Developing a seismic retrofitting solution for wall-to-floor connections of URM buildings with wood diaphragms

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ABSTRACT: Out-of-plane collapse of masonry walls in unreinforced masonry (URM) buildings has been identified as a major local failure mechanism affecting the global stability of a structure. Continuity between structural elements maintains proper load paths during seismic action, discouraging the formation of out-of-plane mechanisms. This is a key factor for an engineer to consider when trying to fully utilize the in-plane capacity of a masonry wall. This paper focuses on the seismic retrofit of wall-to-floor connections of URM buildings with wood diaphragm floors. The solution consists of a tie rod bolted to a timber floor joist and anchored to the masonry wall using an anchor plate. Several cyclic pull-out tests were carried out to characterize the behaviour of the strengthened connection so that an 'engineered' strengthening can be developed. The resultant behaviour has the contribution of different failures from yielding of the steel components to masonry cone breakout. The strengthen connection presents a higher increase in tensile capacity, displacements, and consequently energy dissipation.

Keywords: Connections, out-of-plane collapse, seismic retrofit, URM, wood diaphragm

NOTATION

$s_{j/w,max}$ slip joist/wall maximum displacement;

1 INTRODUCTION

Unreinforced masonry (URM) buildings have been the core of many studies, since URM is a generalized building typology all over the world and its seismic vulnerability is well documented and recognized [1]. Its inability to sustain large deformations without severe strength loss and damage, raises concerns regarding life safety, economic loss or even irrecoverable architectural and cultural damage. Nonetheless, in recent seismic events has been observed that URM buildings perform better than predicted, being able to sustain damage much after their maximum strength capacity is reached [2]. In-plane mechanisms of walls, like rocking, are displacement-controlled which impose more of a ductile and more desirable behaviour to the structure. In order to take advantage of the in-plane capacity of the masonry walls, it is necessary to safeguard the out-of-plane mechanisms of walls. In spite of being local mechanisms, out-of-plane damage culminating in partial or complete collapse of

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walls can compromise the stability of the entire structure. Several studies reported poor connections between structural members, like the masonry walls and the wood diaphragms, as being one of the main causes of walls' overturning and collapse of floors [3; 4]. In Christchurch 2011 earthquake it was observed that strengthened wall-to-floor connections prevented effectively out-of-plane collapses, allowing the development of a global response of the structure [5]. Very few research studies, experimental or numerical, have focused on the behaviour of connections or appropriate strengthening solutions [6; 7] Therefore, an experimental campaign was carried out in order to characterize and develop an 'engineered' strengthening for wall-to-floor connections. The present paper focus on the analysis of the quasi-static cyclic tests of the strengthened connections, since the monotonic behaviour of unstrengthened and strengthened connections, was already discussed by Moreira et al. [8].

Commonly in URM buildings, wood diaphragms and masonry walls are the main structural members that resist vertical and lateral loads. Therefore, is crucial to ensure proper load transmission between these two elements. There's a great variety of wall-to-floor connections, but some are common to URM buildings. In one of the simplest typologies of wall-to-floor connections, the wood floor joist rests on a pocket hole in the masonry wall, leaving a gap between the joist's end and the end of the pocket hole. Lyn & LaFave [7] and Peralta et al. [9] studied this type of connections in typical pre-1950s brick buildings, in the United States, where the joists ends were cut diagonally (fire cut) and rested in pocket holes, in the masonry wall. With a small bearing length of approximately 7.6 cm at the support, the joists were anchored at every fourth joist with iron straps. These were common for joists perpendicular to the wall but not for parallel ones.

