EXPERIMENTAL RESEARCH ON MASONRY WALL AND TIMBER ELEMENTS CONNECTION

Susana Moreira, Daniel V. Oliveira, Luís F. Ramos and Paulo B. Lourenço

ISISE, Department of Civil Engineering, University of Minho, Guimarães, Portugal

Abstract

Studies developed for the past years and recent earthquakes have brought to light the importance that efficient connections between structural elements can have in the global behaviour of the structure. Although out-of-plane mechanisms of masonry walls are considered to be local, the reality is that their occurrence can lead to total collapse of the structure. The scarcity of data, both at experimental and numerical levels, introduces an urgent need to characterize the response of connections to seismic actions and understand their impact on the overall behaviour of the structure. Therefore, a total of 15 quasi-static monotonic and cyclic pull-out tests were performed on representative wall-to-floor (timber pavement beams) and wall-to-timber framed wall connections, unstrengthened and strengthened, in order to assess their performance and allow their characterization. The analysis contemplates parameters like: failure mode, hysteretic curve, strength degradation and total energy. The study of these parameters promotes better understanding of this type of connection and also the development of design recommendations for the strengthening. As further development, the experimental data will allow calibration of representative numerical models, enabling parametric studies of material properties and formulation of backbone curves.

1. INTRODUCTION

The observation of recent post-earthquake scenarios shows the out-of-plane collapse of masonry walls as one of the most common failure mechanisms. Although is a local mechanism, it can compromise the overall stability of the structure. The absence of appropriate structural connections is one of the main factors in the activation of this type of failure mechanism [1].

Usually, in the design and analysis of structures, connections are considered rigid or flexible, which represents two extreme behaviours that often do not reflect the true response of the connection. Recent studies conducted in 'Pombalino' and 'Gaioleiro' buildings have shown that the consideration of these two extreme situations has impact on the numerical results and, consequently, on the assessment of seismic vulnerability of buildings [2-3].

After the 1755 earthquake, which damaged downtown Lisbon severely, it was necessary to adopt a structural system less vulnerable to earthquakes. Engineers, at the time, developed an innovative structural system able to combine the flexibility of a timber structure with the bearing capacity of masonry walls. 'Pombalino' was the name given to the type of buildings made of a timber three-dimensional cage, carefully assembled with carpentry joints and connected to the masonry walls by embedded elements. The urgency in rebuilding due to the need to relocate much of the population has led to the introduction of simplifications in the structure and use of materials of inferior quality, leading to constructive decay and the emergence of a new building typology, so-called 'Gaioleiro'. The transition between the two typologies occurred over 175 years, led to great variability of characteristics and construction details, but this study focus on the most current and deficient connections.

Some aspects which characterize the buildings of lower quality are, when compared to the 'Pombalino' buildings are: loss of continuity of the timber cage, variable thickness of the masonry walls along the height of the buildings and increase on the number of floors (increase of the total height). The differences introduced changes in wall-to-floor and wall-to-timber framed wall connections, which in most cases decreased their efficiency.

In 'Pombalino' buildings the most important connections were made between the elements of the timber cage and less importance was given to the connection between the cage and the masonry walls. The initial design of this typology of buildings contemplated that in case of earthquake, the timber frame would be flexible enough to ensure structural stability in the event of collapse of the masonry walls [4]. As observed, for buildings up to two storeys affected by the 1998 Azores earthquake, this kind of response is possible, but there are many uncertainties regarding taller buildings [2]. The typical wall-to-floor connection consisted of timber pavement beams connected to two timber beams, by carpentry joints and 8 to 30 cm iron nails. The two timber beams were placed horizontally, at the top and bottom of the pavement beam, at 5 cm from the inner face of the wall. Original drawings include the presence of metal connectors, anchoring some of the pavement joists to the masonry walls [4] [5]. In late 'Pombalino' and 'Gaioleiro' buildings the connection was simplified, leading to the omission of carpentry joints or even the embedded timber beams, leading to the direct support of the beam on the wall. With the increased use of steel in construction, connections began to incorporate steel profiles as a supports [6].

