Effective Seismic Retrofit Technique of a Previously Tested Confined Masonry Wall Made with Perforated Bricks

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ABSTRACT

A large number of informal housing buildings in Perú are built using perforated clay bricks or concrete blocks for confined masonry structural walls. The Masonry Code specifies that structural walls must be constructed with solid bricks (less than 30% of holes in the bed area). However, the lack of supervision, the desire to minimize costs, the general lack of seismic conscience of the owners, among other factors, explain the use of inadequate perforated bricks (more than 45% of holes in the bed area) for bearing walls of buildings up to 5 stories. The brittle failure of the masonry made with such units make them vulnerable under the combined action of gravity and seismic loads.

Several experimental projects have been conducted to study reinforcing techniques to enhance the strength of these structures. In this project a confined masonry wall with an axial load equivalent to two stories, was subjected to a cyclic lateral load until shear failure occurred. Then, the wall was repaired and reinforced with jacketing of welded wire mesh on both surfaces. The retrofitted wall was tested under the same conditions as the original one, with improved behavior, a flexural failure, and an increase of lateral load of 40%. The retrofitting technique proved to be efficient and could be used in real buildings to increase and enhance their structural safety.

KEYWORDS

Confined masonry, hollow bricks, cyclic load test, retrofitting, wire mesh.

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INTRODUCTION

In Peru and other Latin-American countries, a significant portion of low and medium size buildings are built using bearing masonry walls. Particularly, confined masonry walls are used in seismic zones, but in Peru a large amount use perforated hollow bricks and are often constructed, without an adequate technical supervision. In such cases, the structural vulnerability of those masonry buildings under seismic events, is increased beyond the Code specifications. The Peruvian National Census of 2017 indicated more than 55% of all housing buildings have masonry walls of bricks or blocks, this percentage has risen significantly in the last 25 years [1]. Many buildings are constructed using low quality materials, by self-construction, or by masons unskilled in seismic concepts with thick joints. The absence of control is motivated by economic reasons (cheap materials and workmanship) and authorities with poor capabilities in structural concepts.

The strong Pisco earthquake of magnitude Mw 8.0, hit the Ica region in Peru, on August 15, 2007. A series of deficiencies in masonry constructions became evidenced in numerous structural failures in masonry walls, such as low-quality materials, foundations without the required dimensions, construction errors (irregular bricks layout, wide mortar joints, inadequate wall braces), inefficient structural configurations (short columns, torsion irregularities beyond limits), and many out-of-plane wall failures (San Bartolomé & Quiun, 2008). A sample of such failures may be observed in Figure 1 [2].

Figure 1: Masonry failures and wall collapses, Pisco 2007 [2]

The Peruvian Masonry Code of 2006 [3], released just one year before the Pisco earthquake, specifies that hollow bricks cannot be used in bearing walls in the seismic areas, due to their fragile failure under high axial stress, due to vertical and seismic loads. Also, the Code defines a hollow brick as that in which the solid bed area is less than 70%, or that the holes in the bed area exceed 30%, while the previous 1982 Code had 75-25% values. However, many masonry walls used as structural bearing walls are made with bricks having 40-50% of holes in the bed area. This is caused by the cheaper value of the hollow bricks, the lack of conscience of the people, and absence of control and supervision by the local authorities.

Regarding the vulnerability of those building structures, some proposals for the repair and retrofit of confined masonry walls with hollow bricks (40-50% holes) have been published, based in experimental cyclic-load tests carried out. For example, San Bartolomé et al. (2008) showed that the use of welded wire meshes, added to walls damaged by cyclic load tests, were effective to recover and increase the shear resistance of the walls, and also, the deformation capacity was improved [4]. Similarly, the addition of such meshes to one masonry wall before being tested, showed an important improvement in the seismic behavior, compared to another conventional wall without reinforcement (San Bartolomé et al., 2012) [5]. Another test (San Bartolomé et. Al. 2009) included one wall with horizontal reinforcement, of 6.3 mm diameter bars every two courses in the mortar bed (ratio of 0.00128%). Figure 2 show how that type of reinforcement could extend the brick's crushing up to a lateral drift of 0.006, while for the conventional wall, the lateral load capacity had decreased 25% for a lateral drift of 0.004, with extensive crushing of the hollow bricks [6]. The previous last test performed with the hollow bricks by Pari & Manchego (2017), using 10 mm joints, also demonstrated that the repaired and reinforced walls can recover significantly their shear load capacity, after the damage caused by the cyclic load tests [7]. Retrofitting seismic vulnerable masonry walls made with hollow bricks, is a promising way to reduce the damage in upcoming earthquakes.

