EXPERIMENTAL IN-SITU TESTING OF TYPICAL MASONRY CONSTRUCTIONS OF FAIAL ISLAND - AZORES

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ABSTRACT :

The present paper focuses on an *in-situ* testing campaign recently developed in the Faial island, Azores, Portugal, aiming at the assessment of the out-of-plane behaviour of traditional stone masonry walls. This experimental activity, carried out by the Laboratory for Earthquake and Structural Engineering (LESE, FEUP, Porto), started developing some years after the Faial/Pico earthquake on July 9th 1998, in order to provide adequate characterization of the structural response under loading types likely to induce out-of-plane motion of walls as the case of the seismic action. A general description is included concerning cyclic tests carried out to date, complemented with the presentation and discussion of the most important results. Numerical predictions of strength based on simple models are also provided for one tested wall indicating good agreement with experimental findings. This *in-situ* programme involved a second stage for structural strengthening according to schemes similar to some of those adopted during the reconstructions process in Faial Island. Further tests were then performed in order to assess the seismic capacity improvement of the strengthened structures, leading to interesting and promising results concerning both strength and energy dissipation enhancement. The paper also highlights that such type of tests is viable and constitute an excellent means of assessing structural parameters for masonry walls as they exist in reality, in order to feed suitable non-linear behaviour models for structural analysis.

KEYWORDS: In-situ testing, out-of-plane, cyclic, traditional masonry, confined masonry

1. INTRODUCTION AND OBJECTIVES

It is well known that traditional stone masonry constructions exhibit a poor behavior under seismic loading, as presented by D'Ayala and Speranza (2002). In Azores this is periodically and dramatically confirmed during earthquake occurrence, as evidenced after the most severe seismic crises of 1980 in Terceira and of 1998 in Faial, Pico and S. Jorge islands.

Besides other important effects arising in the typically cohesion less material of such masonry structures, the poor out-of-plane response of walls is for sure one of the most critical issues during earthquake action since it is very likely to entail severe wall damage as well as extensive or complete failure of floors and roofs supported by the walls. This effect is particularly relevant for these structural configurations essentially based on stone masonry walls (single or double leaf) and wooden floors/roofs, usually without adequate connections between vertical and horizontal elements. Moreover, this type of horizontal systems are often not very effective concerning in-plane diaphragm stiffness that is essential to ensure a global mobilization of the whole structure by transferring horizontal forces (as desirable) to the walls in their own vertical plane.

In this context, the present work provides some experimental evidence on the response of masonry walls of existing Faial constructions damaged by the 1998 earthquake. After first testing, the structures were strengthened by providing walls with steel reinforced cover and introducing parts of wood floors and roofs duly connected to the walls using appropriate steel elements. The first experimental campaign (Arêde et al., 2007) developed in May 2007 on walls with their original (though seismically damaged) conditions while the second testing stage was performed one year later, in May and July 2008. All the experimental activity was carried out by the Laboratory for Earthquake and Structural Engineering (LESE) of the Faculty of Engineering of University of Porto, with the collaboration of local institutions of Faial on the houses´ preparation before testing and on the essential *in-situ* logistics (Costa et al., 2007).

2. TYPES OF MASONRY UNDER STUDY

Traditional building constructions of Azores, as in the Faial island, mainly consist of masonry bearing walls giving support to wooden floor and roof structures. The most typical type is based on basalt or volcanic stone masonry, with dry joints or poor mortar layers between stones, although concrete block masonry is presently very spread throughout the building stock. Concerning the later, the most common situation is based on reinforced concrete frames filled with block masonry panels but, more recently (particularly after the last earthquake on July 1998), an increasing trend is observed to make this type of construction with duly confined concrete block masonry.
Focusing on the traditional stone masonry, that is the main concern of the present paper, essentially four types can be found in Faial island:

- Irregular masonry, made of stones of several types, dimensions and shapes, randomly placed and using poor mortar generally made of several types of materials, commonly clay based material (Figure 1(a)).
- Dry joint stone masonry, occurring quite frequently in Faial and Pico islands, usually consisting of regularly shaped stone blocks without mortar between them (Figure 1(b)).
- Two leaf masonry walls, with the outer faces made of essentially irregular stone blocks and the inner cavity filled with poor rubble material (Figure 2(a)).
- Upper quality stone masonry, quite regular and frequently with straight sides, parallel faces and adequate strength (Figure 2(b)).