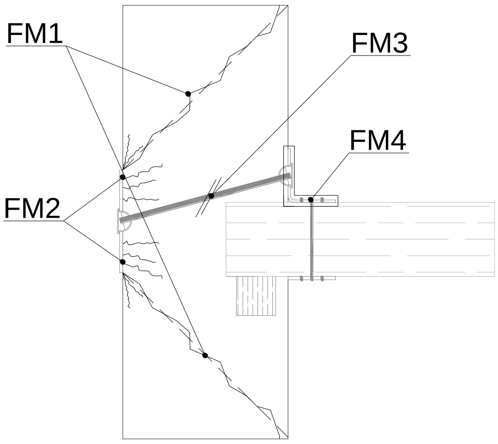
In Portugal, a similar connection to the one previously described but instead the wood joist is nailed to a wood wall-plate along the masonry wall or for better quality buildings, a cross-lap joint is executed between timber elements. Usually this wall-plate is embedded 5 cm relatively to the interior face of the wall. This last type of connection can be found in two typologies of buildings built towards the end of the 19th century, commonly called Late 'Pombalino' and 'Gaioleiro' buildings. Both resulted from the deterioration of construction techniques applied on the construction of the 'Pombalino' typology [10].

These last buildings were the engineering answer to the destruction of downtown Lisbon by the 1755 earthquake. The solution consisted of a mixed construction of masonry and timber, where a three-dimensional timber frame ('gaiola') was encased on external masonry walls. The main elements of this timber 'cage' were the half-timber walls, with a frame composed by St. Andrews crosses with masonry infill that could be rubble stone or solid brick. The wall-to-floor connections in 'Pombalino' buildings were more complex because they were part of the three-dimensional timber frame. Therefore, the timber floor joist would be connected to a top and bottom timber wall-plates embedded in the masonry wall, through carefully done cross-lap joints and the use of 8 cm to 30 cm nails [10]. While for 'Pombalino' buildings was common practice the use of metal straps nailed to the timber joists and going through or into the wall, for the other two styles the usage of these connectors was occasional. These iron straps were placed on joists both perpendicular and parallel to the wall [11]. Mascarenhas [11] states that in most cases the timber floor joists were perpendicular to the façade, and contrary to what one might think the iron straps would go deeper into the party walls than the façades. At the beginning of the 20th century, in Portugal, wrought iron started to have a more general use in buildings' construction. 'Gaioleiro' buildings started having columns, beams, stairways and wall-to-floor connections in wrought iron like wrought iron corbels [12]. With time, wrought iron was substituted by steel [13], originating new forms of connecting the wood diaphragms to the masonry walls. Some examples are the steel hangers, continuous rods between timber floor joists and wall, embedded in both elements (glued-in or screwed) and solutions like the one proposed in this paper. From the experimental campaign, was possible to identify different failure modes, quantify the maximum pull-out force and energy dissipation, and obtain the hysteretic curves of the strengthened connections.

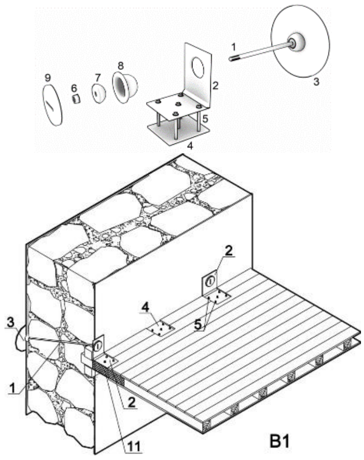
2 TEST SET-UP

The specimens represent a wood floor joist nailed to a wood wall-plate type of connection, as described previously (see Figure 1a). The strengthening solution developed by the company