The wall-to-half-timbered wall connection, present in 'Pombalino' buildings, took advantage of the existence of the 3D timber cage, providing connection between the timber elements embedded in the wall and the timber elements of the half-timbered wall. Over time, this connection deteriorated and, for 'Gaioleiro' buildings, the timber framed walls basically leaning against the masonry wall, being the connection established by the pavement beams. Although, different types of connections exist, the continuity of load transmission was assured by friction, adhesion and shear strength of the masonry wall. In order to promote better seismic behaviour, and consequently reduce the seismic vulnerability of such buildings, some strengthening solutions have been suggested, but few have been investigated experimentally and numerically [7].

The scarcity of experimental and numerical data on the behaviour of connections has functioned as an obstacle to incorporate them in the global analysis of buildings. Thus, this research aims at studying the seismic behaviour of wall-to-floor and wall-to-timber framed wall connections and providing experimental data capable of include their properties in the overall analysis of buildings.

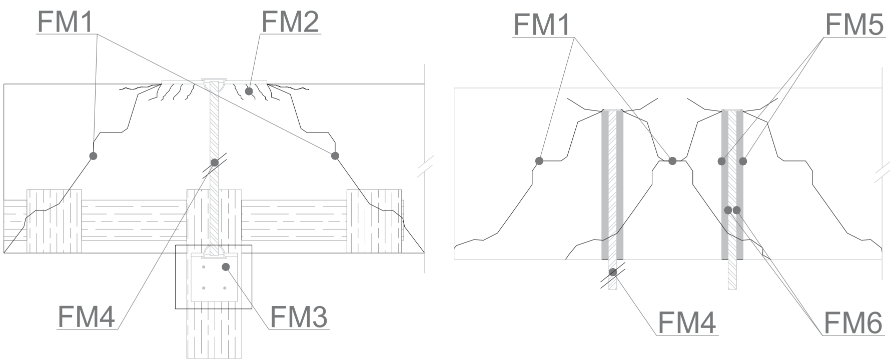
The experimental campaign consists of 15 pull-out tests: 8 quasi-static monotonic tests on wall-to-floor connections and 7 quasi-static monotonic and cyclic tests on wall-to-timber framed wall connections. These tests were accompanied by mechanical properties characterization of the mortar and masonry. The compressive strength of the mortar is approximately 1.4 MPa, while the masonry presents the average value of 1.7 MPa.

2. MATERIALS

Each rubble masonry wall was hand constructed, by professional masons, using limestone with a maximum dimension of 200 mm and at most 50 mm joints. The walls destined for wall-to-floor connections can be 400 and 600 mm, while for wall-to-timber framed wall connections they will be only 400 mm.

For the wall-to-floor connections, a timber beam of 95×95 mm^2 was built in along the wall ('frechal') and nailed to the perpendicular timber joists of 130×180 mm^2, which are spaced of 380 mm. The anchorage length of the timber beam is 150 mm and the nails are located at approximately 80 mm of the edge of the pavement beam inside the wall. Only two of the perpendicular pavement timber beams are extended beyond the walls, since only 2 tests will be performed per wall (see Figure 1a). The strengthening solution for this type of connection is a steel angle bolted to the timber pavement beam which is anchored to the external face of the wall, by means of an anchor plate. The anchor tie is a stainless steel, AISI 304 class 80, diameter □16 and applied at 15° angle. This solution contemplates the application of a GFRP layer, around the timber pavement beam, on the place where the steel angle is placed (see Figure 2a).

