Experimental study on the seismic behavior of masonry wall-to-floor connections

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SUMMARY:
The global structural performance of masonry buildings, under earthquake loading, is affected by the efficiency of wall-to-floor connections, since they assure the continuity of the energy path and prevent the occurrence of most of the local collapse mechanisms. In fact, out-of-plane behaviour of masonry walls observed in recent seismic events showed the critical importance of proper connections in historical buildings. A review of current literature yields little in terms of experimental and numerical data on the subject. Thus, there is an urgent need to study the behaviour of these connections. The present paper presents a series of tests carried out to characterize the wall-to-floor connections. Different specimens were constructed in laboratory to represent connections found in 'Gaioleiro' and Late 'Pombalino' buildings in downtown Lisbon. Pull-out tests of wall-to-floor connections were carried out on unstrengthened and strengthened specimens in order to study failure modes, maximum pull-out forces, and corresponding displacements. These parameters allow better understanding of this type of connection and also the development of design recommendations for the strengthening.

Keywords: wall-to-floor connections, 'Pombalino', 'Gaioleiro', pull-out test, failure modes

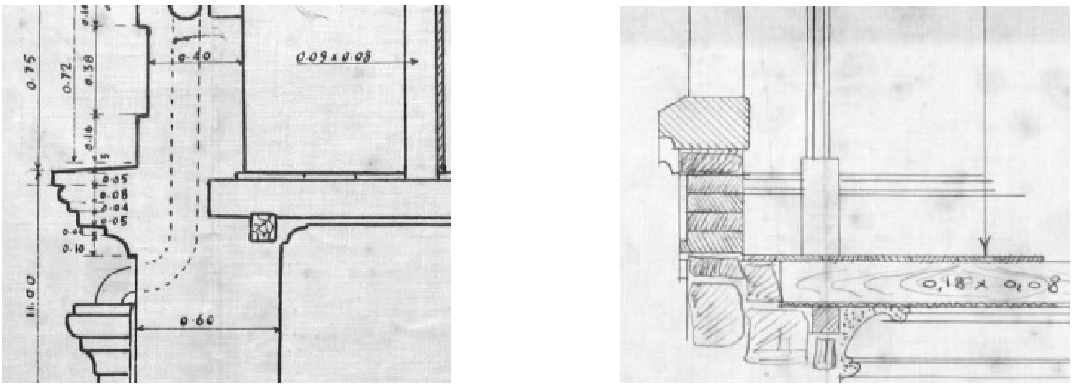
1. INTRODUCTION

Experience from recent earthquakes demonstrated how the presence or the absence of efficient connections plays a fundamental role in seismic resistance. Proper connections prevent overturning of external walls and improve the in-plane behaviour of façades and walls by activating global equilibrated mechanisms (Tomaževič et al, 1996). Some strengthening solutions are suggested to improve the seismic behaviour of connections, but very few were the target of in depth experimental research and numerical modelling (Tomaževič, 1999). Recent works on 'Pombalino' and 'Gaioleiro' buildings call attention to the impact that structural connections have on numerical results and consequently on the assessment of seismic vulnerability of buildings (Bento, 2005) (Mendes & Lourenço, 2010). When analysing or designing structures, it is common to consider the connections rigid or flexible; this in fact only represents the extreme behaviour of the connections and consequently doesn't allow true knowledge of their impact on the global behaviour. Therefore, this research aims at studying the local behaviour of connections subjected to seismic actions and providing enough experimental data for understanding their impact on the building global performance.

The so-called 'Pombalino' buildings resulted from the effort of engineers to answer the necessity of rebuilding Lisbon after the 1755 earthquake. Most of the downtown built environment was destroyed by the devastating actions of the earthquake and resulting tsunami and fires. In response to the need to rebuild with a construction type less vulnerable to seismic events, engineers invented a structural design that combined the flexibility of a timber frame with the bearing capacity of masonry walls. 'Pombalino' buildings rely on the continuity of a three-dimensional timber cage, assured by carefully made carpentry joints and the connection between masonry walls and the timber frame. The necessity to rebuild in order to house to the population of Lisbon in the wake of the earthquake led to faster and cheaper construction with disregard for the details that led to the effectiveness of 'Pombalino' construction. Late 'Pombalino' and 'Gaioleiro' buildings are the result of this loss of quality over time. Some aspects that distinguish these buildings from 'Pombalino' style are: loss of continuity of the timber cage, variable thickness of the masonry walls over the height, and an increase in storeys (increased overall height). These differences had great impact on the evolution of wall-to-floor connections (Mascarenhas, 2009).

