CYCLIC BEHAVIOUR OF TRUSS TYPE REINFORCED CONCRETE MASONRY WALLS

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SUMMARY

Masonry is one of the most antique structural systems in the world. However, it has been losing prestige with the advance of other structural systems such as reinforced concrete and steel. This was to certain extent the result of scarce or even absence of rules, recommendations and design methods available for masonry. On the other hand, masonry has advantages other than good performance as structural system such as durability, fire resistance and thermal and acoustic reasonable behavior As a structural solution, masonry can be used on the construction of buildings inside zones with seismic hazard if steel reinforcement is foreseen (according to EC8 unreinforced masonry is not allowed in zones with moderate to high seismic hazard). Reinforced masonry appears to perform considerably well under seismic loading. It is well known that the use of steel reinforcement in masonry walls allows considerable improvements in their behavior such as the increase of the ductility and the shear strength. The mail goal of this work is thus to obtain insight on the behaviour of reinforced concrete masonry walls under cyclic loads, which is accomplished through an experimental program based on static cyclic tests carried out on reinforced masonry panels with different masonry bond and different reinforcement arrangement. The influence of the horizontal and vertical reinforcement, level of axial loading and bond masonry on the lateral resistance, ductility, stiffness degradation, and failure mechanisms were investigated.

1. INTRODUCTION

Over the years considerable researches has been conducted on masonry structures. Masonry walls were mainly designed to resist gravity loads. However masonry walls have also an important role in improving seismic resistance and global stability of masonry buildings. Due to their role in the masonry buildings, masonry walls can afford significant horizontal loads, induced by earthquakes. Serious damages in masonry walls were observed in some past earthquakes such as the 1931 Hawke's Bay in New Zealand; 1976 Friuli in Italy; 1949 Olympia and 1965 Seattle-Tacoma earthquakes, see Figure 1. This led to the idea that unreinforced masonry walls behave badly under seismic loading, being not allowed in zones with moderate to high seismic hazard. The brittleness of the failure of unreinforced masonry shear walls, which is more remarkable with high axial loads, may be reduced by the use of steel reinforcement. This procedure ensures the increase of the ductility due to redistribution of lateral load, and provides better energy dissipation under seismic loading. The failure mode of a shear wall depends on the combination of applied loads, wall geometry, properties of the materials and as recently pointed out by [1], on the bond masonry. According to [2], when an unreinforced masonry shear wall is subjected to lateral loading a diagonal crack opens producing a severe deterioration in wall strength and a brittle collapse.

The horizontal reinforcement prevents the separation of the wall's cracked parts at shear failure and provides the load transfer between the edges of the cracks [3]. The bed joint reinforcement allows the masonry to carry stresses after initial cracks. The horizontal reinforcement is subjected to tensile stresses and a tendency for its pull-out from the joint because the separation of the two parts of the wall occurs. This mechanism enables the redistribution of lateral loads improving the resistance and energy dissipation capacity of the wall when subjected to repeated reversal lateral loads. Therefore, the walls that are reinforced horizontally present smeared cracking in opposition to the localized shear crack of unreinforced masonry walls. Specimens with horizontal reinforcement uniformally distributed along the length of the wall and specimens with the reinforcement located only at the corners of the wall presented similar behaviour, see [4]. In both cases higher ductility and ultimate load are obtained in relation to unreinforced masonry. However, the placement of reinforcements can increase the complexity of the construction technology of masonry. Note that simple traditional techniques are used to build unreinforced masonry walls. This is particularly evident when vertical reinforcements are to be placed on vertical hollow cells of concrete or brick masonry units.

This work is part of a research program aiming at developing innovative solutions for reinforced masonry walls in the scope of a project (Diswall) financed by European Commission. Different possibilities for the concrete masonry units and arrangement of vertical reinforcements in the vertical joints and in the vertical hollow cells are envisaged. The mechanical validation of the different solutions of reinforced masonry walls is performed by means of a set of static cyclic tests for simulation of the seismic behaviour of the reinforced masonry walls. The main parameters under study consist of the level of axial load and the masonry bond associated to different solutions for the placement of vertical reinforcement.

(a)

(b)

Figure 1: Damages caused by an earthquake in unreinforced masonry walls: (a) 1965 Seattle-Tacoma[5]; (b) 1976 Friuli in Italy [6]

2. EXPERIMENTAL PROGRAM

The experimental program was carried out at Laboratory of Structures of University of Minho (LEST) aiming to evaluate the mechanical behaviour of reinforced masonry walls under lateral load. Five walls were built with different bond masonry and tested under different pre-compression levels. Masonry materials were also characterized, namely units, mortar, reinforcements and masonry as a composite material.

2.1. Properties of materials

Masonry walls were built with three hollow cell concrete blocks, whose shape and geometry are shown in Figure 2. These blocks have a central cell where the reinforcements are positioned. Due to laboratory limitations, half scale concrete blocks were produced. The maximum size of the aggregates had to be reduced to account for the reduced scale of the blocks. Compressive tests were performed on blocks according to [7] and the average values observed for the normalized compressive strength and elastic modulus were 11,4 MPa and 6,8 GPa, respectively.