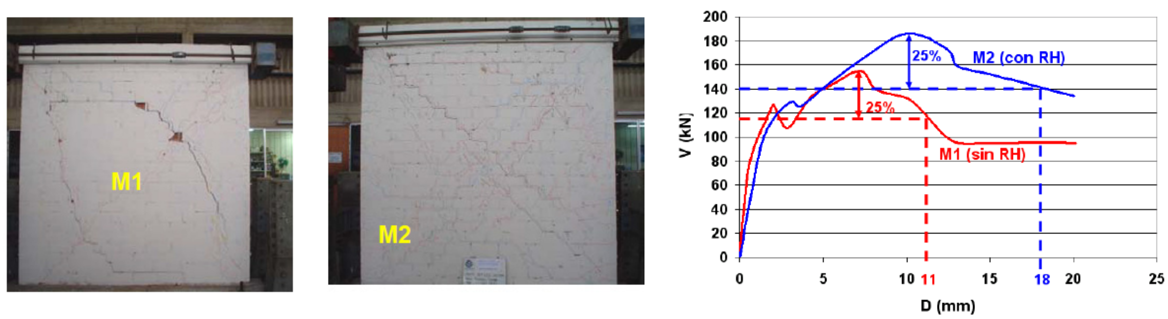


Figure 2: Masonry walls with hollow bricks M1 (conventional) and M2 (with horizontal reinforcement) at a drift of 0.005, and lateral load-displacement envelope [6].

Another brick type which is often used incorrectly for confined masonry walls in Peru, is the horizontally-hollow brick. These units were conceived for use in non-structural walls, and are not permitted in seismic regions per the Peruvian Masonry Code. They are only permitted for 1-2 story homes in regions of low seismicity. For such cases, cyclic load tests have been performed in full-scale confined walls, published by San Bartolomé et. Al (2013) [8], Quiun and Mamani (2017) [9], and Quiun and Diaz (2024) [10]. In all cases, the masonry walls reinforced with welded wire mesh, had increased lateral load capacity, control of cracks in the masonry panels, and good inelastic deformation and ductility. These walls did not fail at low levels of lateral drift like the walls constructed of only horizontally-hollow bricks.

Within this context, the present paper has the objective to analyze the structural behavior of a masonry wall made of hollow bricks, subjected to both constant axial load and cyclic lateral load, with the novelty that the masonry has a 15 mm mortar joint, usual in bearing wall constructions. Previously, few experiments have been done using hollow bricks, only one test had also both axial and lateral load [7], but the masonry used had 10 mm joints, which are not as usual.

The experimental study begins with the behavior of the conventional wall under test, the process of repair and reinforcing with welded wire mesh, and finally, the test under similar loads as the original wall. By this experience, we intend to contribute with a practical methodology to enhance the structural safety and behavior of many housing buildings, having bearing confined masonry walls, built with inadequate hollow bricks in regions with high seismic activity.

MATERIAL PROPERTIES

Perforated hollow clay brick

The brick units used in this research are locally called King Kong industrial, with 18 holes, with dimensions 230 mm length, 125 mm thick and 90 mm height (Figure 3a). The variation of the dimensions is only 0.28% (less than 1% Code limit), the surface of laying has a warping of 1.19 mm maximum, and the compressive strength is 20.5 MPa; all three properties are within what the Peruvian Masonry Code considers the best quality unit. However, the area of holes in the bed area reaches 47%, which exceeds the 30% Code limitation to be considered as a solid brick; therefore, it is a perforated hollow brick, not allowed in bearing masonry walls in the Peruvian seismic zones 4, 3, and 2. It is only allowed for a maximum of 2 stories in the seismic zone 1 (Z=0.1g) which is the lowest seismic active (the Amazonic region).