To date, new questions arise concerning the safety of these buildings. Are connections active enough to assure the desirable structural seismic performance on the original buildings? What type of strengthening techniques for the connection will work better to improve the seismic response of the buildings? To answer part of these questions, the present paper addresses an experimental campaign in order to study the behaviour of the tradition wall-to-floor connections, with and without strengthening.

For this research, a wall-to-floor connection was defined as the connection between the masonry wall and timber pavement beam placed perpendicular to it. There are distinguishable differences between connections found in 'Pombalino' and buildings built afterwards. For the original 'Pombalino' building, it was more important to assure a proper connection between the timber elements of the cage than between these and the external masonry walls. The pavement beams and the beam embedded in the wall (5cm from the internal face) were connected by carpentry joints and were nailed to each other with 8 cm to 30cm iron nails. On top of the pavement beam was placed another embedded beam, parallel to the one under it, pinning the pavement beam between them. There was the presence of metallic connectors between the pavement beams and the masonry walls, but with different purposes if the wall was external or a party wall (Mascarenhas, 2009). Pinho (2000) refers that, in some cases, there were metallic ties assuring a good wall-to-floor connection because the timber pavement beams would extend deep into the wall or would be anchored to the external face of the wall. The wall-to-floor connections of Late 'Pombalino' buildings still used embedded beams along the wall as well as carpentry joints (see Figure 1a). 'Gaioleiro' buildings simplified the construction by having the pavement beam directly supported on the masonry wall or nailed to an embedded beam (see Figure 1b). However, carpentry joints were entirely omitted. With the increased use of iron in construction, the connections began to incorporate iron cantilever beams and other connectors as support members (Appleton, 2005). In spite of the details of each type of connection, they all rely on friction, adhesion, and shear resistance of masonry to assure continuity of load transmission.



(a) (b)

Figure 1 Detail of a wall-to-floor connection: (a) 'Gaioleiro' building; (b) Late 'Pombalino'

The importance of timber in construction decreased from 'Pombalino' to 'Gaioleiro' buildings, reducing its structural use exclusively to pavement beams. The type of timber used at the time is much better characterized than the masonry because it was the main material of the seismic resistant structure. According to Appleton (2005), *Pinus Pinaster* and *Pinus Silvestris* were the most common species used in timber elements of 'Gaioleiro' buildings. These species were also applied in 'Pombalino' buildings, meaning the decrease in quality was due to defects like knots, cracks, deviation of the grain, and general state of conservation.

Stone masonry was mainly used in the confining walls of each building (external, party and light shaft walls) but it could also work as infill of the half-timbered walls. As shown in Figure 2a, the rubble masonry of the walls is composed by irregular limestone units of different sizes and bound together by a poor mortar to form an unorganized mixture. These walls were hand built with the units placed to as much as possible avoid voids with mortar. The mortar resulted from the mixture of air lime and sand, usually in the proportion of 1:2, but could take other ratios like 1:2.5, 2:5 or 5:9. Existing material descriptions specify that the sand should be of good quality, without clay and from a specific place of pine trees, probably referring to Leiria. They also prescribed that the stone should be soft (like limestone), of good quality, and should come from the nearby regions of Monsanto or Sacavém.

(a)

(b)

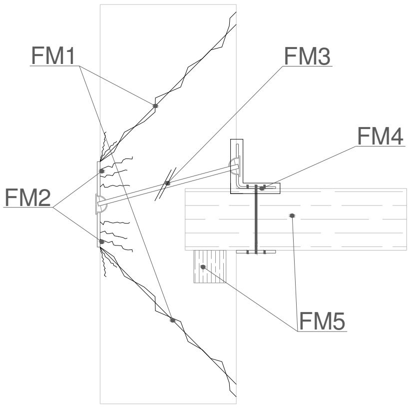
Figure 2 Masonry wall: (a) from a 'Pombalino' building; (b) detail of the connection built in the laboratory