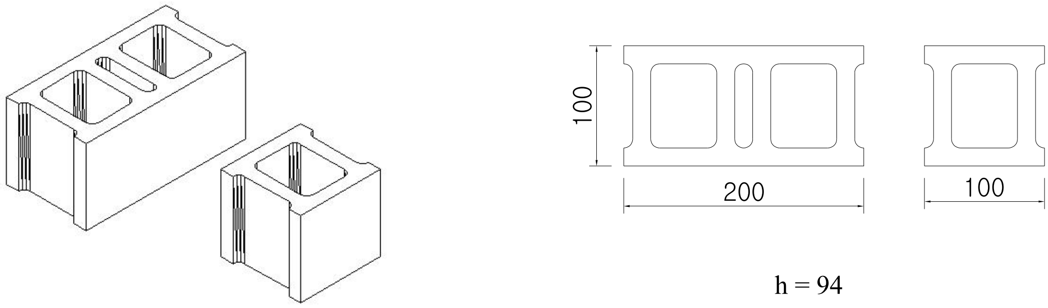


Figure 2: Geometry and shape of the concrete units. (dimensions are in mm)

General purpose mortar corresponding to an admixture with a binder/aggregate ratio of 1:3 (cement:sand) was adopted in the construction of the walls. This mortar was used so that a compressive strength of about 10 MPa could be obtained. According to [8], a M10 should be used when masonry walls are used in seismic areas. It should be stressed that this general purpose mortar was modified by addition of water until an appropriate consistence enabling the filling of the reinforced central hollow cell of the concrete units could be reached. It was provided three prisms of mortar for each constructed wall and compressive tests were performed according to **Error! Reference source not found.**] in these specimens in the day of the respective wall test. Besides, the characterization of masonry as a composite material was carried out by means of uniaxial compressive tests and diagonal tests following [10] and [11], respectively. Average values of 6MPa and 10.5GPa were obtained for the compressive strength and modulus of elasticity in masonry prisms. An average shear strength of 0.4MPa and a transversal elastic modulus of 3.6GPa were obtained in diagonal tests.

Reinforcements used in the construction of the masonry wall panels were of type MURFOR RND/Z, see Figure 3. Reinforcements with 4mm and 5mm diameter and a lateral spacing between longitudinal bars of 80 mm and 50mm were used for the bed joints and vertical hollow cells of the units, respectively. Three samples were submitted to direct tensile tests, being the average value of the yield stress of 580 MPa (ε_y = 4,95 ‰) and the modulus of elasticity about 196 GPa.

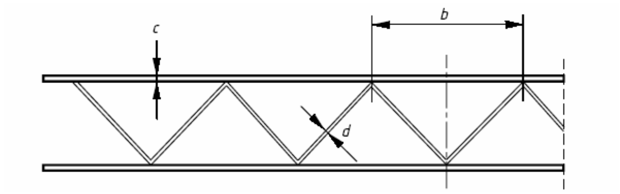


Figure 3: Truss type reinforcements

2.2. Masonry walls

Four reinforced masonry walls were built with two different masonry bonds (B1 and B2). B1 corresponds to running masonry bond (units were overlapped on consecutive courses) see Figure 4a. This masonry bond implies that vertical reinforcements are placed both in the bands of the three cell masonry units and in the internal hollow cell. In the second masonry bond (B2), the vertical reinforcements are placed only in the vertical core defined by the bands of the units, defining a continuous vertical joint, see Figure 4b. The latter masonry bond has advantages concerning the construction technology as the masonry units can be laid after the placing of the reinforcements without any change on the traditional constructive technique applied in the construction of unreinforced masonry walls.

(a) (b)

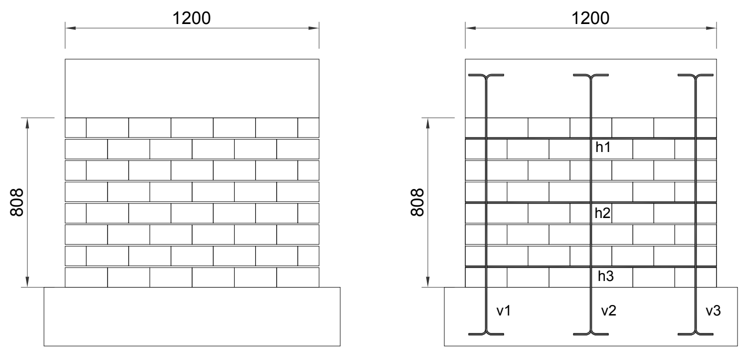
Figure 4: Masonry bonds: (a) running masonry bond, B1 (b) bond with continuous vertical joint, B2

Besides, one unreinforced wall (UM) was built to evaluate the influence of reinforcement. Two different levels of axial force, 1,25MPa and 0,5MPa (N150 and N60), were applied in the masonry specimens. Table 1 shows the details of the tested masonry walls.

Table 1 – Tested masonry walls

Wall	Bond masonry	Vertical reinforcement diameter (mm)	Horizontal reinforcement diameter (mm)	Pre-Compression (MPa)
N60-UM	B1	-	-	0,50
N150-B1	B1	5	4	1,25
N150-B2	B2	5	4	1,25
N60-B1	B1	5	4	0,50
N60-B2	B2	5	4	0,50

The dimensions adopted for the masonry panels were 1200mm width and 808mm height corresponding to a height to length ratio of approximately 0.67, see Figure 5a. These dimensions were selected based on a pre-design study. Reinforced masonry walls have three vertical truss-bars with diameter of 5 mm and three horizontal truss-bars with diameter of 4 mm. Reinforced concrete beams were placed at bottom (280 mm x 280 mm x 1400 mm) and at the top (280 mm x 280 mm x 1200 mm) of the walls in order to anchor the vertical reinforcements, see Figure 5b.