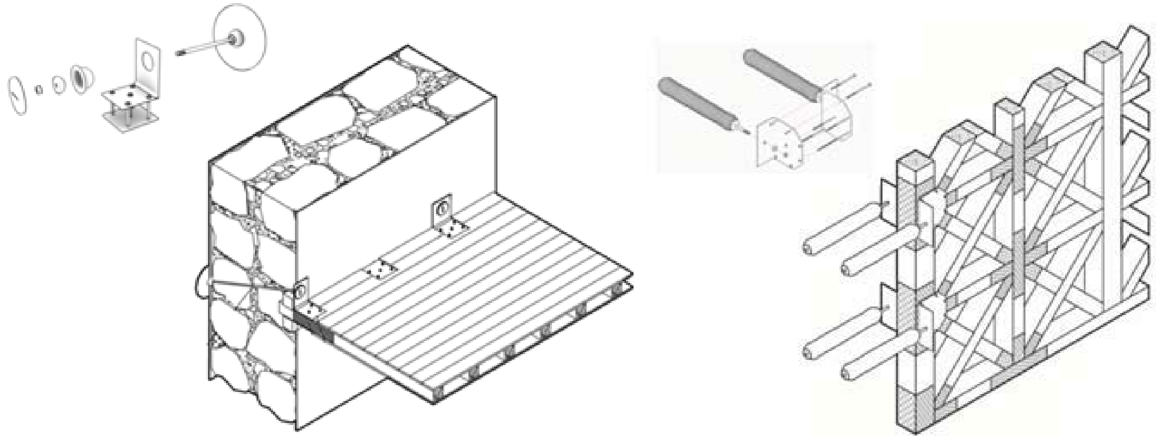
For wall-to-timber framed wall connections, the masonry wall does not have timber elements incorporated since no connection is assumed between the two structural elements. Therefore, only the interaction between the strengthening solution and the masonry wall will be studied (see Figure 2b). The injected anchors were placed in pairs, on boreholes of 50 mm, spaced of 280 mm, considering that a 120 mm thick timber framed wall could fit between them (see Figure 2b). The steel bars were stainless steel AISI 304 class 70 and had a diameter of ϕ20 (wall 1) and ϕ16 (wall 2).



(a) (b)

Figure 1: Failure modes: (a) wall-to-floor connections;
(b) wall-to-timber framed wall connections.

As shown in Figure 1, the expected failure modes are: formation of the shear pull-out cone (FM1), crushing of the masonry under the anchor plate (FM2), failure of the bolted connection between the steel angle and the timber pavement beam (FM3), yielding of the anchor tie (FM4), sliding at the interface between the borehole surface and the sleeve with injected grout (FM5) and sliding at the interface between the steel anchor and the injected grout (FM6).



(a) (b)

Figure 2: Strengthening solutions: (a) wall-to-floor connection;
(b) wall-to-timber framed wall connection.