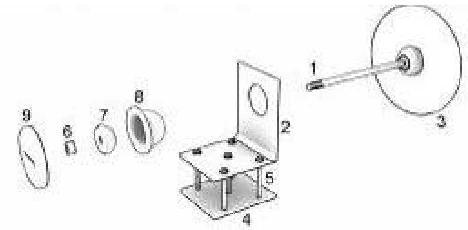
2. TEST SET-UP AND PROCEDURE

The specimens consist of a masonry wall with a pavement beam placed perpendicular to the wall and nailed to an embedded beam (see Figure 2b). The typologies and materials presented previously guided the entire design process of the specimens. Of the three specimens built to represent wall-to-floor (WF) connections, two were strengthened (A) and one was left unstrengthened (U). The strengthening solution is a tie rod anchoring the wall (anchor plate) to the pavement beam through a steel angle, as shown in Figure 3b. The particularity of this strengthening resides on the use of two hinges, one at each end of the tie rod, which helps the application of the tie rod with an angle. In this case, it was chosen a 15° angle. As part of the strengthening solution, the timber beam was confined with a GFRP sheet.

Taking into consideration existent literature, the strengthening solution, laboratory limitations, and possible failure modes, it was possible to define the dimensions of the specimens and the stress level to be applied. The specimens have a rectangular shape and the dimensions are: 2000×400×1600 mm^3 (specimens WT.40.A.1 and WT.40.U.1) and 2000×600×1600mm^3 (specimen WT.60.A.1). The thicknesses of 400 mm and 600 mm were chosen to represent values found on last floor and 1st floor walls of 'Gaioleiro' buildings, respectively. Thus, distinct levels of compressive stress due to permanent vertical (service) loads were applied during the tests. Those two levels are: 0.2 MPa (400 mm) and 0.5 MPa (600 mm).

The following failure modes were predicted (see Figure 3a): pull-out cone formation in the masonry (FM1), crushing of masonry under the anchor plate (FM2), failing of the steel tie rod (FM3), failing of the connection between the steel angle and the pavement beam (FM4), and ripping of timber elements (FM5). FM3 accounts for problems related to the tie rod or the steel nuts. Yielding of the tie rod was deliberately prevented by using a high grade of steel (8.8) and a ϕ16 diameter bar. When it comes to FM4, this failure mode considers crushing of the pavement beam due to the steel angle or the bolts, shear failure of the bolts and bending of the steel anchor.





(a) (b)

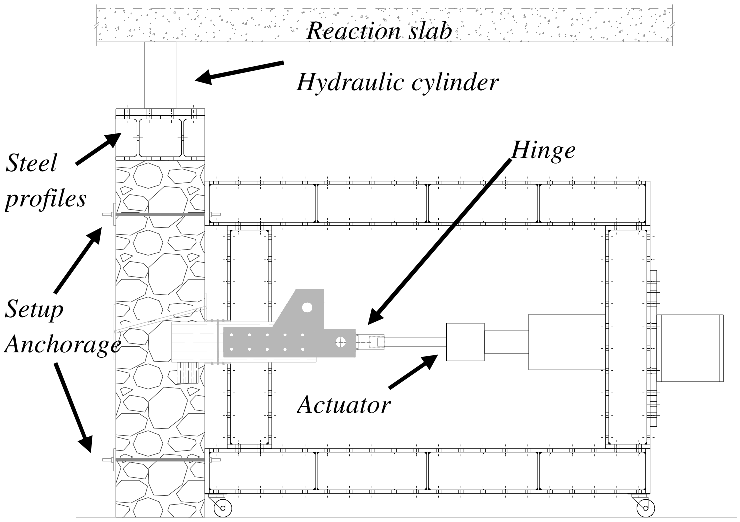
Figure 3 Wall-to-floor connection: (a) Expected failure modes for wall-to-floor connections; (b) adopted strengthening solution

Rubble masonry walls were hand constructed by professional masons, using limestone with a maximum dimension of 200 mm and at most 50 mm joints. A timber beam of 95×95 mm^2 was built in along the wall ('frechal') and connected 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 2b). In total 3 specimens were built, which gives a total of 6 tests.

A set of LVDTs was distributed on the wall and strain gauges were applied on the tie rod to control the formation, behaviour, and influence of the different failure modes. LVDTs measured out-of-plane displacements. Distribution of LVDTs contemplated especially FM1, measuring points following a vertical and horizontal alignments centred on the strengthening.