(a) (b)

Figure 5: Masonry walls: (a) geometry, (b) distribution of the reinforcements

2.3. Experimental setup and test procedure

The static cyclic tests of the masonry walls were performed following the typical test setup shown in Figure 6 used for masonry walls under combined vertical and horizontal loading [1]. The bottom reinforced concrete beam of the wall was fixed to a steel profile through eight ties and two adjustable clamping angles to avoid rotation and slip of the base. In turn, the steel profile was connected to the strong floor through a couple of steel rods. The axial load was applied by using a vertical actuator with vertical steel cables anchored at the strong floor. A stiff steel beam was used for the distribution of the vertical load from the actuator and a set of steel rollers was placed to allow relative displacement of the wall with regard to the vertical actuator. The horizontal load is transmitted to the wall by means of two steel plates fixed at the top of the concrete beam connected by four steel ties.

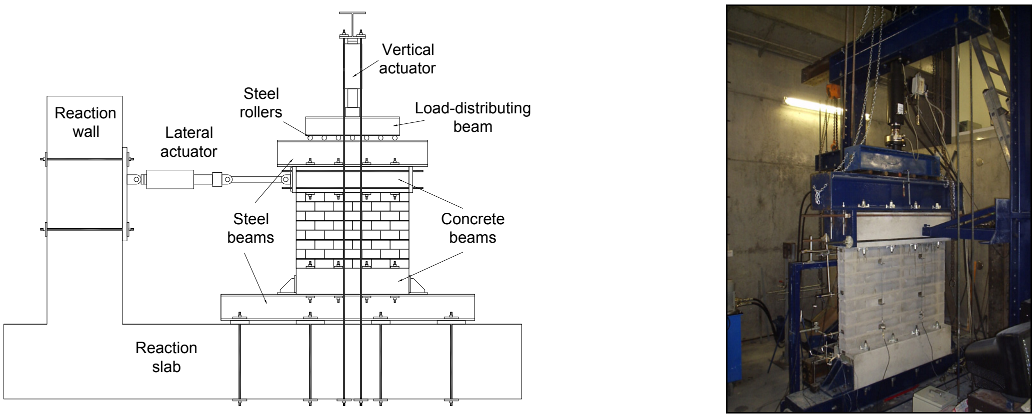


Figure 6: Test setup.

The testing procedure was divided in two phases. Firstly, only a vertical load (100kN) was applied to evaluate the elastic modulus of the wall at a rate of 0,25kN/s. After that, the wall was unloaded and reloaded again up to a vertical stress equal to 1,25MPa or 0,5MPa. After keeping the vertical load constant, the horizontal displacements were imposed to the wall by following the displacement laws indicated in Figure 7. The cyclic tests were thus carried out under displacement control by means of the horizontal LVDT connected to the horizontal actuator at a rate of 70 μm/s. The displacement-time history shown in Figure 7a was applied in the first test. However when the drift reached the plastic level of the wall, the increment of displcement were very wide. Thus, the displacement-time history shown in Figure 7b was adopted in the second specimen but to the unreinforced masonry this displacement-time history had wide increments yet.