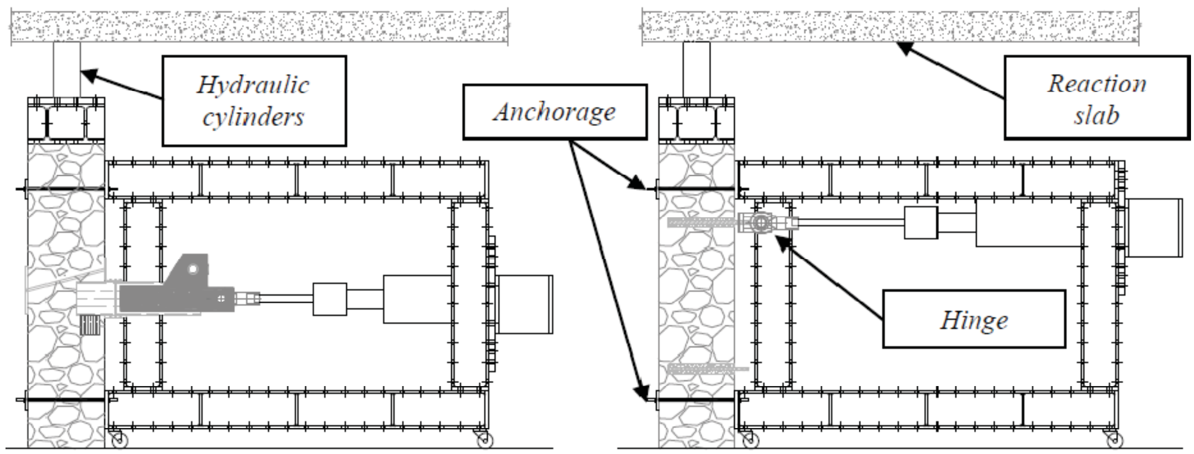
2. TEST SET-UP

Considering the laboratory limitations in terms of space as well as the size of specimens, it was possible to develop a self-balanced set-up capable of redirecting the pull-out force back to the specimen, as shown in Figure 3b. The pull-out load, which intends to recreate the main seismic action, was applied perpendicular to the wall in order to activate the tensile capacity of the injected anchors. A hinge was used between the actuator and the specimen to accommodate small deformations. In order to perform cyclic tests, the set-up had to be anchored to the masonry wall.

In order to simulate the compressive state of the walls resulting from permanent loads, four hydraulic cylinders were placed over rigid steel profiles on top of the wall (see Figue 3b). Since the execution of the strengthening until testing, the compressive state of 0.2 MPa or 0.4 MPa, was kept constant through manual control of the pressure. These compressive stresses of 0.2 and 0.4 MPa, correspond, respectively, to the thicknesses of 400 and 600 mm.

The instrumentation of the pull-out tests was guaranteed by LVDTs and strain gauges. The LVDTs were distributed on the strengthening and its surrounding area (wall and timber elements), with the aim of capturing the formation of the shear pull-out cone. The strain of steel bars was monitored with strain gauges: 2 on wall-to-floor connections and 8 on wall-to-timber framed wall connections.

The quasi-static monotonic tests were conducted under displacement control, at a velocity of 10 μm/s, to prevent explosive failures and monitor transitions between elastic and plastic phases. As stopping criteria were considered: 50% drop of the peak force, damage progression outside the expected area of damage and for wall-to-floor connections, a maximum out-of-plane displacement of 120 mm of the timber pavement beam. To date, quasi-static cyclic tests were performed on wall-to-timber framed wall connections and the procedure was defined based on the monotonic behaviour. The procedure consists of loading and unloading cycles, until 20 mm, with two repetitions per cycle and a range of velocities from 10 μm/s to 40 μm/s.



(a) (b)

Figure 3: Test set-up: (a) wall-to-floor connections; (b) wall-to-timber framed wall connections.

3. RESULTS

3.1 Wall-to-floor connections

To date, a total of 7 monotonic tests were performed: 4 tests on unstrengthened specimens and 3 tests on strengthened specimens. As mentioned before, one of the main objectives is to study the different possible failure modes and pull-out forces. The performed tests reflected this concern and also gave insight into some of the necessary design considerations for the strengthening of these connections. The performance parameters relative to the pull-out tests are presented in Table 1. The diversity of failure modes is the result of single changes made on the specimens, while maintaining the same test set-up throughout. To make clear the designation of the specimens: WF stands for wall-to-floor connection, 40 or 60 refers to the thickness of the wall in cm, A is for anchor plate (type of strengthening), U is for unstrengthened and 1A or 2B is the number of the specimen plus the location at the right (A) or left (B).

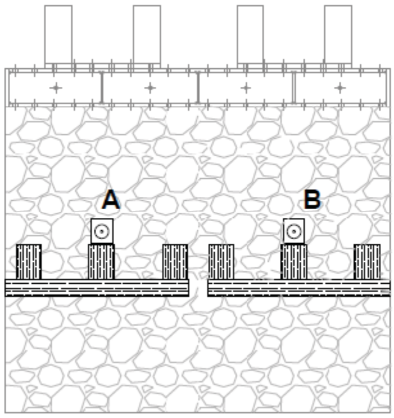
As expected, the prevalent failure mode on unstrengthened connections is the pull-out of the nails between the timber beams. The out-of-plane displacement correspondent to failure is related to anchorage length of the nails of each connection.