Monumenta, Ltd. consists of a steel angle bolted to the wood joist that is anchored to the masonry wall by a steel rod with anchor plate system, as shown in Figure 1b. The particularity of this strengthening resides on the use of two hinges, one at each end of the steel rod, which enables the steel tie to be placed at an angle, rather than placed horizontally. In this case, it was chosen a 15° angle with the horizontal plane. The anchor plate applied on the 0.40 m walls had a squared shape and the dimensions of 0.175 × 0.175 × 0.006 m^3. After conducting the tests on the 0.40 m walls, one decided to increase the thickness of the anchor plate to 0.020 m, to discourage bending. In addition, as part of the strengthening solution, the timber beam was confined with a GFRP sheet.
The following failure modes were predicted (see Figure 1a): masonry cone breakout (FM1), crushing of masonry under the anchor plate (FM2), yielding of the steel tie (FM3) and failing of the connection between the steel angle and the wood floor joist (FM4). To prevent premature yielding of the steel rod, ϕ16 diameter bars of high-grade steel (class 8.8) were applied. FM4 is a very complex failure mode because is the result of combined effects that occur at the bolted connection. This failure mode comprises crushing of the wood floor joist, bending and shear failure of the bolts, and yielding of the steel angle.



(a)



(b)

Figure 1. Wall-to-floor connection: (a) Expected failure modes of strengthened wall-to-floor connections; (b) adopted strengthening solution [14].

Taking into consideration the existent literature, the strengthening solution, laboratory limitations, and possible failure modes, was possible to define the dimensions of the specimens and the stress level applied. Two different wall thicknesses, 0.4 m and 0.6 m, were chosen in order to represent an upper floor (4th floor) and a ground floor, respectively. Thus, two distinct levels of compressive stress caused by quasi-permanent service loads were applied during the tests. Those two levels are 0.2 MPa (0.4 m) and 0.4 MPa (0.6 m). The length and height defined for the specimens are the same for both thicknesses: 2.0 m and 1.6 m, respectively. Rubble masonry walls were hand constructed by professional masons, using limestone with a maximum dimension of 0.2 m and at most 0.05 m joints. In each wall were built two connections. Each connection has a wood wall-plate of 0.095 × 0.095 × 1.000 m^3 embedded in the masonry wall, at 0.03 m of its front face, and a wood floor joists of 0.13 × 0.18 m^2 placed perpendicularly to the wall-plate. The wood floor joist is 0.15 m into the wall and the nails are located at approximately 0.08 m of the end of the joist. In total two specimens were built for each thickness, which gives a total of 8 tests.
A set of 15 LVDTs were distributed on the wall, joist, wall-plate, steel rod and anchor plate. Two strain gauges were applied at mid-length of the steel tie to assess load transmission. LVDTs distribution took into consideration the failure modes predicted and described previously, especially FM1, and also the need to characterize the relative displacement between wood floor joist and masonry wall front face (see Figure 2a).
Considering 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 2b. 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 the quasi-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 2). The distributed vertical load was kept constant during the entire test, 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 final apparatus, as presented in Figure 2b, includes a vertical support of the joist, equipped with a load cell, which monitored the vertical reaction throughout the test. Only cyclic tests were performed with this support, in an attempt to control the rotations of the wood floor joist.