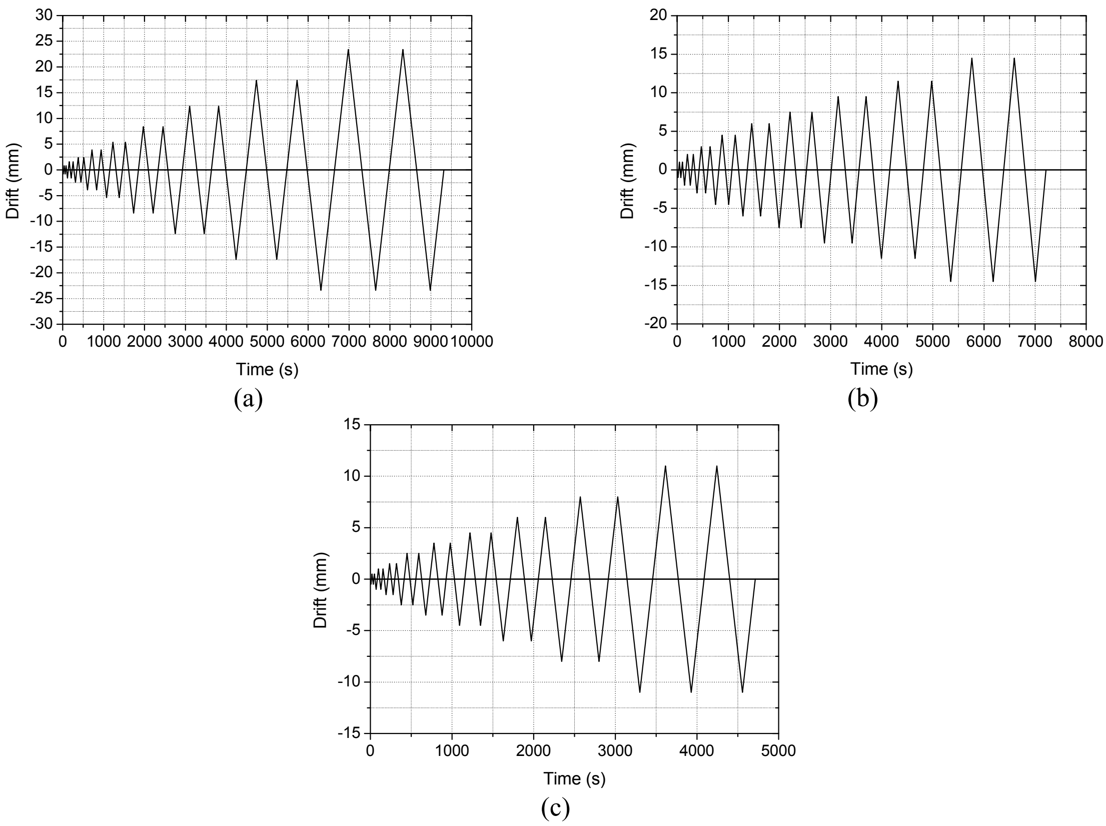


Figure 7: Displacement-time history: (a) N150-B1, (b) N150-B2 and N60-B1 and (c) N60-B2 and N60-UM

The displacements of the wall under cyclic loading were measured by means of a set of LVDTs as indicated in Figure 8a.

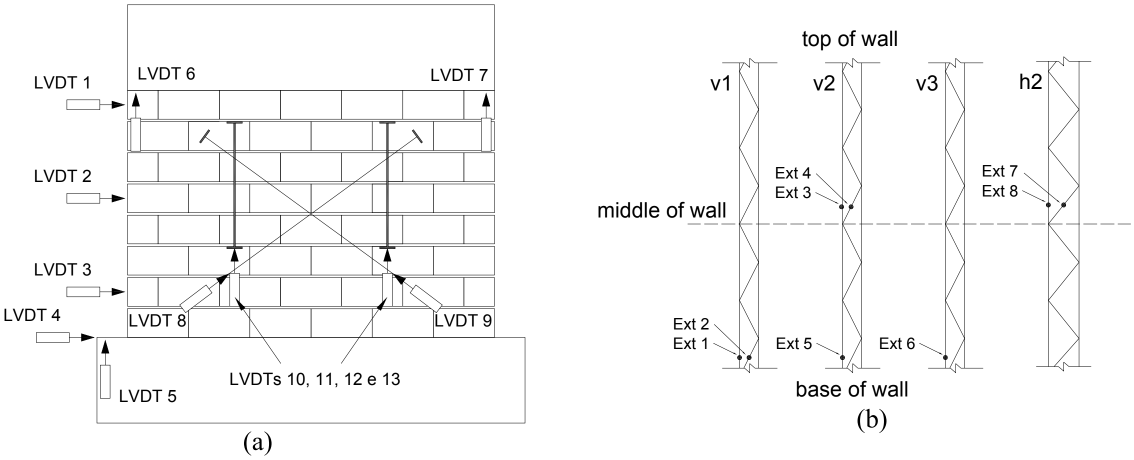


Figure 8: Instrumentation of the wall: (a) LVDTs to measure the displacements of the walls: (b) strain-gauges to measure the deformation of the reinforcements.

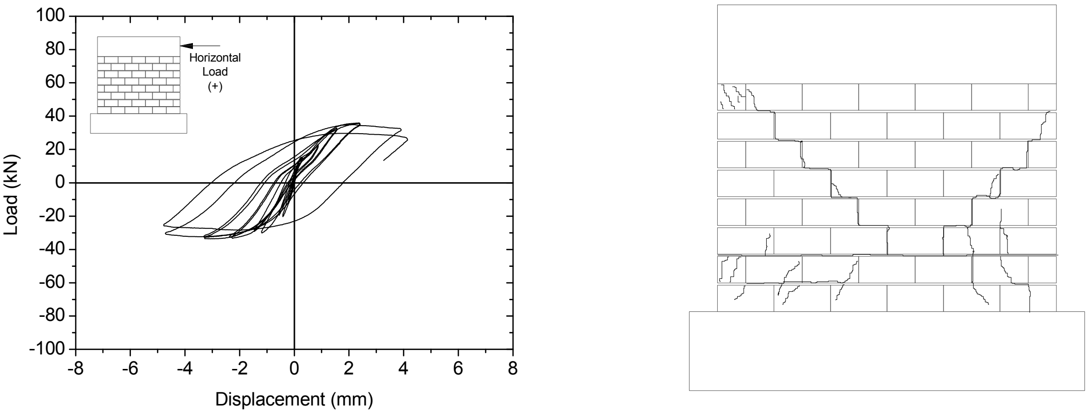
LVDTs 1,2 and 3 measured the lateral deformation of the wall. LVDTs 4 and 5 were placed to measure the slip and rotation of base of the wall, respectively. LVDTs 6 and 7 intend to measure the rotation of the top concrete beam. LVDTs 8 and 9 measured the diagonal crack openings of the wall indicating also possible movements of rigid body. The vertical LVDTs 10, 11, 12 and 13 were fixed to both sides of the wall in order to obtain the elastic modulus of the wall. Besides, strain-gauges were glued to reinforcements to evaluate their contribution to the response of the wall, see Figure 8b. In specimens N150-B1 and N150-B2, strain gauges were also glued in the top and bottom horizontal reinforcements at the same position as the one indicated in Figure 8b.

3. RESULTS

In this section a general overview of the results obtained in the cyclic tests is given to all the specimens. Force-displacement diagrams and the failure modes are some of the aspects under analysis.