For the first strengthened specimen, WT.40.A.1A, were used 4 $\phi 6$ bolts to connect the steel angle to the pavement beam, which failed in shear as can be seen in Figure 4a. Each of the drops of the force-displacement curve corresponds to the failure of each bolt. In total, the bolts ripped the timber beam approximately 50 mm. The majority of the displacement obtained is due to sliding of the beam, since small displacements were measured on the wall, below 1 mm.

For the following specimen, WF.40.A.1B, the ϕ6 bolts were substituted by □10, to prevent the failure of the bolts. Consequently, another failure mode was found for a higher load – this one associated with the tie rod. At each end of the tie rod only one nut was used to tighten it onto the hinges, which surprisingly proved to be insufficient when tested with the large diameter bolts. There was a sudden drop of the force, defining this failure mode as brittle. Again large displacements were caused by sliding of the pavement beam.

Table 1: Performance parameters relative to the pull-out tests of wall-to-floor connections

Specimen	Failure mode	F_u (kN)	d_u (mm)	Specimen
WF.40.U.1A	Nails pull-out	9.57	24.07	WF.40.U.1A
WF.40.U.1B	Nails pull-out	4.84	22.50	WF.40.U.1B
WF.60.U.1A	Nails pull-out	4.43	15.25	WF.60.U.1A
WF.60.U.1B	Nails pull-out	4.12	37.21	WF.60.U.1B
WF.40.A.1A	Shear failure of the bolts (FM3)	65.50	39.64	WF.40.A.1A
WF.40.A.1B	Stripped threads of the anchor tie and nut (FM3)	76.87	38.20	WF.40.A.1B
WF.40.A.1A - Repetition	Formation of the pull-out cone (FM1)	85.20	52.42	WF.40.A.1A - Repetition
WF.60.A.1A	Crushing of the embedded beam (FM3)	108.62	56.14	WF.60.A.1A



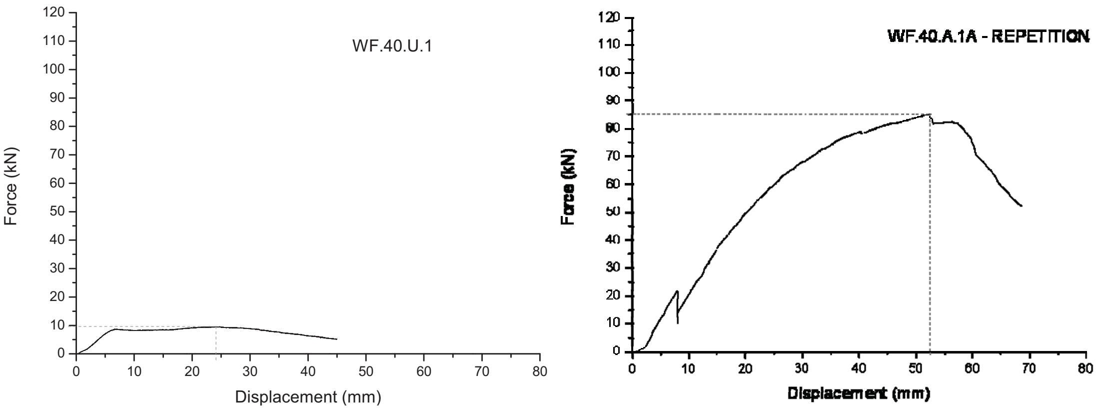
Since small displacements were measured during the test of WF.40.A.1A and no visible damage was observed on the wall and embedded beam, it was performed another test on the same location of the specimen. Only the timber pavement beam was replaced and the strengthening reapplied (WF.40.A.1A - Repetition). The only disregarded detail was the nails connecting both timber beams, which was considered admissible due to their small impact in the previous tests. To avoid the failure mode of the previous test, the tie rod was tightened with 3 nuts on each end. With this one exception the test was otherwise conducted identically to the previous. This test resulted in the formation of the pull-out cone on the masonry wall.

As shown in Figure 4b, general diffused cracks starting from the anchor position were observed, in parallel with the internal separation of the wall "leaves" and crushing of the masonry under the anchor plate.