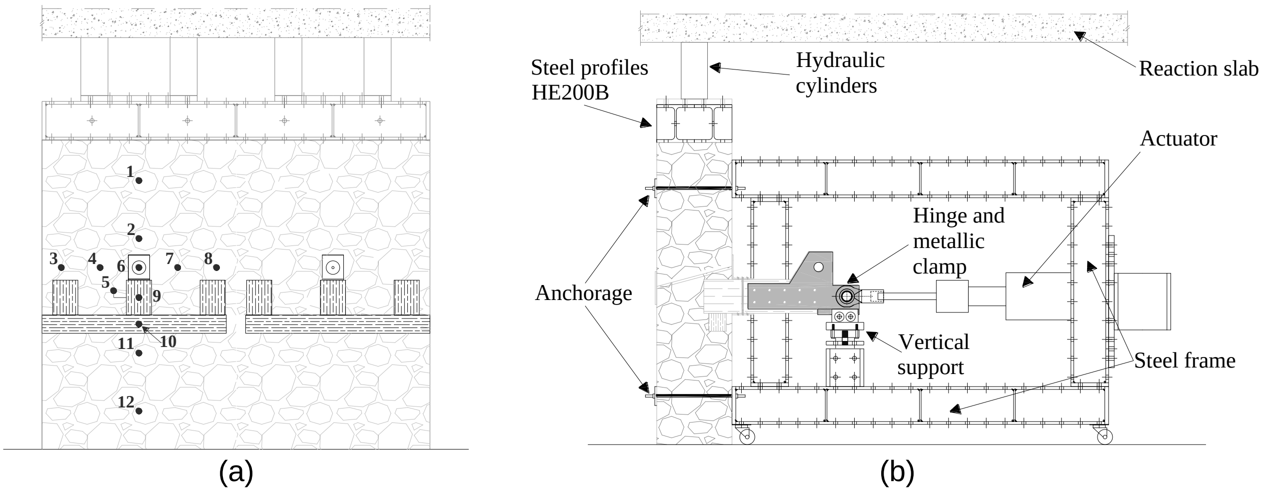


Figure 2. Pull-out test set-up: (a) specimen and location of the instrumentation; (b) apparatus.

The horizontal load was applied directly on the wood floor joist. The monotonic tests were carried out under displacement control at a rate of 10 μm/s. From these tests, it was possible to define a cyclic procedure with 10 steps, from 10 mm up to 100 mm. Since no pounding effect of the wood joists on the wall was being studied, the unloading branches stopped at different displacements to prevent the development of high compressive forces on the specimen. The procedure applied on the tests also included a monotonic branch at the end of the cycles, which was carried out to obtain failure when necessary. A maximum amplitude of 100 mm was established so that no damage could propagate to areas of surrounding tests. The range of velocities was between 25 μm/s to 150 μm/s, respecting always a minimum of 120 s for duration of each loading or unloading branch.
The stopping criteria adopted were: completion of the procedure, propagation of cracks beyond the expected area of damage or compromise of the test apparatus.

3 RESULTS

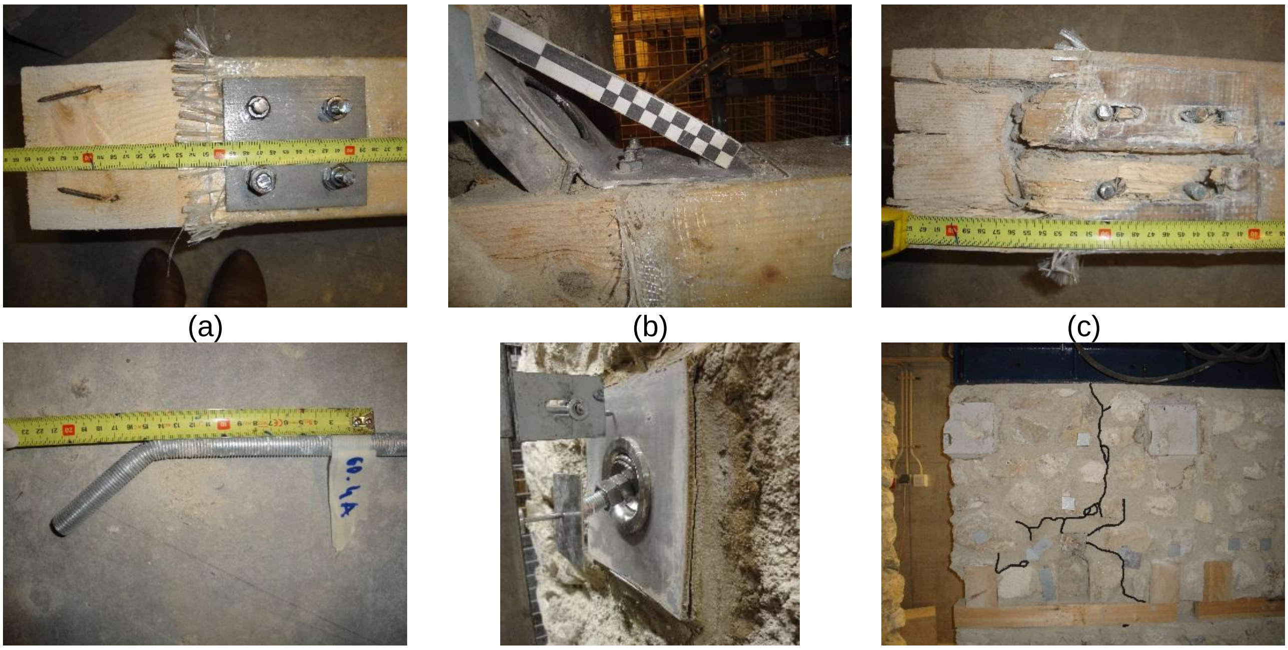
Eight quasi-static cyclic pull-out tests were carried out, four for each thickness. Maximum pull-out force and corresponding slip joist/wall displacement, $s_{j/w,max}$, are presented in Table 1. The slip joist/wall refers to the displacements measure by the Linear Variable Displacement Transducer (LVDT) on position 5 (see Figure 2), which as the name suggests describes the relative displacement between wall and floor joist. The averages maximum pull-out forces for both thicknesses are very close in value, 96.8 kN and 91.9 kN, and their coefficients of variation (CoV) are both below 10% (see Table 1). In spite of the different failure modes observed, the tensile capacity of the strengthened connections doesn't vary too much. The higher value of the tensile capacity obtained for the 40 cm walls is possible related to the fact that less damage occurs during these tests.
Different types of damage were observed during tests and were common to both thicknesses of walls, as can be seen in Figure 3. Damage on the bolted connection and yielding of steel components are very ductile modes that contribute to the large joist/wall slip.