Concrete

The concrete was used for the foundation beam where the wall was built (41 MPa), for the confining columns (21 MPa), and the top beam (21 MPa). The minimum strength required for the concrete is 17.6 MPa, according to the Peruvian Masonry Code.

Joint Mortar

The mortar in the joints had a mixture of 1:4 parts of cement and coarse sand. This proportion is commonly used in masonry constructions. The compressive strength was obtained at 17 and 60-days age, with a compressive strength between 25-27.5 MPa.

Welded wire mesh

After the test of the original wall, it was repaired and reinforced with a welded wire mesh at both wall surfaces. The material complies with ASTM A1064/A1064M-18a[11]. The wires were 6 mm diameter with spacing every 150 mm in two perpendicular directions.

Masonry prisms

The masonry mechanical properties were measured in small specimens. Four prisms of overall dimensions 230x125x615 mm were built, each had 6 courses, with 15 mm mortar thickness. The axial compression test gave an axial strength of f'_m=8.94 MPa and also, the elastic modulus was E_m=5 272 MPa. Also, four small walls of overall dimensions 615x615x125 mm were built and tested under diagonal compression. The shear strength was obtained as v'_m=1.11 MPa and also, the shear modulus was G_m=1265 MPa. One of the tested prisms is shown in Figure 3b, while one of the tested small walls is shown in Figure 3c.

Figure 3: a) Clay brick "King Kong" with 18 holes (47% of the bed area); b) prism under axial compression; c) small wall under diagonal compression

Theoretical Analysis and Construction of the Wall

Theoretical analysis

Using the geometry of the wall (Figure 4) and the masonry properties, the size of the confining elements and the reinforcement bars is given as follows. The confining columns are each 125x200 mm with 4-12.7 mm bars, stirrups of 6 mm diameter. The upper beam has a section 200x200 mm, with 4-9.5 mm bars and also, stirrups of 6 mm diameter. Additionally, a set of U-shaped hooks are included at both ends, where the lateral load is to be applied, to enhance the stability and reduce local deformations. The foundation beam is 300x340 mm, with 6-15.9 mm bars and stirrups of 9.5 mm.

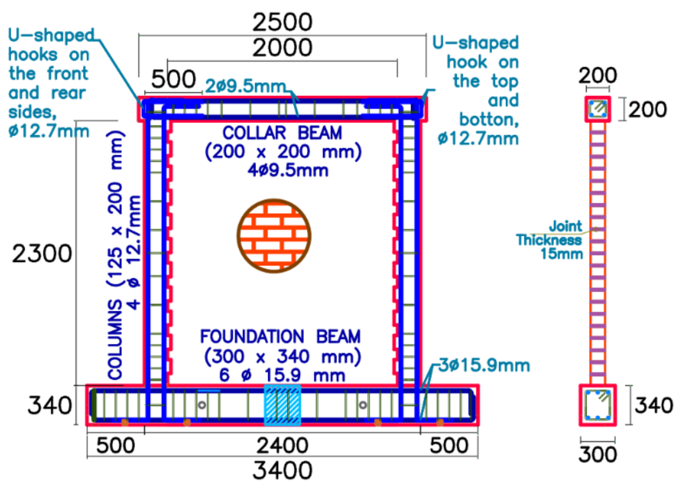


Figure 4: Confined wall for vertical and cyclic lateral load test

Lateral stiffness

The elastic behavior of the confined wall requires the use of the transformed section criteria, so the concrete of the columns was transformed into equivalent masonry using the ratio of the elastic modulus. The lateral stiffness (K) was then obtained using Eq. (1), considering the wall in cantilever, and the flexural and shear deformations. The masonry elastic modules were Em = 5 272 MPa and Gm = 1 265 MPa. The height was taken as h=2.4 m, giving a theoretical lateral stiffness K = 111 kN/m.