(a) (b)

Figure 4: Failure modes: (a) shear failure of the bolted connection pavement beam/steel angle (FM3); (b) generalized cracking from the formation of the shear pull-out cone (FM1).

In Figure 5, are presented two of the force-displacement curves obtained for unstrengthened and strengthened specimens, respectively. As observed, the difference in behaviour is substantial, presenting much larger ductility for the strengthened connection. It was observed (see Figure 5b) that the specimens present a drop of load between 20 kN and 40 kN. This can be explained by the detachment between the steel angle and the timber beam, since they are glued. Before placing the steel angle, the timber beam is wrapped with FRP, which promotes better adhesion between those two elements. This behaviour is affected by the distribution of adhesive, which can explain the decrease of load at different levels or even its absence.



(a) (b)

Figure 5: Force-displacement curves: (a) WF.40.U.1A; (b) WF.40.A.1A -Repetition

After obtaining the formation of the pull-out cone for a 40 cm specimen, it was decided to replicate the same strengthening details on a 60 cm specimen and the results were notably different. The connection failed by cracking and ripping the embedded beam from along the full length of the beam due to the compression exerted by the pavement beam. It was observed that the ripping pattern followed the plan of the nails and also during the test was observed a significant rotation of the pavement beam. It is possible that the pavement beam rotated when being pulled due to the 15° angle of the tie rod, which in association with an increase of the compression of the wall (0.5 MPa) led the embedded beam to rip along the nails plan.

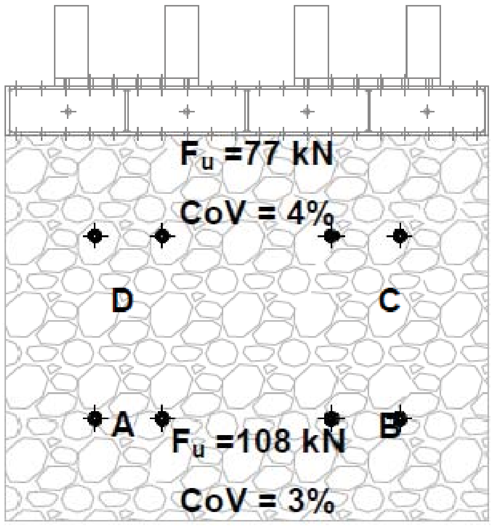
3.4 Wall-to-timber framed wall connections

A total of 7 pull-out tests were performed: 2 monotonic and 5 cyclic. Some of the characteristics of the tests are presented in Table 2. One of the main objectives was to characterize the failure modes related with the masonry wall, so high class steel was chosen for the steel bars, in order to prevent steel yielding. Formation of the shear pull-out cone was the recurrent failure mode. Nevertheless, total displacement is a combination of the masonry shear cone with sliding of the interfaces.

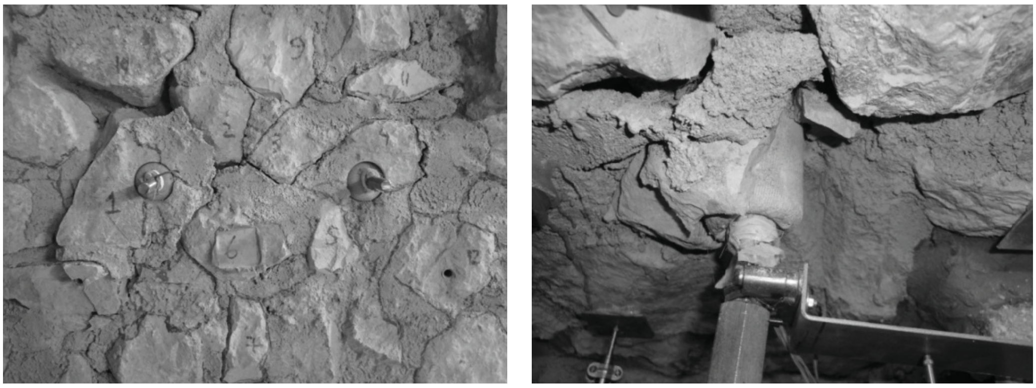
From Table 2, is possible to confirm that boundary conditions influenced the behaviour of the anchors. At the base of the wall, the maximum pull-out force was 108 kN, with a CoV of 3 %, while at the top the same parameter reached 77 kN, with a CoV of 4 %, showing that the base is more rigid than the top.