Table 1. Maximum pull-out force and displacement

Specimen	Pull-out force [kN]	$s_{j/w,max}$ [mm]
WF.40.A.3A	93.09	81.08
WF.40.A.3B	105.38	60.11
WF.40.A.4A	94.50	84.32
WF.40.A.4B	94.07	58.75
Average	96.76	71.07
CoV (%)	5.97	19.01
WF.60.A.2B	92.42	60.31
WF.60.A.3A	82.67	37.57
WF.60.A.3B	102.44	104.19
WF.60.A.4B	90.02	59.19
Average	91.89	65.32
CoV (%)	8.89	42.79

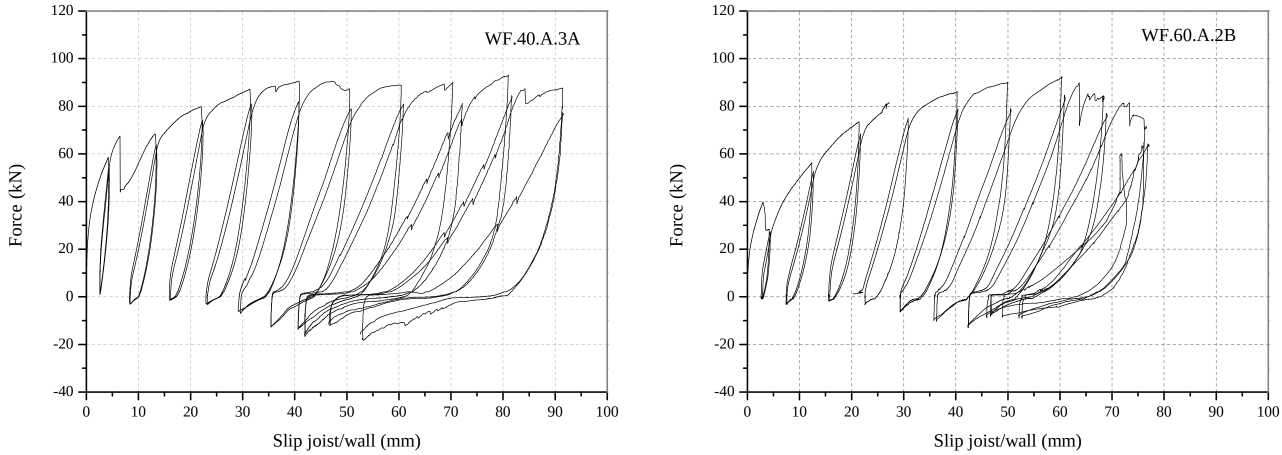
For the for 40 cm walls, failure in all specimens resulted from the combination of masonry cone breakout with failure of the bolted connection (FM1 + FM4), resulting in great similarity of the hysteretic and envelope curves (see Figure 4a and Figure 5a). The 60 cm walls presented mainly failure modes FM3 and FM4 but with similar hysteresis loops and envelope curves until failure (see Figure 4b and Figure 5b). Specimens WF.60.A.3 and WF.60.A.4B had brittle failure modes, bending of the wood joist at the bolted connection, which broke completely, and failure of the steel rod (see Figure 3d and Figure 5b). Specimens WF.60.A.2B and WF.60.A.3B failed by ripping of the wood joist at the bolted connection. Envelope curves from tests on the 40 cm walls show the formation of a plateau at maximum load as result of sliding of the wood joist relatively to the wall (see Figure 5a).

