# Performance evaluation of traditional timber joints under cyclic loading and their influence on the seismic response of timber frame structures

Elisa Poletti[a,*], Graça Vasconcelos[a], Jorge M. Branco[a], **Aikaterini M. Koukouviki[b]**

[a]ISISE, Department of Civil Engineering, University of Minho, Campus de Azurém, 4800-058, Guimarães, Portugal

[b]**Civil Engineer**

[*]corresponding author: elisapoletti@gmail.com, Tel.: +351 253 510 200, Fax: +351 253 510 217

**Abstract**

Timber joints represent the governing part of a timber structure, particularly when assessing its seismic response. In order to better assess the seismic capacity of traditional timber frame structures, **particularly timber-framed shear walls**, pull-out and in-plane cyclic tests were carried out on their joints (half-lap joints). **The aim is to better understand the influence of the joints on the walls and their influence on failure mechanisms and capacity**.

Their seismic characterisation was obtained via the analysis of the hysteretic behaviour and dissipative capacity of both unreinforced and retrofitted joints (using self-tapping screws, steel plates and GFRP sheets).

Results show that all strengthening techniques were able to improve the dissipative and load-bearing capacity, but care should be taken into not over-stiffening the joints, as it would lead to an overly rigid structure.

**Highlights:**

- Joints govern the behaviour of a timber structures

- Joints constitute the dissipative mechanism of timber structures for seismic events
- The quality of the joint (presence of gaps) greatly alters the response of a joint
- Strengthened joints present significantly higher stiffness and dissipative capacity

***Keywords***: Half-lap joint, seismic retrofitting, self-tapping screws, GFRP, NSM, steel plates, dissipative capacity

# 1 Introduction

Timber frame construction is a popular constructive technique that is **typical of** many historic city centres worldwide. They became popular both for their cheap and easy construction in areas where wood was abundant (North America, Scandinavia, UK) and for their good seismic performance (e.g. Portugal, Italy, Greece, Turkey, Peru), as timber frame walls act as shear walls.
While they are recognised as an important world cultural heritage, only recently some restoration efforts have been made and in general many buildings have been abandoned for decades. Another issue that concerns these structures is that modifications have been made in many cases without taking into account the new structural response of the structure and without considering concepts such as reversibility or re-treatability.

Different studies (Meireles et al. 2012; Poletti and Vasconcelos 2015; Ruggieri et al. 2015; Vieux-Champagne et al. 2014) have shown that the response of timber frame structures depends essentially on the resistance of the connections, since they represent the dissipative mechanism of the structure. Concerning the global rigidity of the structure it is fundamental to understand how the connections work, in particular the **relative movements of the components**.

Traditional timber joints are used in a great variety of timber structures and structural elements, from floors to walls to roofs. In literature, it is possible to find numerous experimental results on traditional timber connections. Various studies are available on bird's mouth connections, typically used for roofs (Branco 2008, Parisi and Piazza 2002, Parisi and Piazza 2015) and on mortice and tenon joints regarding pull-out, bending and shear tests (Descamps et al. 2006, Koch et al. 2013), as well as on dovetail joints with and without pegs (Sobra et al. 2015) and dowel-type joints (Xu et al. 2009). Moreover, studies exist on the characterisation of specific traditional joints, e.g. Taiwanese Nuki joints and Dou–Gon joints (Chang et al. 2006; D'Ayala and Tsai 2008) **and** Japanese Kama Tsugi and Okkake DaisenTsugi joints (Ukyo et al. 2008).

The response of traditional timber joints depends greatly on compression and friction among their elements. Due to the production process, i.e. the manual work of the carpenter, there can be some irregularities and gaps which influence their performance, therefore commonly the contact between the members is improved by strengthening the connections with metal elements. Traditional timber connections rely for their performance mainly on notches, wedges, bearing faces, mortices, tenons and pegs, while metal fasteners are less **common**, though nails can be inserted to improve the connection's **performance**. On the other hand, metal fasteners constitute an important tool in rehabilitation works.

