crack in one of the columns, losing the bond between steel bars and concrete, while the other column’s base was crushed (Figure 8).

When the harmonic load was applied to wall W1, the bricks crushed a lot, even though the lateral displacement amplitude was 15 mm, less than 20 mm, which was the last step amplitude of the cyclic load test. In the case of wall W2 the crushed column base ended completely broken. Figure 7 shows Wall W1 and Figure 8 shows wall W2 at the end of the harmonic tests.

Figure 7. Both sides of Wall W1 at the end of the Test

Figure 8. Wall W2 with detail of the column failures (bond and crushing)

7. RESULTS AND COMPARISON OF THE WALL TESTS

7.1 Hysteretic Loops and Envelopes

In figure 9, the loop hysteretic plots of Lateral Load (V) – Horizontal displacement (D1) are shown for both walls. These thin curves, which pass through the origin of the axes, are typical of systems of degrading stiffness and are very similar to each other, despite the fact that the walls had different failures (shear in W1 and flexure in W2).

However, it can be observed in wall W1 that for D1=4 mm, a sharp drop in resistance occurs due to the shear failure. Such drop is not present in wall W2, attributable to the wire mesh that controlled the diagonal cracking. It may also be seen that after the maximum resistance is reached, there is a continuous degradation in resistance.

The envelopes of shear-displacement V-D1 (Figure 10) are useful for comparing the behaviour of both walls. To obtain the envelope curves, the positive branch corresponding to the first cycle of each step was considered because the largest resistance degradation occurred there. Wall W2 exhibits larger stiffness and shear resistance than wall W1, despite the fact that at the end of the test both show similar degrading resistance. However, wall W2 had a width of 185 mm, larger than wall W1 width of 125 mm, so the unit shear stress was calculated dividing the force V between the wall rectangular area, as shown in Figure 11. It can be observed that the shear resistance is practically the same for both walls.

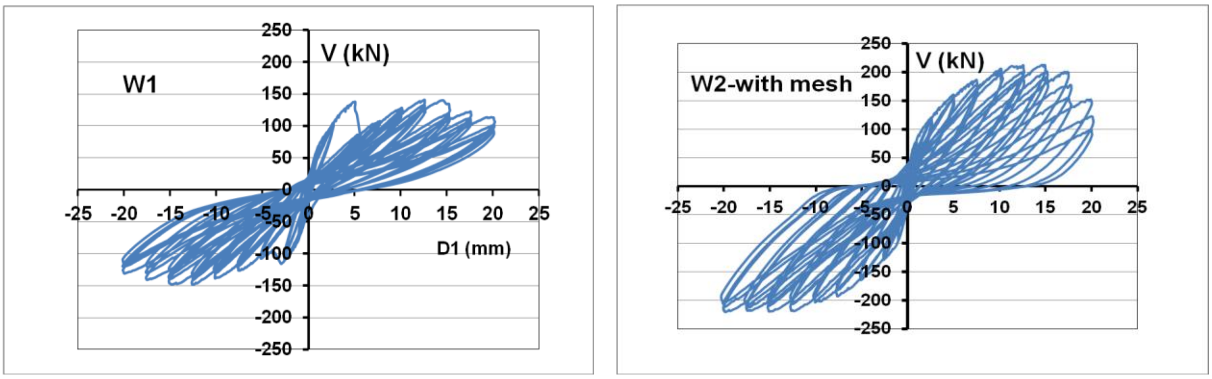

Figure 9. Load-displacement hysteretic loops for W1 (left) and W2 (right)

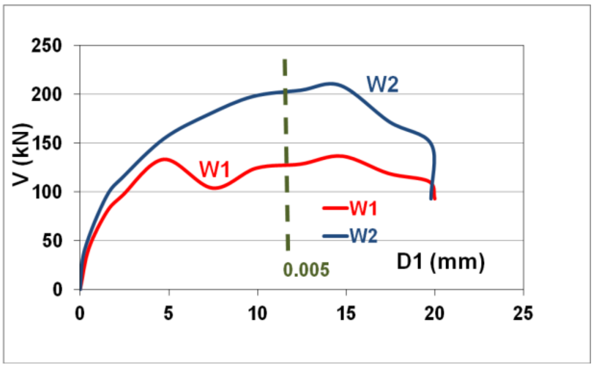

Figure 10. Load-displacement envelope.

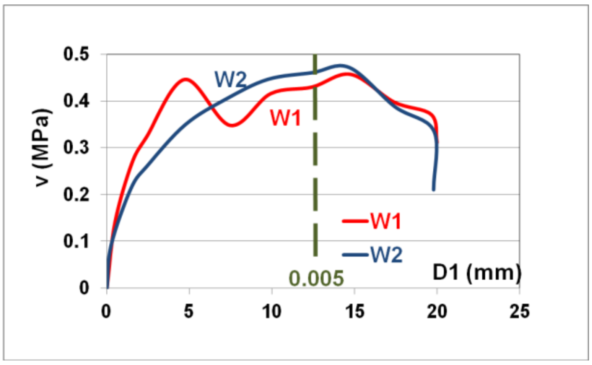

Figure 11. Shear stress-displacement envelope.

7.2 Initial Lateral Stiffness

In step 1 before the first tension flexural cracks appear in W1, the lateral stiffness (Ko) was obtained experimentally to be 98 kN/mm, while the theoretical value was 120 kN/mm, 23% larger.

For W2, the initial lateral stiffness obtained experimentally was 162 kN/mm, which is 65% larger than the corresponding Ko of W1. This is explained because the increased width of W2 respect to W1, and that W2 had a mortar covering.