As seen in Figure 4, typical force-displacement curves show great similarity regarding the shape of the loops, during the pre-peak phase, regardless of the failure mode. This effect is due to the initial damaging mechanisms being governed by the single shear bolted connection between the timber joist and the steel angle. These mechanisms are crushing of the timber joist and shear failure of the bolts. The hysteretic behaviour encloses loss of strength between cycles, stiffness degradation and pinching. As one can see, compression forces associated with reversing the cycle are small, as result of the imposed test procedure. In all tests, there is a loss of force in the range of 20 kN to 70 kN because of the detachment of the steel angle from the timber joist.



(a) (b) (c) (d) (e) (f)

Figure 3. Different types of damage: (a) pulling of the nails (b) bending of the steel angle; (c) crushing of timber and shear damage of the bolts; (d) bending of the steel rod; (e) crushing of the masonry under the anchor plate; (f) masonry cone breakout.



(a) (b)

Figure 4. Typical cyclic force-displacement curve of strengthened connections: (a) 40 cm walls; (b) 60 cm walls.

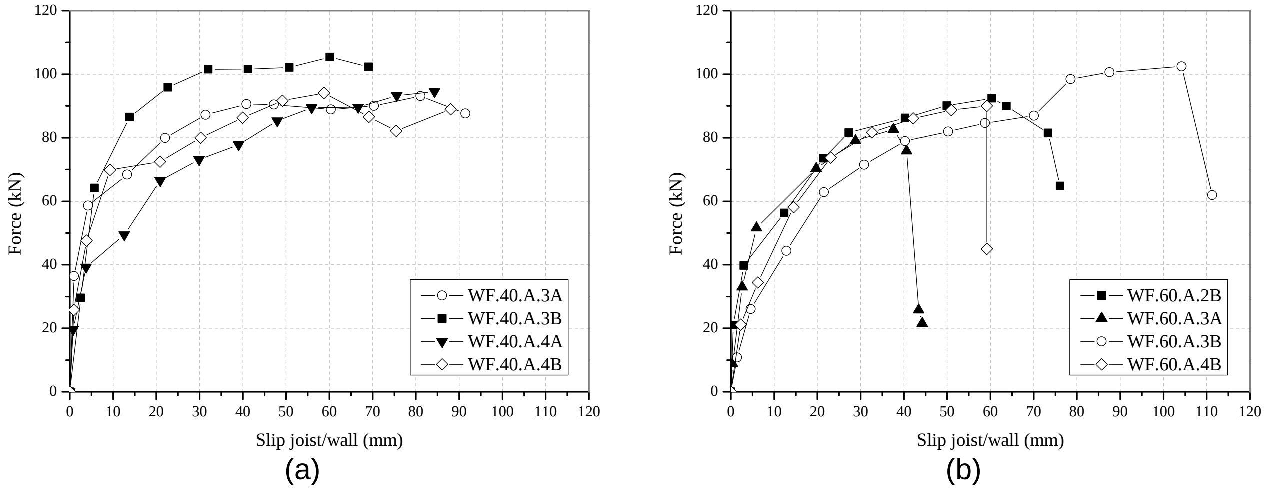


Figure 5. Envelope curves: (a) 40 cm walls; (b) 60 cm walls.