$$(1)\ K = \frac{E_m}{\frac{h^3}{3 \cdot I_{equi}} + (f \cdot h \cdot \frac{E_m}{G_m \cdot A})}$$

Tension by flexure cracking in the concrete of the columns

The end of the theoretical elastic behavior can be calculated by the moment of tension cracking of the concrete in the columns, due to the combined effects of compression axial load (Pver = 110 kN) and bending moment caused by the lateral load F [9]. The tensile stress of the concrete is a function of *f'c*=21 MPa, and the ratio of the elastic modules is $n=E_c/E_m = 4$. Also, the distance from the neutral axis to the border of the columns is x'=1.2 m. The other geometric characteristics are the transformed area A'=0.45 m^2 and I_{equi}=0.326 m^4. In Eq. (2) the lateral load F=108 kN could be obtained.

$$(2)\ F = \frac{\left(\frac{0.62\sqrt{f'c}}{n} + \frac{P_{ver}}{A'}\right) \cdot I_{equi}}{h \cdot x'}$$

Lateral load resistance due to flexure and shear failure

The flexure failure and the corresponding lateral load capacity V_f was defined as the load that produces the yielding of the reinforcing bars of the columns in tension, using Eq. (3). In it, the area of the vertical bars of one end column is As=508 mm2, the steel yielding tension is fy=420 MPa, the effective depth of the wall is taken as d=0.8 L=1920 mm, and the wall height is h=2.4 m. Substituting these values in Eq. (3) gives us a capacity due to flexural failure of 171 kN.

$$(3)\ V_f = \frac{A_S \cdot f_y \cdot d}{h}$$

The shear capacity of the confined wall V_m is determined by the expression of the Peruvian Masonry Code (Norma E.070 Albañilería), Eq. (4). In it, the shear stress is v'm = 1.11 MPa, the thickness of the masonry wall is t=125 mm, the wall length including the confining columns is L=2400 mm, the slenderness reduction factor is α=1, and the vertical load to be applied constantly is Pg=110 kN. Substituting these values in Eq. (4) gives us a capacity due to diagonal shear failure of 192 kN.

$$(4)\ V_m = 0.5 \cdot v'm \cdot \alpha \cdot t \cdot L + 0.23 \cdot P_g$$

Expected wall failure

From the two above calculations, the shear capacity (V_m=192 kN) is larger than the flexural capacity (V_f=171 kN). Therefore, the theoretical failure should be by flexure, and previously, for a lateral load F=108 kN, the tension cracks in the concrete of the columns should develop.

Reinforcement effect previous to the second test

The original wall test will enter into the inelastic range, producing cracks without collapse. Then, the wall will be repaired and reinforced using a welded wire mesh over both surfaces. The additional contribution to shear resistance provided by the mesh is calculated using Eq. (5), with steel at yield. In this equation, the steel area is As=28 mm^2 for wires with spacing s=150 mm, the yield stress is fy=420 MPa, the total length is L_s = 2000 mm, the. After calculating, one mesh resists 157 kN. The mortar cover of the meshes is effective for corrosion protection and long-term performance.

$$(5)\ V_{m,mesh} = \frac{A_S f_y L_S}{S}$$

Construction of the confined masonry wall

The original wall was named as M1-CA, its dimensions are 2300 mm height, 2000 mm length and 125 mm thickness. The mortar used was cement-sand 1:4 with 15 mm joint thickness. Figure 5 shows the construction process, with the masonry panel built before the concrete of the columns, and the top beam.

a) Masonry and toothed connection; b) Concrete into columns; c) Upper beam

Figure 5: Construction of the masonry wall

CYCLIC LOAD TEST FOR ORIGINAL WALL M1-CA

The cyclic load test for the original wall was carried out by increasing top horizontal displacements (called D1), in 7 steps. The maximum displacement was 12 mm, for a drift of 0.5%, this is the limit set by the code E.070 for which the wall can be repaired. Table 1 indicates some details of the steps.