(a) (b)
Figure 1 Stone masonry types: (a) irregular with poor mortar; (b) essentially regular, with dry joints.

(a) (b)
Figure 2 Stone masonry types: (a) two leaf with poor filling material; (b) upper quality regular stone block masonry

The last seismic occurrence of July 1998 made available several damaged constructions, mainly in rural areas of the Faial island. Many of these constructions were declared unusable and some of them still wait for demolition, ten years after the earthquake. Therefore, conditions were set up to adopt some of these constructions to perform in-situ testing that can provide realistic estimates of material and structural characteristics of traditional masonry constructions located in such earthquake-prone zones like Azorean islands.
In this context, the present study mainly concerns experimental testing of masonry constructions existing in rural areas hit by the referred earthquake. The typical and most available types of constructions fit within the typologies shown in Figures 1(b) and 2(a), and further illustrated in Figures 3(a) and 3(b) that refer to the masonry of two houses effectively tested within this experimental campaign.

(a)

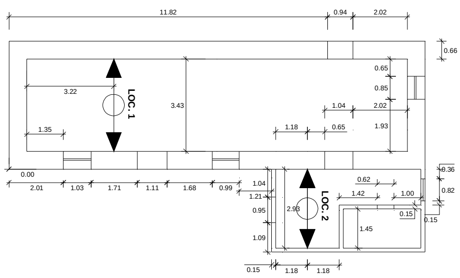
(b)

Figure 3 Effectively tested masonry types: (a) two-leaf irregular masonry with poor fill material; (b) two-leaf regular masonry with dry joints.

3. EXPERIMENTAL PROGRAMME AND SET-UP

The basic idea of the adopted experimental system relies on simultaneous testing two opposite walls of a given house, by applying horizontal forces one against the other and resorting to a pair of hydraulic actuators operating under displacement control. Loading has been applied at the top of walls in the form of quasi-static increasing forces during repeated and alternate cycles, in order to simulate the horizontal action of roofs on masonry walls. Figure 4(a) provides an overall view of the experimental set-up in one tested house, where two experiments were performed on the locations indicated in the plan layout shown in Figure 4(b).

(a)



(b)

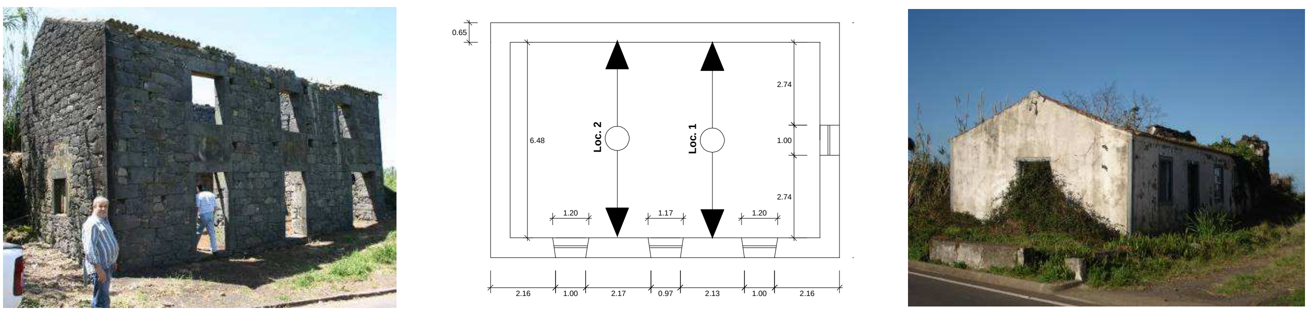
Figure 4 Basics of the testing layout in house 1: (a) general inside view; (b) plan layout and test locations.

For each test the load was applied centered in a given wall panel between door/window openings and distributed along its length resorting to steel and wood pieces. Actuators were powered by a portable hydraulic rig and the whole system allowed applying forces up to about 120kN and maximum stroke +/-250mm. Displacements were measured by means

of draw-wire transducers attached to external reference structures and actuator control could be done in terms of any of the transducers, depending on the particular features of each test.
Measurement points were always chosen according to a T-configuration in the plan of the tested wall as shown in Figure 5(a); this allowed obtaining the vertical deflected shape of the wall panel as well as any torsion movement that could occur at the top level due to different boundary conditions in the left and right sides. This measurement configuration was adopted for both opposite walls involved in the test which provided force-displacement information concerning two different walls for each test. As shown in Figure 5(b), the force was measured resorting to a load cell inserted in the loading system where appropriate hinged bearings have been include to avoid any bending moments.