Table 2: Performance parameters relative to the pull-out tests of wall-to-timber framed wall connections

Specimen	Procedure	F_u (kN)	d_u (mm)
WT.40.I.1A	Cyclic	111.68	5.93
WT.40.I.1C	Monotonic	76.93	9.16
WT.40.I.1D	Cyclic	81.21	3.94
WT.40.I.2A	Cyclic	107.21	8.52
WT.40.I.2B	Cyclic	104.93	10.21
WT.40.I.2C	Cyclic	74.95	7.73
WT.40.I.2D	Monotonic	74.26	7.03



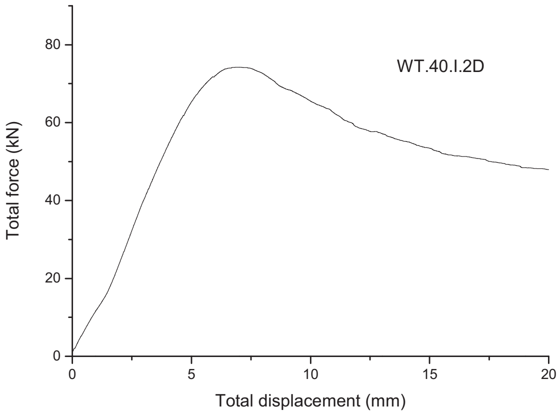
Shear failure of the masonry was always characterized by generalized formation of cracks around the two anchors and in some cases detachment of mortar or stones. Minor cracking was also found close to the boreholes at the end of the anchors. Further tests will allow the characterization of the contact area of the borehole, which is expected to have major importance on adhesion.



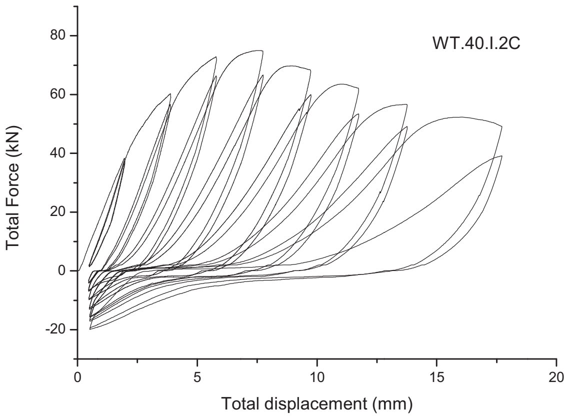
(b)

Figure 6: Failure modes: (a) generalized cracking from the formation of the shear pull-out cone (FM1); (b) sliding at the interface between the borehole surface and the sleeve with injected grout (FM5)

Envelope curves obtained for cyclic tests show great resamblence with the monotonic force-displacement curves (see Figure 7), demonstrating high increase of stiffness up to 10 mm, followed by softening behaviour. For the cyclic tests, there is a visible decrease of strength between cycles, which is made clear in Figure 7a. It is interesting to observe how stiffness of the repeated cyle is similar to the one of the following first cyle.