Table 1: Steps of the cyclic load test for wall M1-CA

Step	1	2	3	4	5	6	7
Drift (percentage)	0.02	0.04	0.10	0.21	0.31	0.42	0.50
Displacement (mm)	0.5	1	2.5	5	7.5	10	12
Number of cycles	1	2	3	3	3	3	3

A total of seven sensors LVDT were located in the wall, to measure displacements. D1 records the top horizontal displacements, D2 records the relative displacement between the columns, D3 and D4 are used to see the shear deformation of the wall, D5 records the vertical displacements in the middle. D6 and D7 are used to record the vertical deformation of the column's bases. Figure 6 shows the location of the sensors and the overall setup prior to the test.

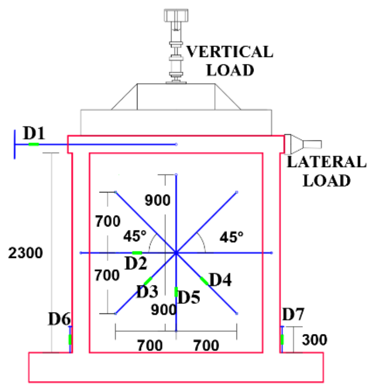


Figure 6: Setup of the test for original wall M1-CA

The wall behavior is summarized by the hysteretic loops, in which the lateral stiffness, degradation and resistance capacity can be observed (Figure 7). The diagonal cracking, or shear failure occurred in step 4, with a drift of 0.2%. The wall at the end of the test exhibits important diagonal cracks.

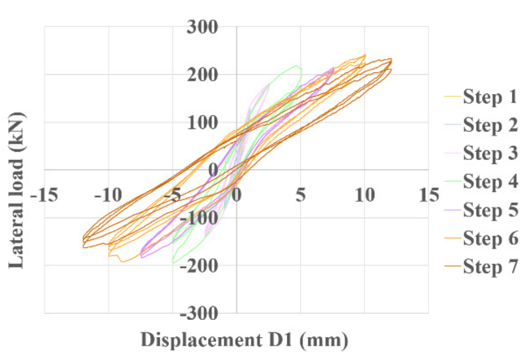
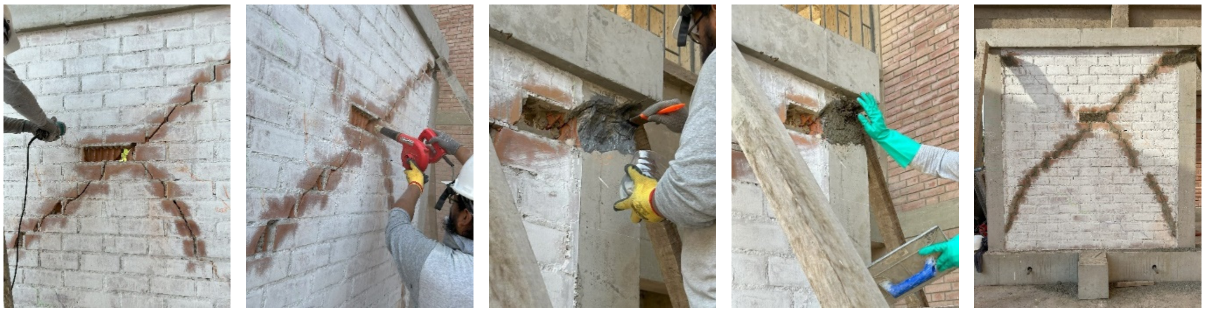