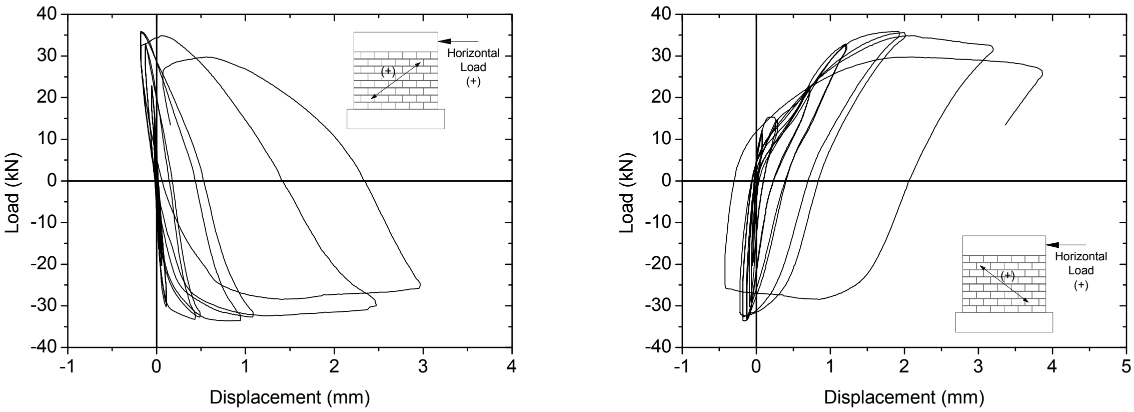
3.1. N60-UM

The wall N60-UM (unreinforced masonry with a pre-compression level of 60kN) reached a maximum lateral resistance of 35kN and exhibited an almost symmetrical hysteretic force-displacement relationship, see Figure 9a. Diagonal cracks, following the pattern indicated in Figure 9b, appeared for a lateral force of 20kN. These cracks are the result of the development of diagonal tensile stresses. The appearance of these cracks leads to a decrease on the lateral stiffness as can be seen in Figure 9a. The progressive increase of the lateral drift led to the continuum cracking of the bed joint between the second and third courses, which resulted in the sliding of the wall along the bed joint. This sliding is clearly revealed by the diagonal displacements indicated in the diagrams of Figure 10. In spite of the sliding, high compressive stresses concentrated at the bottom corners of the wall since several cracks developed in the concrete units, see Figure 9b.



(a) (b)

Figure 9: Behaviour of specimen N60-UM: (a) Diagram Load *vs.* Displacement and (b) cracking pattern

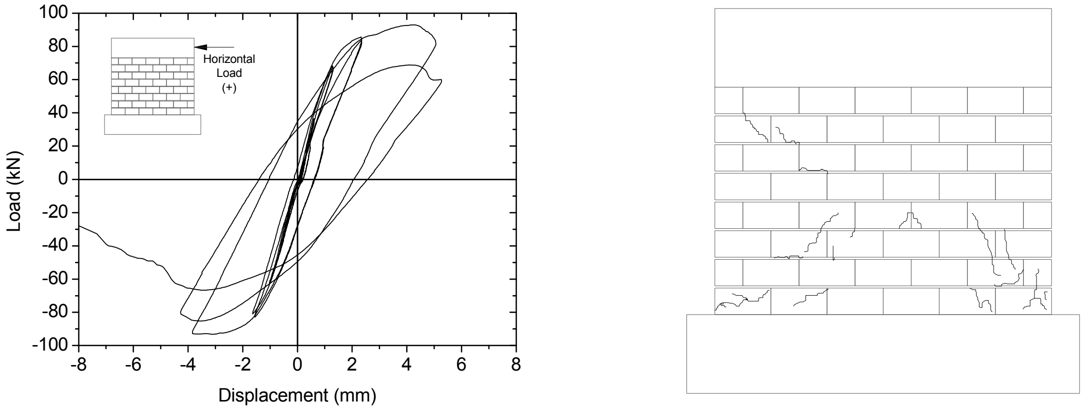


(a) (b)

Figure 10: Diagonal displacements: (a) LVDT 8 and (b) LVDT 9.

3.2. N150-B1

This wall reached a maximum lateral force of 93kN and exhibited symmetrical hysteretic force displacement relationship, see Figure 11a. The failure of the masonry wall occurred due to the high compressive stresses at the base of the wall. This led that only few cracks developed in the wall with the exception of the bottom corners, where crushing occurred, see Figure 11b. The crushing at the base of wall generates buckling in the reinforcements as can be seen in Figure 12. The results of the strain-gauges glued to the horizontal reinforcements h1 and h2 indicate that low stresses developed in these bars, despite the horizontal reinforcement h2 exibits plastic deformation after 70 kN. It should be stressed that vertical reinforcement did not yield and thus it did not contributed to the failure response of the wall. It should be also noticed that unless in the last displacement cycles, the response is almost linear without any energy dissipation.