(a) (b)
Figure 5 Measurement details: (a) layout of displacement measurement; (b) load cell.

It is worth mentioning that with this testing system involving a pair of walls, *a priori* it is not known which wall will perform as the reaction wall because both can move. This is not a drawback of the system and, indeed, by taking adequate care with the actuator control, it is an advantage since two walls are actually being tested. Obviously, when the weaker walls starts degrading or failing, the other is no further explored in terms of imposed displacement, but in the general the obtained outcome consists of good information on the hysteretic behaviour of one wall up to failure and on the initial incipient cracking phase of the other wall.
Three houses were available for testing throughout this campaign, namely two single storey houses in the Salão parish (Horta district) and a two-storey house in Cedros location. The one-storey house 1 is illustrated in Figure 4, while house 2 refers to that shown in Figure 6(a) corresponding to the horizontal layout depicted in Figure 6(b). An outside view of house 3 (similar to house 1) is included in Figure 6(c), for which only one test was made in the same location as for the first test on house 1.



(a) (b) (c)
Figure 6 House 2: (a) general outside view and (b) plan layout with test locations. (c) House 3: outer view

Houses 1 and 3 have two-leaf walls made of basalt stone masonry, quite irregular and of poor quality. House 1 has also a more recent annex made of unconfined hollow brick masonry panels. In turn, house 2 exhibits a much better and regular masonry construction of two-leaf walls without any signs of mortar in the joints.

4. STRENGTHENING SCHEMES

Seismic strengthening for this type of structures was based on proposals implemented after the 1998 earthquake (Costa and Arêde, 2006) and defined according to the following main guidelines: *i)* to improve the strength of masonry walls aiming at preventing their disaggregation during seismic events; *ii)* to enforce a global behavior of the construction as a whole by improving connections between structural elements, namely walls, floors and roof.
In order to pursue these main objectives, masonry walls were strengthened with steel reinforced mortar cover, about 3-4cm thick as shown in Figure 7(a), placed in the outer and inner wall faces (Costa, 2002). Transversal steel rods were included to ensure a more monolithic behavior of the wall cross section by improving the shear connection between the new mortar covers and the existing wall core. This technique leads to a sort of "poor" reinforced concrete wall section, while preserving the original material inside that is quite suitable for hygrothermic and acoustic purposes.
Aiming at simulating the presence of wooden floor/roof structures effectively connected to the walls, two or three criptomeria wood beams (9x20cm2, cross section) were placed between opposite walls, supported and bolted to right angle steel shapes fixed to each wall by steel rods crossing its total section (Figure 7(b)). This procedure was adopted at the roof level of houses 1 (loc. 1) and 2 (loc. 1 and 2). Where floor presence should be simulated, wood boards were also nailed to the beams as adopted in house 2 (loc. 1) and shown in Figure 7(c).

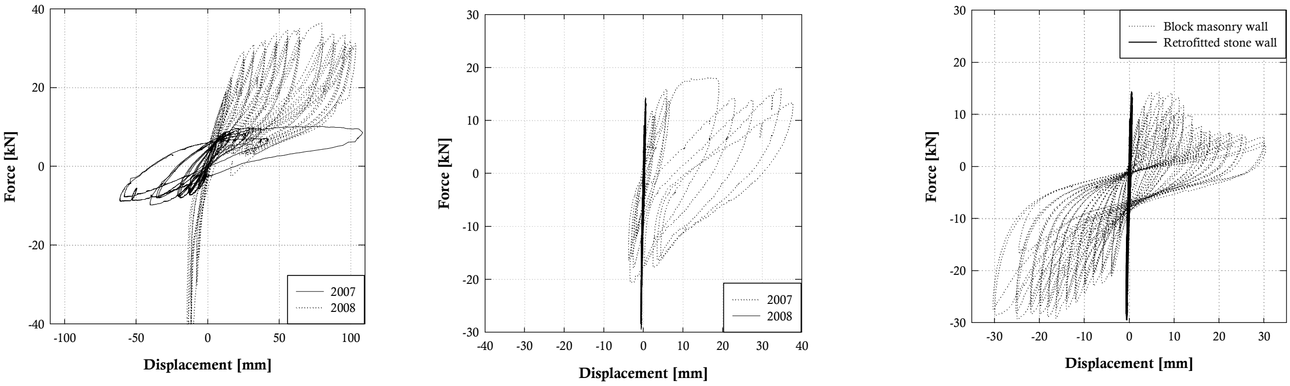
(a) (b) (c)