Considering laboratory limitations in terms of space as well as the size of specimens, it was possible to develop an auto-balanced set-up capable of redirecting the pull-out force back to the specimen, as shown in Figure 4. The pull-out load, which intended to recreate the main seismic action, was applied perpendicular to the wall in order to simulate a horizontal seismic force. A hinge was used between the actuator and the specimen to accommodate small rotations. As previously stated, a distributed vertical load was applied on the top of the wall to simulate the effect of permanent loads on the structure. This was achieved by placing HE200B steel profiles on top of the wall, which distributed the load provided by four hydraulic cylinders compressed against a reaction slab (see Figure 4). The distributed vertical load was kept constant during the entire test by using a manual control. A metallic clamp was designed for this connection, rigid enough to apply the force to the specimen without interfering on the test results.

The horizontal load was applied directly on the timber pavement beam. The monotonic tests were carried out under displacement control at a rate of 10 μm/s. The stopping criteria adopted were: a 50% decrease in load, a maximum displacement of 150 mm, which is the anchorage length of the pavement beam inside the wall or a propagation of cracks beyond the expected area of damage.



(a)

(b)

Figure 4 Set-up for the pull-out test: (a) sketch with the main components; (b) photograph taken during WF.40.A.1A test

2. MATERIALS AND SAMPLES

Experimental tests and subsequent numerical modelling require extensive material characterization, since material properties have a major influence on the behaviour of specimens and thus on the results. To date, only mortar and masonry samples have the compressive strength and elasticity modulus characterized, based on EN 1015-11 and EN 1052-1 specifications, respectively.

Cylindrical mortar samples were collected during the construction of the specimens and tested at the ages of 28 days and 90 days. The results of the compressive strength tests performed on mortar specimens are presented in Table 1 for 28 days and 90 days.

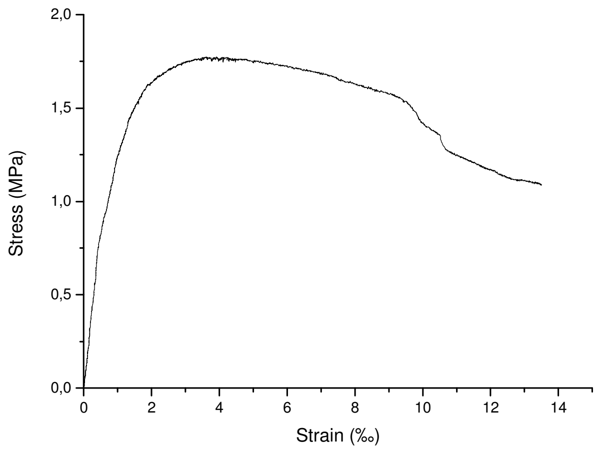
Table 1 Results of the compressive strength of the mortar tested at 28 and 90 days

Specimens	$f_{c\,(28\ days)}$ **(MPa)**	$f_{c\,(90\ days)}$ **(MPa)**
WF.40.A.1	1.24	1.27
WF.60.A.1	1.24	1.27
WF.40.U.1	1.59	1.51
Average (MPa)	1.36	1.35
CoV(%)	14.9	10.1

From a parametric study performed on mortar ratios before the construction of the walls, the average compressive strength at 28 days obtained for the chosen ratio was 1.27 MPa. The value obtained for the samples is only 7% higher, which confirms the rigour applied on the construction of the walls. It also shows that the compressive strength is hardly affected by the testing age, thus allowing the testing of specimens at different ages without major consequences in terms of mortar strength.

The masonry prisms were defined according to EN 1052-1, with dimensions of $0.40\times0.50\times0.80\ m^3$. Initially, a monotonic test was performed, which was followed by a cyclic test on a similar specimen built at the same time. Vertical cracks and lateral separation of the prism were observed on both specimens, as it was expected. The average compressive strength of the 2 prisms was around 1.60 MPa. Values found in literature range between 0.50 and 1.50 MPa, thus placing the tested specimens slightly above the range. Prisms reached an elastic modulus approximately equal to 1000 MPa. The stress-strain diagram of one of the prisms tested and a detail on the position of the

LVDTs is presented in Figure 5.



(a)

(b)

Figure 5 Compressive test: (a) Stress-strain diagram of a masonry prism; (b) instrumentation detail.

3. RESULTS

To date, five of the six tests were already performed. 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 force-displacement curve relative to out-of-plane displacement of the pavement beam as well as the description of the failure mode is presented for each specimen on Table 2. 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).

The test performed on WF.40.U.1A showed the beam sliding out of the wall, reaching the peak force around 10 kN for 25 mm of displacement (see Table 2a). This length is coincident with the anchorage length of the nails connecting the timber beams. As the beam slid, it also started rotating taking more advantage of friction. For the first strengthened specimen, WT.40.A.1A, 4 ϕ6 bolts were used to connect the steel angle to the pavement beam, which failed in shear as can be seen in Table 2b. 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.