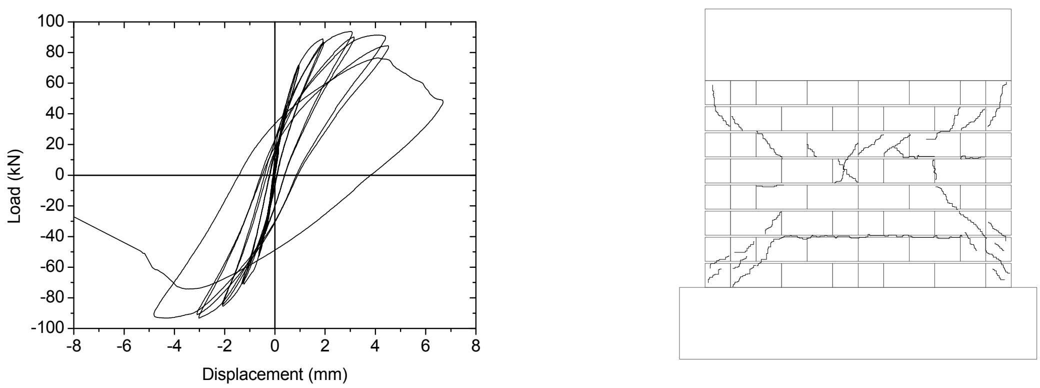
(a) (b)

Figure 11: Behaviour of specimen N150-B1: (a) Diagram Load *vs.* Displacement and (b) cracking pattern

Figure 12: Buckling of the reinforcement because of the high compression stresses.

3.3. N150-B2

The wall N150-B2 reached a maximum lateral strength of 93 kN. Although with a more dissipative response, this wall exhibited an hysteretic force displacement relationship similar to specimen N150-B1, see Figure 13a. The failure of the masonry wall also occurred due to the high compressive stresses in the base of the wall, with crushing of the bottom corners and buckling of reinforcement. However, the cracks were considerably more distributed in whole wall, see Figure 13b. As in the wall B150-B1, only the horizontal reinforcement h2 developed significant strains.

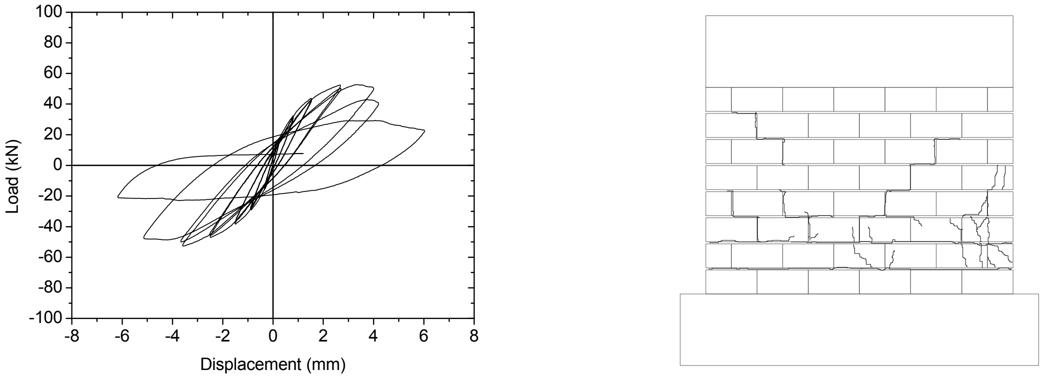


(a) (b)

Figure 13: Behaviour of specimen N150-B2: (a) Diagram Load *vs.* Displacement and (b) cracking pattern

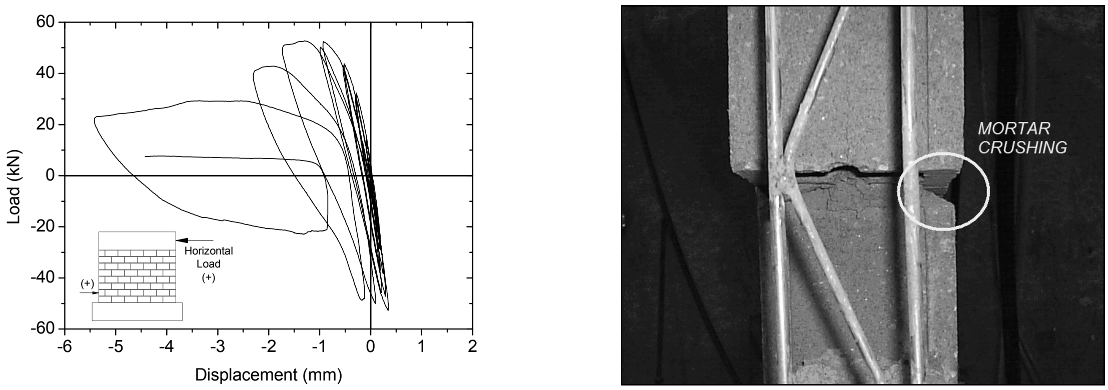
3.4. N60-B1

The force-displacement diagram of wall N60-B1 displayed in Figure 14a shows a symmetrical hysteretic response of the wall. The histeresis loops have associated more dissipation of energy than the walls with a pre-compression level of 150kN. Besides a considerable reduction on the lateral resistance was see, from 93kN to 52.5kN. The failure occurred by toe crushing of the wall followed by sliding of the bed joint between the first and second course, see Figure 14b. This behaviour is also revealed by the evolution of LVDT 3 located at the base of the wall, see Figure 15a. Moreover, a slight crushing of the units was observed above the joint slipping.



(a) (b)

Figure 14: Behaviour of specimen N60-B1: (a) Diagram Load *vs.* Displacement and (b) cracking.