In Figure 6b one can observe the great difference between ductile and brittle failure modes. Specimens WF.60.A.3 and WF.60.A.4B have lower total energies compared to ductile failures. Tests from both 40 cm and 60 cm walls, dissipated most of their energy through the ripping of the wood joists, consequently there is not a big difference between them. The high increase in total energy on specimen WF.60.A.3B was the result of a monotonic increase on load, after the completion of all cyclic steps.

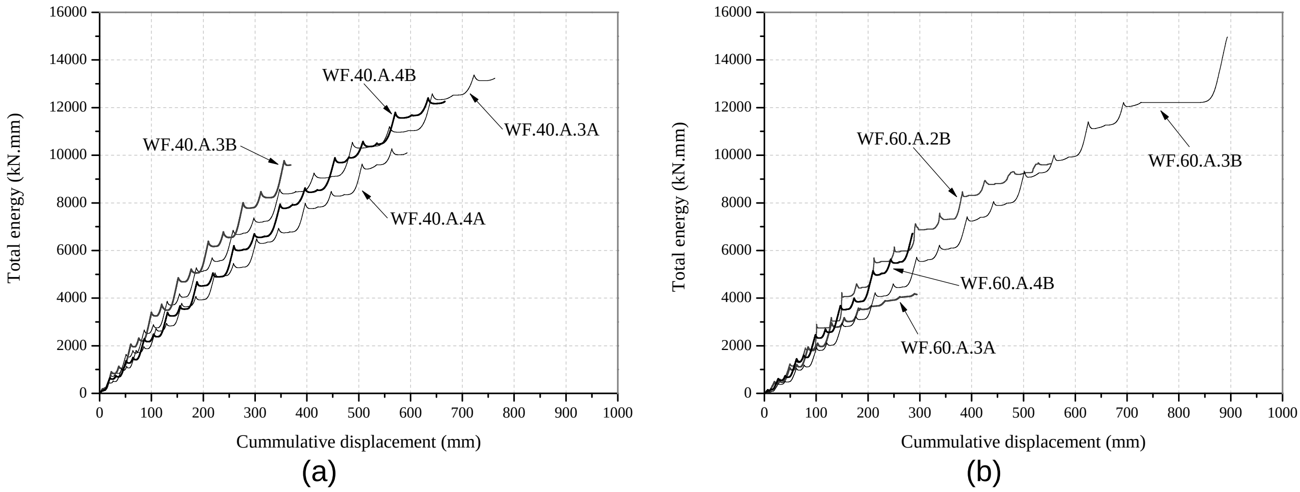


Figure 6. Total energy: (a) 40 cm walls; (b) 60 cm walls.

4 CONCLUSIONS

The pull-out tests were carried out successfully and it was possible to characterize the cyclic behaviour of the strengthened wall-to-floor connections under study. The failure modes obtained were within the behaviour predicted prior to the experimental campaign. Combination of masonry cone breakout with failure of the bolted connection was observed for all tests on the 40 cm walls, while failure of the bolted connection and yielding of the steel rod occurred in tests on the 60 cm walls.

The high ductility of the strengthening is associated with the combined behaviour of the wood joist, the bolted connection and the steel angle, which display high deformations.

A deeper analysis of the results has to be performed, in order to understand the different contributions of the possible failure modes, and the influence of the thickness of the wall and vertical compression levels.

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