Interventions in timber frame buildings can be necessary due to different problems, e.g. decay as a consequence of poor maintenance, change in use and therefore need of additional strength, cracks **and** local failures. Many examples are available on restoration works **carried out** on traditional timber frame buildings, and in some cases the end result is the loss of the original structural system (Appleton 2003, Appleton and Domingos 2009, Cóias 2007, Tsakanika and Mouzakis 2010). While numerous studies are present on the reinforcement of joints for roofs and floors, **using either traditional techniques or dowel type connections** (Parisi and Piazza 2015, Dietsch and Brandner 2015) **or FRP materials (Schober et al. 2015),** little information is available for vertical elements (Chang 2015) and their connections in particular. In this paper, a contribution is given to the better understanding of the behaviour and the retrofitting of traditional connections for vertical elements subjected to seismic actions.

## 1.1 Timber frame walls general behaviour and the importance of their connections

As already mentioned, different studies showed that the seismic performance of traditional timber frame walls depends mainly on their connections. To experimentally study the seismic response of traditional Portuguese timber frame walls, typical of the so called Pombalino buildings (Cóias

2007), in-plane cyclic tests were performed on **full** scale specimens (Poletti and Vasconcelos 2015). Half-lap joints were used for the connections between posts and beams, while the diagonal bracing elements were simply nailed to the main frame. In general, the walls **displayed** a good capacity and ductility. Results greatly depended on the level of vertical pre-compression and on the presence of infill, which could alter the response of the wall from a shear one to a flexural one. **Figure 1 presents the hysteretic response of an unreinforced infill wall (half-timber wall) for two vertical pre-compressions, namely 25 kN and 50 kN (UIW25 and UIW50 respectively**). Damage **was** concentrated at the connections and uplifting of the non-continuous lower half-lap connections was severe, particularly for infill walls. **In particular, a lower vertical load led to higher uplifts for all bottom connections (BL=bottom left; BM=bottom middle; BR=bottom right).** For a full description of the experimental results, see Poletti and Vasconcelos (2015).

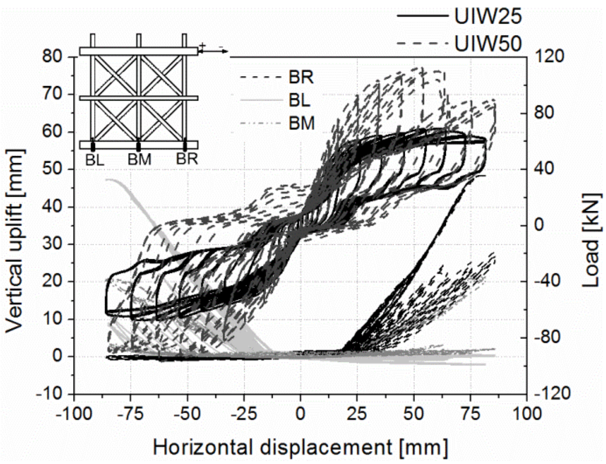



**Fig. 1** Results of in-plane cyclic tests performed on traditional timber frame walls: experimental results showing influence of vertical pre-compression

## 2 Experimental campaign on half-lap joints

An experimental campaign on traditional joints used in Portuguese timber frame walls was carried out in order to better study their behaviour, since they are the key elements of the walls. In order to do this, a significant connection of the wall tested was selected, namely the bottom half-lap joint which is a tee halving joint and therefore weaker than a cross halving joint. This choice was made due to the fact that during the tests on unreinforced walls, this joint **governed** the behaviour of the walls, as **its uplifting led to** the rocking movement of the wall (Poletti and Vasconcelos 2015).

To understand the response of these joints for vertical elements, the deformation patterns and the damage progress will be analysed in order to confirm the selection of the most appropriate retrofitting solutions which were previously adopted for the walls (Poletti et al. 2014; Poletti et al. 2015). This study will help fill the research gap currently present on retrofitting of traditional timber frame walls and their joints.