Figure 7 Strengthening schemes: (a) steel reinforced mortar cover in masonry walls; (b) wood beams supported and fixed to opposite walls; (c) wood boards nailed to floor beams to include pavement contribution

5. TEST RESULTS AND ANALYSES

5.1. Experimental evidence

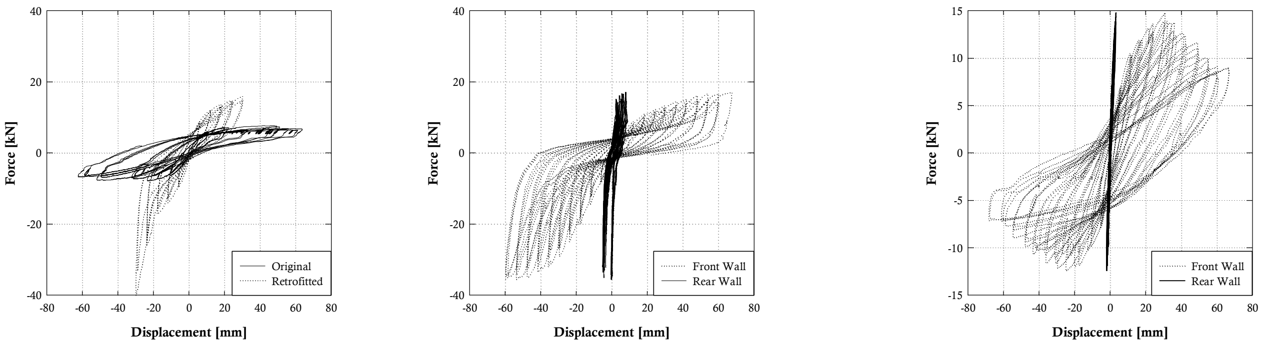
Although not yet fully processed, some of the main tests results are briefly referred here in terms of force-displacement diagrams, where the displacement always refer to the top section of the walls. Plots shown in Figure 8 refer to tests on house 1, including comparisons between results from original (2007) and strengthened conditions (2008).



(a) (b) (c)

Figure 8 Test results for house 1. (a) Location 1: original v.s. retrofitted, (b) location 2: original v.s. retrofitted and (c) brick v.s. retroffited stone masonry panels

Figure 8-a) clearly shows the benefits of the strengthening strategy where the strength increased more than three times the original value for the wall in location 1; the strange response in the negative sense (walls approaching) is due to wood beams that become mobilized in compression. Results for the wall in location 2 are depicted in Figure 8-b), showing that in 2007 the stone masonry panel was seriously damaged during a test where appropriate reaction could be mobilized thanks to the return walls orthogonal to the panel. After strengthening, this panel became so stiff that it hardly move in 2008 tests, which in turn allowed exploring the opposite brick masonry panel as shown in Figure 8-c) somehow reproducing similar results as the ones obtained by Griffith *et al.* (2007) with a different test setup.
Figures 9-a) and 9-b) illustrate test results for house 2, where again the comparison between original (2007) and strengthened (2008) puts into evidence a clear gain of strength for both senses of wall displacement (positive outwards) on location 1; note that wall deflection in 2008 was not pushed so far as in 2007 for safety reasons related to eminent wall instability. The response of both front and rear walls in location 2, connected by roof beams and tested in 2008 for the first time, shows a quite stable behavior with appreciable energy dissipation throughout a loading history that reached about 1.2% drift (equivalent to t/10, where t is the wall thickness).



(a) (b) (c)

Figure 9 Test results: house 2, (a) Location 1, original v.s. retrofitted, (b) location 2, front and rear walls and (c) house 3, front and rear walls

Finally, Figure 9-c) shows the response of tested walls of house 3 where a clear degrading mechanism is evidenced after reaching 1% drift, although with significant capacity for energy dissipation.