Figure 7: Hysteretic behavior and final stage after test of wall M1-CA

REPAIR PROCESS AND REINFORCEMENT OF THE WALL

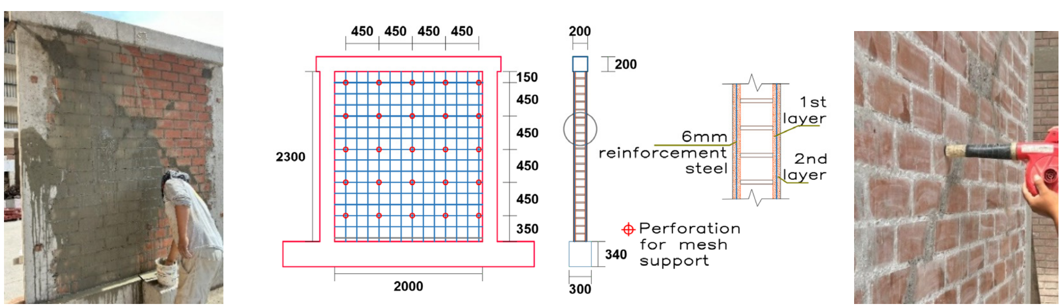
The repair process is illustrated in Fig. 8. In the masonry wall it started by eliminating the loose particles, the cracks that had a width larger than 0.8 mm were widen and cleaned, an epoxy resin was applied to those surfaces. Later, a portion of damaged concrete at the upper part of one column was repaired and all the visible cracks were filled up with a mortar mixture of 1:3 (cement-sand).



a) Widening cracks; b) Cleaning cracks; c) Resin applied; d) Concrete replacement; e) Fill of cracks

Figure 8: Repair process for damaged wall M1-CA

A thin layer of mortar was applied to the wall surfaces (Figure 9a). Then, perforations were done following the scheme of Figure 9b, to insert the connecting bars (4.5 mm) that support the wire mesh at both surfaces, followed by blowing air to clean the perforations (Figure 9c). A thicker layer of mortar cover was then applied (Figure 9d), followed by the installation of the wire meshes (Figure 9e), then the perforations were filled up with 1:3 mortar mixture, and finally, a 10 mm thick 1:4 mortar mixture was applied to the wall surfaces, covering the meshes (Figure 9f). The repaired and reinforced wall is called as M1-RR-CA.



a) Thin layer of mortar; b) Scheme of perforations for connectors; c) Air blown into perforations

d) Gross mortar cover; e) Mesh installation; f) Wall covered with mortar

Figure 9: Reinforcement of wall M1-CA, it becomes wall M1-RR-CA

Cyclic test of the repaired and reinforced wall M1-RR-CA

Similarly to the original wall M1-CA, the repaired and reinforced wall M1-RR-CA was subjected to the same vertical load of 110 kN, and the set of increasing horizontal displacements as cyclic lateral load test, described in Table 2. The instruments for data recording were the same as the first test. This new test had a total of 10 steps with a maximum lateral displacement of 20 mm.

Table 2: Steps for cyclic lateral load test for wall M1-RR-CA

Step	1	2	3	4	5	6	7	8	9	10
Drift %	0.21	0.42	1.04	2.08	3.13	4.17	5.00	6.25	7.29	8.33
Displacement (mm)	0.5	1	2.5	5	7.5	10	12	15	17.5	20
Cycles	1	2	3	3	3	3	3	4	3	4

In Figure 10 the hysteretic curves show the stiffness degradation and resistance capacity. Also, in the center of Figure 10 is the wall at the end of the test. Finally, the wall was subject to a harmonic load in two parts (Figure 10 right), 0.5 Hz with 10 mm of maximum top displacement, and then, to 0.25 Hz with 20 mm maximum top displacement. This last test was important to capture the predominant failure mode, which was a flexure failure.