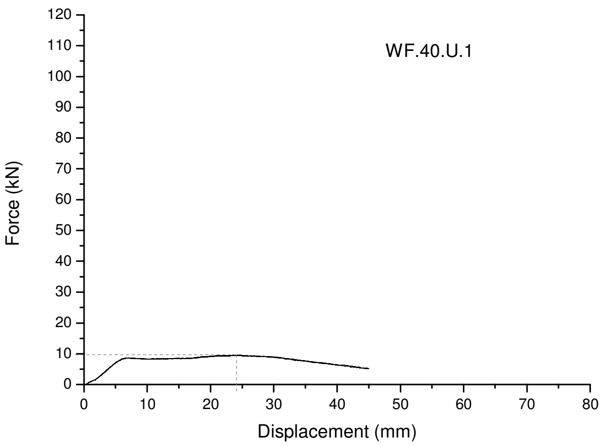
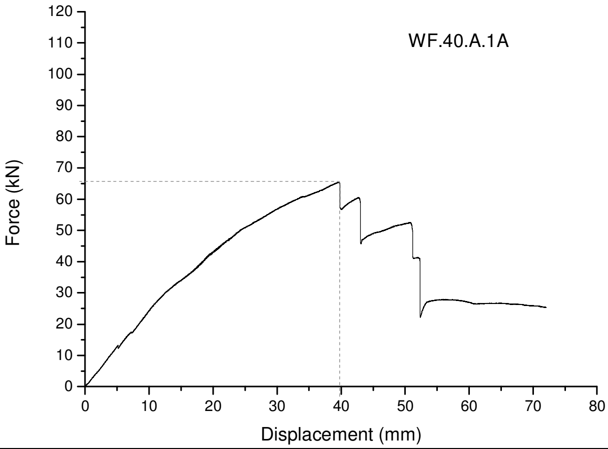
For the following specimen, WF.40.A.1B, the ϕ6 bolts were substituted by ϕ10, to prevent the failure of the bolts. Consequently, another failure mode was found for a higher load – this one associated with the tie rod. At each end of the tie rod only one nut was used to tighten it onto the hinges, which surprisingly proved to be insufficient when tested with the large diameter bolts. As shown in Table 2c, 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.

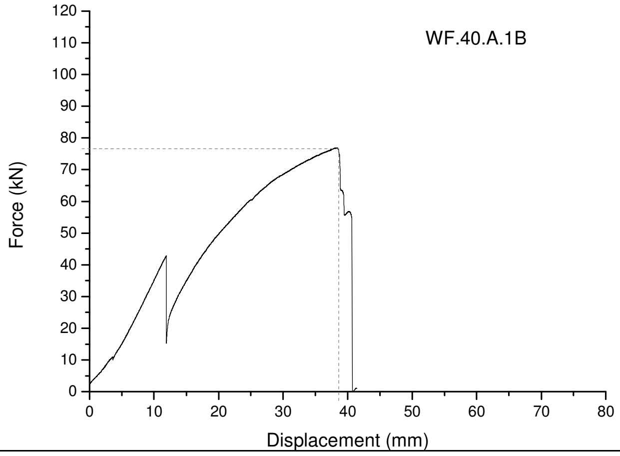
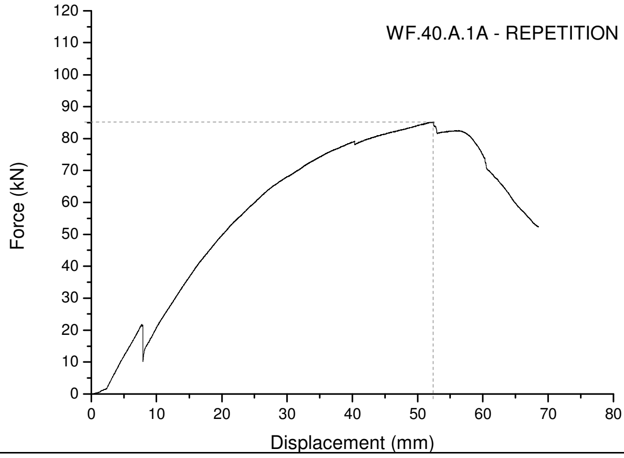
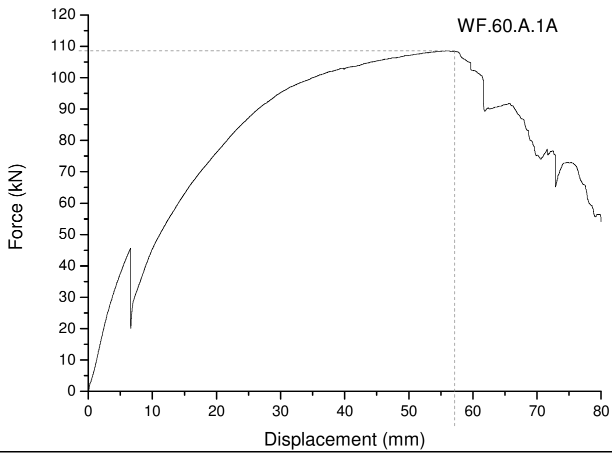
Since small displacements were measured during the test of WF.40.A.1A and no visible damage was observed on the wall and embedded beam, it was performed another test on the same location of the specimen. Only the timber pavement beam was replaced and the strengthening reapplied (WF.40.A.1C). The only disregarded detail was the nails connecting both timber beams, which was considered acceptable 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 Table 2d, 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.