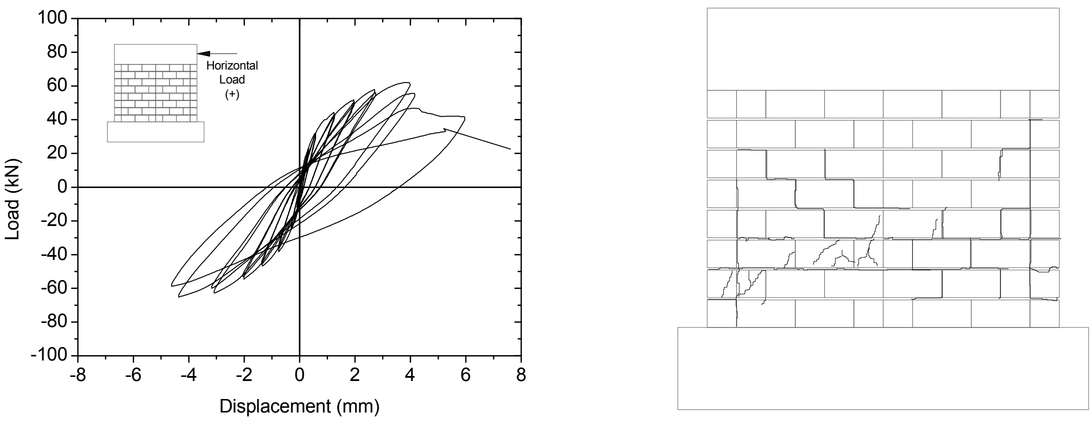
(a) (b)

Figure 15: Behaviour of specimen N60-B1: (a) Slipping of the horizontal joint noted in LVDT 3 and (b) crushing of the mortar in horizontal joint.

In spite of the laboratory mortar production has been controlled, it was seen that a reduced compressive strength of mortar used in the construction of this wall was obtained. This can be a reason for the crushing of the mortar, see Figure 15b. Vertical reinforcement showed some level of buckling but in a lower extent comparing to the walls with axial compression of 150kN. No reinforcement reached the yield strength.

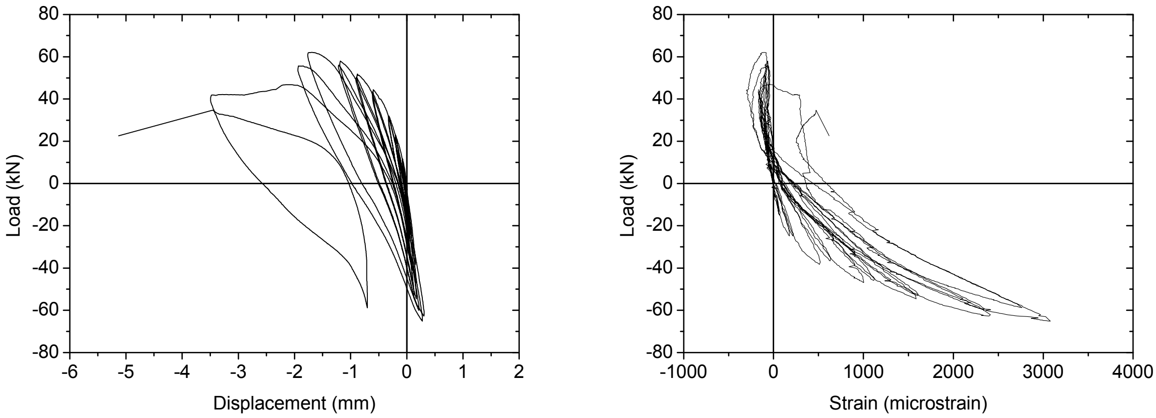
3.5. N60-B2

The wall N60-B2 reached a maximum lateral resistance of 65kN and also exhibited a symmetrical hysteretic force-displacement relationship, see Figure 16a. In the masonry wall, bed joint cracking due to the tensile stresses developed. When the reinforced masonry wall reached the maximum lateral load the crushing of the concrete units of the left corner occurred, see Figure 16b. The post-peak behaviour is characterized by some sliding in horizontal joint between the second and third courses, see Figure 17a. Any reinforcement reached the yield strength, see Figure 17b.



(a) (b)

Figure 16: Behaviour of specimen N60-B2: (a) Diagram load *vs.*displacement and (b) cracking pattern



(a) (b)

Figure 17: Behaviour of specimen N60-B2: (a) horizontal displacements measured byLVDT 3 and (b) strains measured in the vertical reinforcement v1

4. DISCUSSION

The elastic modulus obtained during the application of the first phase of the experimental procedure for the wall panels is displayed in Table 2. It was calculated by averaging the vertical displacements measured by the vertical LVDTs connected at each side of the walls. It is clear the unreinforced masonry wall presents the lowest value of the axial stiffness. On the other hand, the stiffness of the reinforced masonry walls is rather similar. The lowering of the axial stiffness in the wall N60-B1 can in part be explained by the lower compressive strength of mortar. Apart from the absence of vertical reinforcements, this is also valid for the unreinforced masonry. Nevertheless, the lower strength of the mortar does not seem to have influence on the lateral behaviour of the walls. In spite of N60-B1 and N60-B2 present different mortar strength, almost no difference were recorded in the cylic response.

Table 2 – Elastic modulus of the masonry walls

Wall	Elastic Modulus (GPa)
N60-UM	5,1
N150-B1	7,9
N150-B2	8,1
N60-B1	7,1
N60-B2	8,0

The bond masonry appeared not to influence the lateral behaviour of the masonry walls. In fact, the behaviour of wall with continuous vertical joints had no differences in comparison to the wall with running masonry bond., see Figure 18. This result reveals advantages in terms of construction technology since the bond masonry B2 with continuous vertical joints simplifies the placement of vertical reinforcements and masonry units and traditional techniques can be used in its construction.