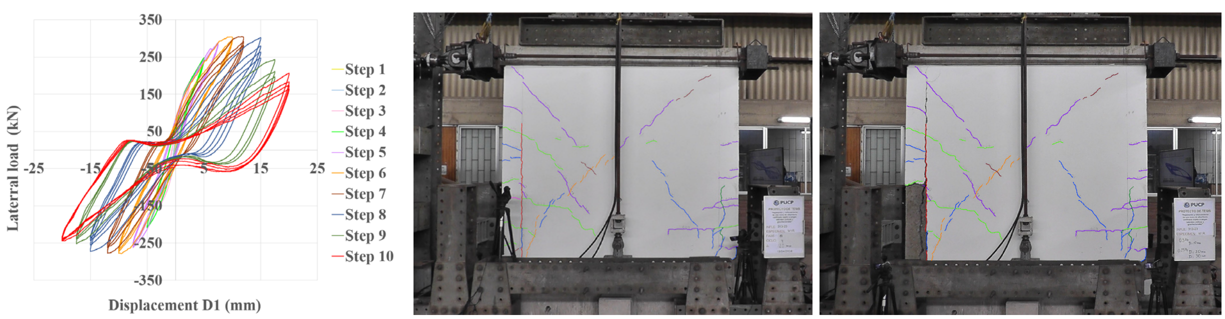
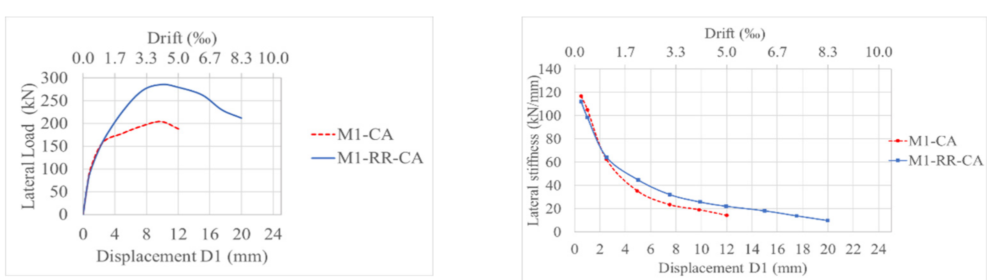


Figure 10: Hysteretic behavior and final stage after test of wall M1-RR-CA

Comparison of walls M1-CA and M1-RR-CA

The original wall M1-CA reached a lateral load of 204 kN, but the test was stopped at 12 mm (0.5% drift), while the reinforced wall M1-RR-CA reached aa load of 286 kN, larger load capacity as well as more ductility and as shown in Figure 11a. Wall M1-RR-CA recovered nearly 96% of its initial lateral stiffness, and was able to undergo larger deformations. In Figure 11b the stiffness degradation for both walls is displayed, and it is clear that the original wall had more degradation.

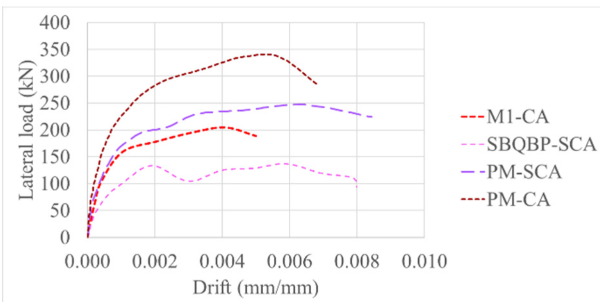


a) Load-displacement envelope; b) Stiffness degradation

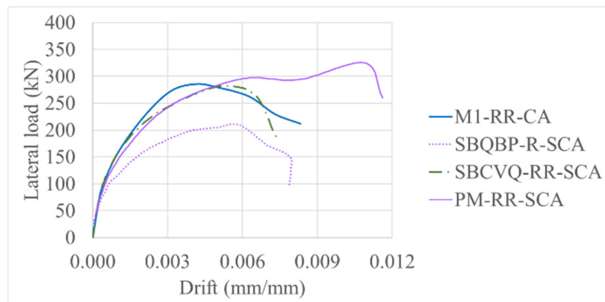
Figure 11: Comparison of the behavior for both walls M1-RR-CA