After obtaining the formation of the pull-out cone for a 40 cm thick specimen, it was decided to replicate the same strengthening details on a 60 cm specimen. As depicted in Table 2e, 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 compressive stress of the wall (0.5 MPa) led the embedded beam to rip along the nails plan.

It was observed that at least three of the tests 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.

Table 2 Observed failure modes

Specimen	Failure mode	Force-displacement curve	Photographic survey
(a) WF.40.U.1A	FM5 – Pull-out of the nails connecting the pavement beam and the embedded beam.		
(b) WF.40.A.1A	FM4 – Bolts connecting the steel angle and the timber beam crushed the timber beam, until they bent and failed due to shear.		

(c) WF.40.A.1B	FM3 – The nut stripped the threads of the steel tie rod.		
(d) WF.40.A.1C	FM1 – The masonry wall failed forming the pull-out cone.		
(e) WF.60.A.1A	FM5 – Sliding and rotation of the pavement beam crushed the embedded beam, ripping it from one end to another.		

5. ANALYSIS OF THE FAILURE MODES

Knowing the different failure modes and what can cause their appearance is very important when designing strengthening. Each failure mode obtained experimentally gave a threshold of the resistance (see Figure 6) and also showed what kind of damage to expect. In Figure 6 are presented the maximum pull-out forces obtained for 40 cm walls versus the respective displacement. The formation of the pull-out cone requires the highest force, but at the same time is the failure mode causing the most damage to the masonry wall. Retrofitting of masonry walls requires more expertise and is more time consuming than simply replacing the strengthening. So one must consider if is more appropriate to favour other failure mode with less impact on the wall when proposing strengthening connections.

By varying the diameter of the tie rod and the steel grade is possible to place the failure within or above the failure modes found. In this case, it was used ϕ16 bars with 8.8 steel grade, which places the limit beyond FM1, at 1.89 (160.85kN). This is a good way of controlling the failure of the connection, since the behaviour of steel is well known and it also presents a ductile behaviour.

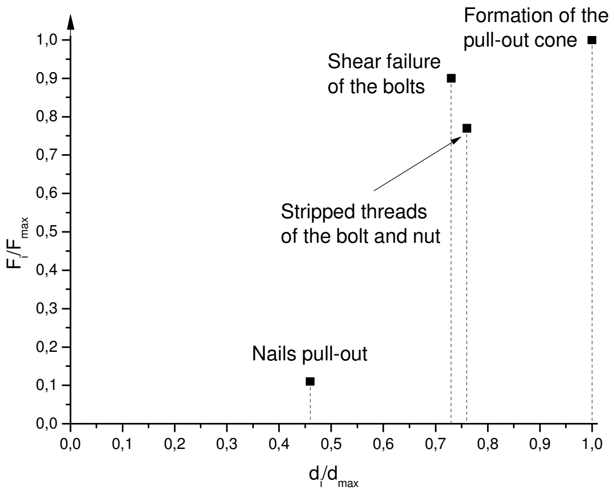


Figure 6 Relationship between the different failure modes

6. CONCLUSIONS

The presented research adds critical experimental information about wall-to-floor 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.

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 walls and others.

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