7.3 Resistance to Tension by Flexure

In W1 the first tension crack by flexure in the columns occurred during step 1 of the test, for a load of 64 kN. It may be recalled that the theoretical associated value was 69 kN, only 8% larger. For W2, such crack appeared in step 3 of the cyclic test, with a load of 113 kN, which is 77% larger than the corresponding load in W1. The difference is due to the larger width of W2 respect to W1.

7.4 Resistance to Shear Diagonal Cracking

The diagonal cracking for W1 occurred in step 4 of the cyclic test with a load of 123 kN, while the theoretical prediction was 106 kN, that is 16% less. For W2, in step 5 very thin diagonal cracks appeared for a load of 159 kN, very similar to the theoretical resistance of Vm (W2) of 160 kN using eqn. 1 and the 185 mm width (see 3.2). This result confirms that to evaluate the shear capacity of a masonry wall with a mortar cover with mesh reinforcement, the total width should be used.

7.5 Maximum Resistance

The maximum resistance for both walls was reached in step 7 of the cyclic test, 145 kN for W1 and

215 kN for W2. The maximum load obtained in W2 is 48% higher, but in terms of unit shear stress both walls have similar values, as shown in Figure 11: v (W1) = 145000 / (125x2400) = 0.48 MPa; and v (W2) = 215000/(185x2400) = 0.48 MPa.

7.6 Maximum Drifts

The Peruvian Seismic Code (Sencico 2003) establishes that the maximum allowable drift for masonry is 0.005, so that the walls could be repaired after a severe earthquake. Such drift corresponds to step 7 of the cyclic test (D1=12.5 mm). In this slow test wall W1 resistance degraded significantly in step 8. However, this does not mean that hollow bricks can be allowed for structural walls, because when the harmonic test with 15 mm amplitude was performed, , the brick crushing was very notorious.

7.7 Crack's Width

The most important cracks were selected as follows. Wall W1 had a diagonal shear failure and wall W2 had a vertical crack in the boundary between column and wall due to flexure. These cracks' widths appear in Figure 12, where it can be seen that the maximum crack width for wall W2 was much less than the one for wall W1.

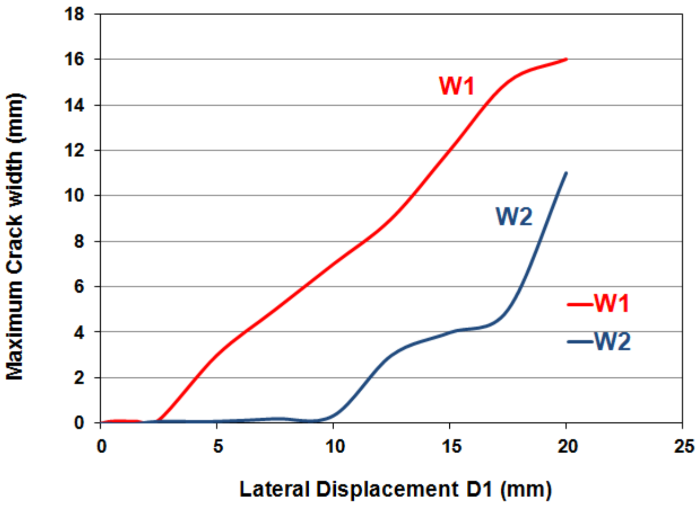

Figure 12. Maximum crack width.

8. CONCLUSIONS

The conclusions given here are limited because only few walls were tested. However, they indicate that a substantial improvement was achieved when the hollow-brick wall W2 was reinforced with the wire mesh.

The bricks employed in this project had good quality (class V, highest according to the Peruvian Masonry Code) but had 46% of holes in the bed area, and therefore are not allowed for use in confined structural walls, unless such walls have an elastic behaviour for a severe earthquake. The test on the small specimens showed crushing of the hollow bricks, as well as in confined wall W1 after the diagonal cracking with a severe degradation in shear force resistance. Such crushing and deterioration was greater in the harmonic test, and would have been even more if wall W1 had vertical load.

The design of wall W1 followed the Masonry Code (except for the use of hollow bricks). The theoretical prediction of the initial lateral stiffness, the load which causes the first tension by flexure and the load for diagonal shear cracking, were very close to the experimental values (22%, 8% and 16% difference, respectively).

The extra reinforcement used in wall W2 (welded wire mesh covered with mortar), increased the effective thickness of the wall in 48% (185 mm against 125 mm). This fact increased the shear resistance and modified the final to failure, from a shear failure in wall W1 to a flexural failure in W2. Increases in the initial lateral stiffness, the load that causes the first tension by flexure, the load at diagonal shear cracking, and the maximum load resistance were 65%, 77%, 50% and 48%, respectively, in wall W2 over W1. However, the unit shear resistance of both walls was very similar.

The maximum load resistance of wall W2 could be predicted with less than 10% of error.

The diagonal cracks that appeared in wall W2 (despite its flexural failure), were very thin, as they were controlled by the wire mesh. This also avoided the crushing of the hollow bricks, fulfilling the main objective of this project. In the previous research (San Bartolome et.al. 2009), masonry walls which only had the addition of horizontal reinforcement, that problem was barely attenuated.

In both walls the lateral load capacity degraded for drifts of 0.006, larger than the maximum allowed by the Seismic code which is 0.005. In wall W1 with shear failure, the degradation was caused by the crushing of the hollow bricks, while in wall W2 with flexural failure, the degradation was due to a vertical crack in the column with loss of bond to the reinforcing bars. In W1 the failure is quite dangerous because the crushed masonry could not resist vertical loads. Suggestions to control the observed failure in W2, can be studied in a new experimental research project, where the wire mesh extends over the confinement columns, or the horizontal wires of the mesh are welded to the vertical bars of the columns.

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