Comparison with other tests using similar hollow bricks

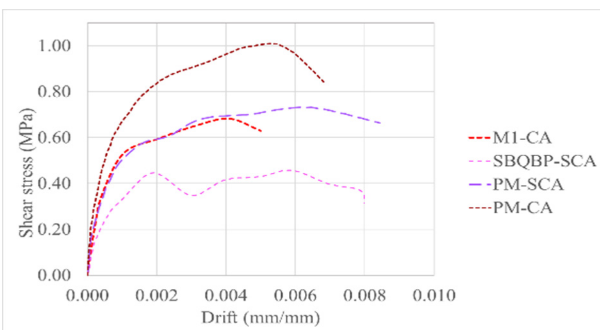
There have been previous researches using the hollow bricks, with 45-47% of holes in the bed area, which are labelled as (SBCVQ) [4], (SBQBP) [5], and (PM) [7]. In this section, a comparison of the main results of these tested walls is done, together with the present case, in which a process of repair and reinforcement was performed. According to the test conditions, it is necessary for identification purposes, that the walls with axial load are named CA, without axial load are named SCA; also, RR indicates a wall repaired and reinforced, while R indicates only reinforced previously to the cyclic lateral load test. In Figure 12, the plots of lateral load vs drift (V-Dr) are given in Figures 12a and b; and the plots of the shear stress vs drift (S-Dr) are given in Figures 12c and d. The application of the wire mesh reinforcement produced significant improvements in the load capacity and the lateral stiffness, with respect to the original walls.



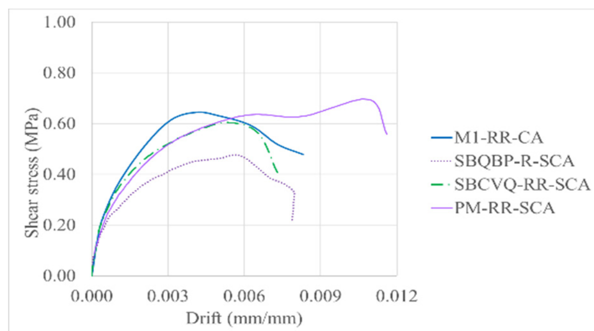
a) V-Dr of original walls



b) V-Dr of repaired or reinforced walls (R) & (RR)



c) S-Dr of original walls



d) S-Dr of repaired or reinforced walls (R) & (RR)

Figure 12: Comparison of envelopes with other researches

In Figure 12a, the walls with axial load PM-CA and M1-CA differ in the mortar joint thickness, 10 mm for PM-CA (340 kN shear load) vs 15 mm for M1-CA (204 kN shear load). Thicker joints are more common in popular housing; the load capacity is about 67% more in wall PM-CA, supposedly due to the thinner joints. The influence of the axial load in providing more shear capacity is noted by comparing M1-CA and SBQBP-SCA (137 kN shear load), and PM-CA with PM-SCA. In Figure 12b, the beneficial effect of the repair and reinforcement process is observed as M1-RR-CA (286 kN shear load) and SBCVQ-RR-SCA (284 kN shear load), have larger load capacity than SBQBP-R-SCA (211 kN shear load). In Figure 12c, original walls are compared, wall M1-CA reaches a shear stress of 0.68 MPa, shows more initial stiffness than PM-CA, however PM-CA was able to have larger displacements but with load decrementing. Finally, in Figure 12d, repaired or reinforced walls are compared, wall M1-RR-CA reaches a shear stress of 0.64 MPa, for large drifts, it has with smaller load decrement than the other reported walls without axial load SCA, despite the fact of that the axial load could influence negatively. Therefore, the reinforcing process in wall M1-RR-CA was successful, providing improvements in stiffness, shear resistance and ductility, which the original wall had lack of. It may be considered an efficient way to repair and reinforce vulnerable confined walls built with perforated bricks (45-47% holes in the bed area).

Conclusions

The reinforcing process with the welded wire mesh has shown to be effective, improving the structural behavior of confined masonry walls built with perforated bricks and thick joints, having axial and cyclic lateral loads. The lateral load capacity increased by 40%, and a recovery of the lateral stiffness despite the axial load. Furthermore, the reinforcement was able to delay the masonry diagonal cracking. Comparison to previous tests with similar walls and bricks, this case showed excellent results. Hopefully, a large-scale application of this kind of reinforcement to masonry walls could reduce significantly the vulnerability of many informal housing buildings, protecting lives and extending the lifetime of the structures.

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