5.2. Analitycal predictions

House 2 has a structural typology that allowed a clear definition of the wall zones involved in the tests and, therefore, permitted obtaining simple estimates of the maximum lateral out-of-plane strength of the tested wallet. Thus, resorting to limit analysis, according to which the forces compatible with equilibrium must follow Eqn. 5.1 (where tensile strength of joints between blocks is neglected), it is possible to estimate the maximum force H_{top} to be applied at the top of the wallet. In this expression, h_{top} refers to the wall height (at the level of the applied lateral force), W is the wall weight and t stands for the corresponding thickness.

$$M_0 = H_{top} \cdot h_{top} - W \cdot \frac{t}{2} = 0 \tag{5.1}$$

In order to estimate the lateral force compatible with the wallet self-weight in the non-strengthened version of the tests, the unit weight of masonry was assumed as $\gamma = 18\,kN/m^3$ according to mean values obtained in previous studies (Costa, 2002). The remaining values were taken as $l = 2.13\,m$, $t = 0.66\,m$, $h = 5.0\,m$, $h_{top} = 4.6\,m$, from which the wallet weight (W) was obtained as 127.1 kN. By introducing these values in Eqn. 5.1 the maximum top force compatible with limit equilibrium is $H_{top} = 9.1\,kN$.
Moreover, using this maximum force and the maximum allowable displacement compatible with the system equilibrium ($d_{max} = t/2 = 0.33\,m$), it is possible to obtain the maximum envelope foreseen for the wallet response given by the dashed line included in Figure 10 since for $d = 0\,m \rightarrow H_{top} = 9.1\,kN$ and for $d = d_{max} \rightarrow H_{top} = 0\,kN$. The same figure includes the results obtained for the non-strengthened wall test (both front and rear walls) and it is quite apparent that the front wall response curve is in fairly good agreement with the above estimated limit lateral force.

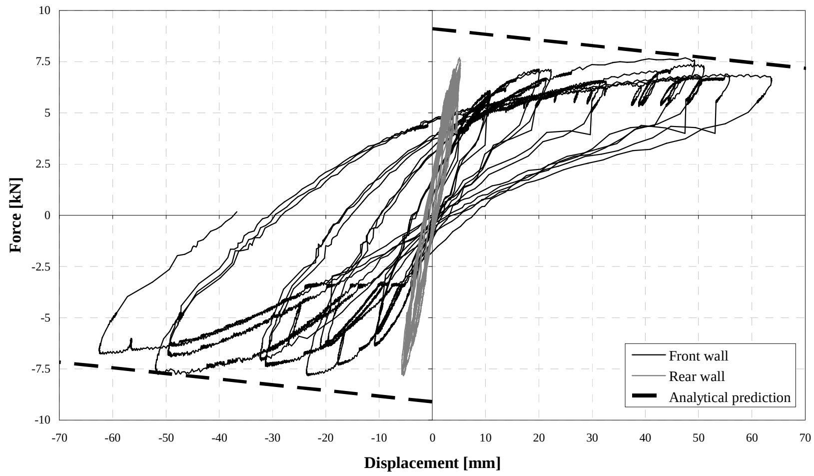


Figure 10 Top force-displacement diagram and limit analysis predictions

Figure 10 shows also that the tested wall exhibited a very dissipation of energy, though with stiffness decrease that becomes quite significant after reaching about 5mm top displacement (approximately 0.1% total drift). Regarding strength, degradation started beyond 50-60mm, thus about 1.2% drift. The envelope of the measured response shows a strong trend of fast loss of strength, such that, a mere extrapolated draft of the referred envelope would suggest the total loss of strength for displacements on the order of 100 to 120mm (2.5% drift). However, this value would stay far behind the theoretical maximum displacement (330 mm) estimated from limit analysis, which points to a very deficient capacity of the wall to accommodate out-of-plane displacements while ensuring adequate strength reserve.
In the framework of out-of-plane behaviour of this type of masonry, the mentioned deficiency can be the critical issue in the case of real seismic events because, should residual displacements keep significant (as in the present case for one response sense) then instability can develop leading to the collapse of these elements.
Figure 10 shows also that the rear wall behaved as a reaction wall since the level of reached peak displacements was far below (5 mm) that of the front wall, thus allowing the later to be more explored in terms of non-linear behavior.

6. FINAL REMARKS

Although in a very succinct way, the above described allows concluding the great potential of the in-situ tests as performed in the last two years on typical masonry constructions of Faial Island, Azores. The outcome of such tests is of utmost importance for understanding the behavior of traditional masonry walls as they exist, particularly concerning the out-of-plane response as addressed in this study. A significant contribution is also achieved for appropriate numerical modeling calibration. In addition, concerning the assessment of actually implemented strengthening interventions, the brief insight herein provided allows concluding the clear improvement of structural behavior under cyclic loading. This fact is even more important since these strengthening schemes were effectively used in the reconstruction process after the 1998 earthquake.

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