The results confirms also that the higher axial loading lead to higher strength of the wall under lateral loading. The increasing in the axial loading generates high compressive stresses at the bottom corners promoting a more fragile displacement-time history shown in Figure 7a behavior traduced by a sudden collapse of the walls. From the monotonic envelopes of the cyclic histeresis loops, it is observed that no increase of the maximum displacements is achieved by reducing the the axial load.

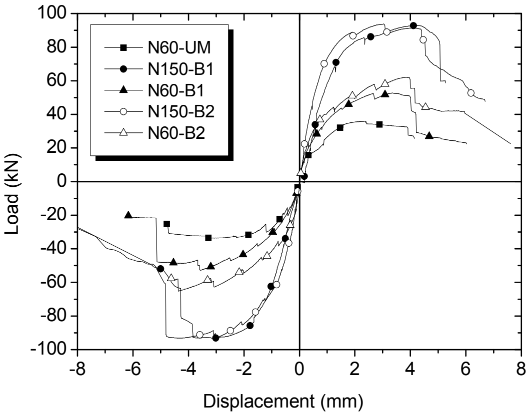
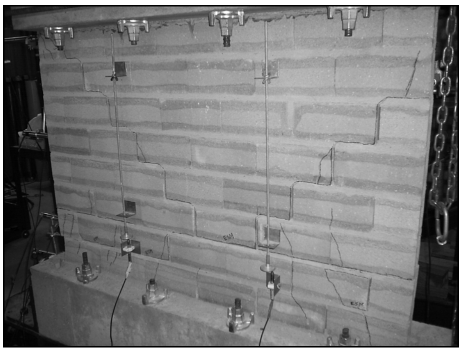


Figure 18: Enveloping curve of the studied masonry walls.

Comparing the behaviour of the unreinforced masonry with the reinforced walls it is possible to verify that the reinforcement increases considerably the lateral strength and, additionally, makes the masonry a more homogeneous material. The unreinforced masonry walls exhibited localised cracks with considerable opening which divided the specimen into two parts, see Figure 19. After the crack opening, the stress transfer between both parts is achieved almost exclusively at the bottom corners where compressive stresses concentrate. This behaviour may be also observed from in-plane rotation of the wall measured by the vertical LVDTs located at the top of the wall. Unreinforced masonry wall had unsymmetrical rotations while the reinforced walls had symmetrical behaviours. In reinforced masonry walls, the stress transfer between the part is made by the horizontal reinforcements enabling a more effective stress redistribution and deformational behaviour as a whole.



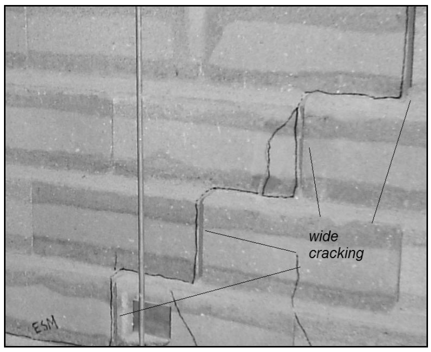
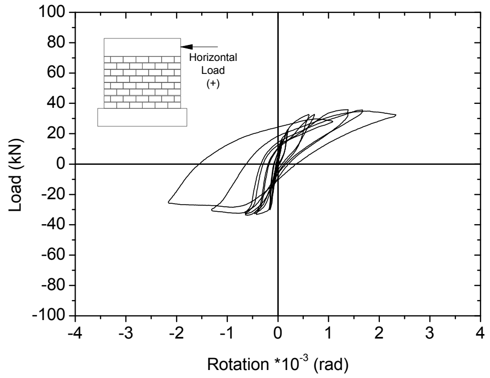
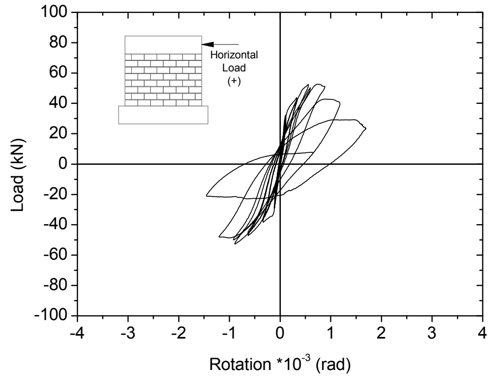


Figure 19: Unreinforced masonry – detachable parts.



(a)



(b)

Figure 20: In-plane rotation: (a) unreinforced masonry wall N60-UM and (b) reinforced masonry walls N60-B1

5. CONCLUSIONS

In order to evaluate different possibilities of masonry walls' reinforcement based on their behaviour under horizontal cyclic loading, an experimental study was planed. Two different masonry bonds and two axial load levels were considered. From the experimental results, the following preliminary conclusions can be drawn:

a) the masonry bond did not influence the behaviour of the reinforced masonry walls
b) reinforcement of walls increased the lateral strength, energy dissipation and becomes the masonry a more homogeneous material
c) reinforced concrete masonry walls exhibit in general reduced lateral drifts but are even higher than the lateral drifts obtained in the unreinforced masonry wall
d) high axial load level increases the reinforced masonry wall strength but leads to a more brittle behaviour with sudden collapse and almost inexistence of energy dissipation

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