(a)



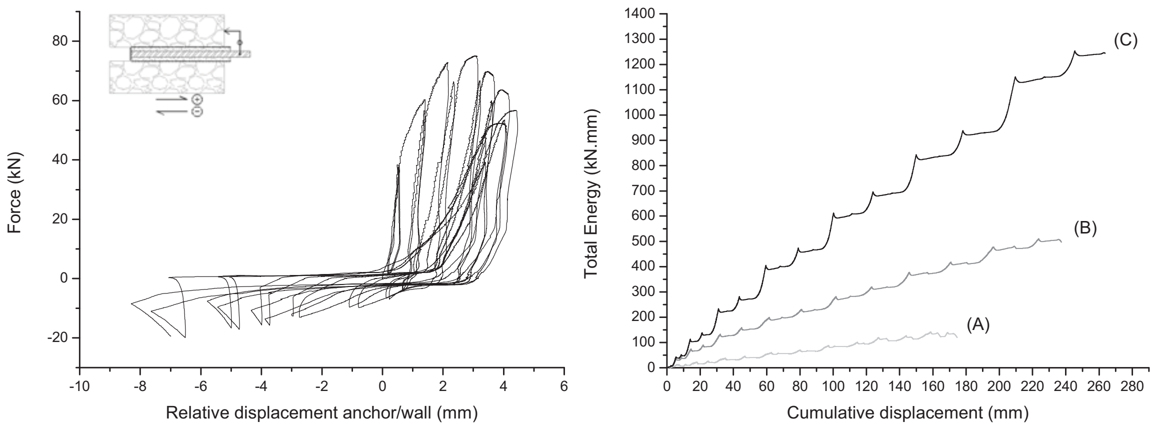
(b)

Figure 7: Force-displacement curves of the strengthening solution
(a) montonic test and (b) cyclic test

The LVDTs measured out-of-plane absolute displacements, allowing the determination of relative displacements between different parts of the specimen. The following relative displacements were considered to be of major importance due to their contribution to the failure mode and energy dissipation:

(A) loaded end to free end of the anchor;
(B) loaded end of the anchor to surrounding points in the front side of the wall;
(C) loaded end of the anchor to the back side of the wall.

The first relative displacement (A) characterizes the contribution of the anchor; the second (B) gives information about the contribution of the anchor plus the anchoring system, while the third relative displacement (C) accounts for all contributions (anchor, anchoring system and wall). Figure 8a shows one of the force-displacement curves obtained for the relative displacement B (loaded end of anchor 2 to the front side of the wall) for specimen WT.40.I.2C, considering the force distributed to this anchor.



(b)

Figure 8: Cyclic pull-out of WT.40.I.2C: (a) Force-displacement curve for the relative displacement between anchor 2 and front side wall; (b) different contributions of the cumulative energy of anchor 2

To better understand the contribution of each portion, their total energy was determined (see Figure 8b). As expected, relative displacement C presents higher values since it comprises all the energy dissipated. It is also confirmed that the formation of the shear cone (difference between relative displacements C and B) is responsible for most of the dissipated energy, confirming it as main failure mode. The steel bar itself (relative displacement A) represents a lower contribution when compared to the shear cone, probably as a consequence of the higher grade of the steel used.

4. CONCLUSION

The presented research adds critical experimental information about wall-to-floor and wall-to-timber framed wall connections, which allows a better understanding of the subject. The laboratorial campaign was successfully carried out, allowing the definition of several failure modes, in terms of maximum strength and damage to be expected.

For wall-to-floor connections, results showed that the behaviour of unstrengthened and strengthened connections is the combination of different contributions, even if a specific failure mode stands out. For strengthened specimens, bending of the bolts and crushing of the timber beam against the bolts are always present in the behaviour of the connection. To date, only these tests were carried out but further work will comprise the repetition of the tests and also the study of the same connections under cyclic loading. This research raised the necessity of performing smaller tests to characterize specific properties as the friction coefficient, the shear strength of the masonry wallets and others.

For wall-to-timber framed wall, as expected, shear failure of masonry was obtained and a range of maximum pull-out forces was established, approximately from 77 to 108 kN. This interval is mainly explained by the different boundary conditions of the wall, at the top and bottom. Nevertheless, displacements due to sliding of the interfaces contributed to the final damage distribution. The analysis of the total energy of each anchor showed that the formation of the masonry shear cone is responsible for a great portion of the energy released and also enable the quantification of the contribution of each portion to the total displacement.

This research adds critical experimental information about connections, which creates the base to better understand their behavior and to develop numerical and analytical models.

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