To perform this study, 14 specimens have been tested for the mechanical characterisation of traditional joint in timber frame walls. Pull-out and in-plane static cyclic tests have been performed. In the following section, details about the geometry of the specimens, the test setup and procedure and retrofitting techniques adopted are presented. Subsequently, the results obtained will be analysed in detail.

## 2.1 Specimens

The specimen selected has the same geometry of the bottom joint in the wall, see Fig. 2 (Poletti and Vasconcelos 2015). The influence of the diagonal bracing member was not considered for this study, but its effects should be studied. The bottom beam was anchored on both sides of the connection, as done in the walls, at the same distance from the connections that was used during the wall tests.

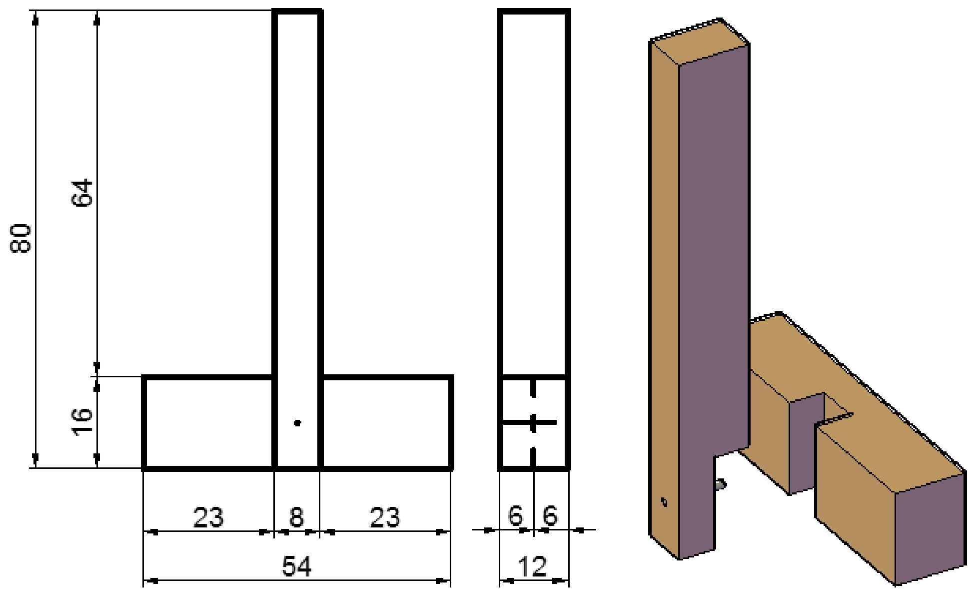



**Fig. 2** Geometry of specimens tested (dimensions in cm)

For simplicity purposes, the influence of infill was not taken into account in these tests, even though it has an important confining effect on the timber frame and adds stiffness and strength to the frame (Poletti and Vasconcelos 2015). The non-consideration of the infill represents the most unfavourable condition, since the connection is weaker without infill.

The specimens were built with the same type of wood **as** the walls, *Pinus pinaster*. **A wire nail (4.5 mm x 10 mm)** was inserted in the centre of the connection, similarly to what was done in the connections of the walls (Poletti and Vasconcelos 2015).

## 2.2 Set-up and test procedure

Two types of tests **were** performed on traditional joints: (1) pull-out tests and (2) in-plane static cyclic tests. This choice is justified by the fact that, during the tests performed on the walls there was a tendency for the bottom connections to uplift, particularly in infill walls (Poletti and Vasconcelos 2015). Therefore, it was decided to test the uplifting capacity of the connections in the unreinforced and retrofitted condition. To do this, the beam of the connection was anchored to a steel profile which was linked to the reaction floor. The post was pulled-out by means of a hydraulic actuator which was linked through a hinge at the top of the post by means of a U profile gripping the post with four 12mm rods (Fig.3). Notice that, in order to prevent failure at the top gripping device, GFRP sheets were glued to strengthen the zone.

**A total number of 4 specimens was used for these tests (Table 1); the tested specimens were later retrofitted (see section below) and tested again.**

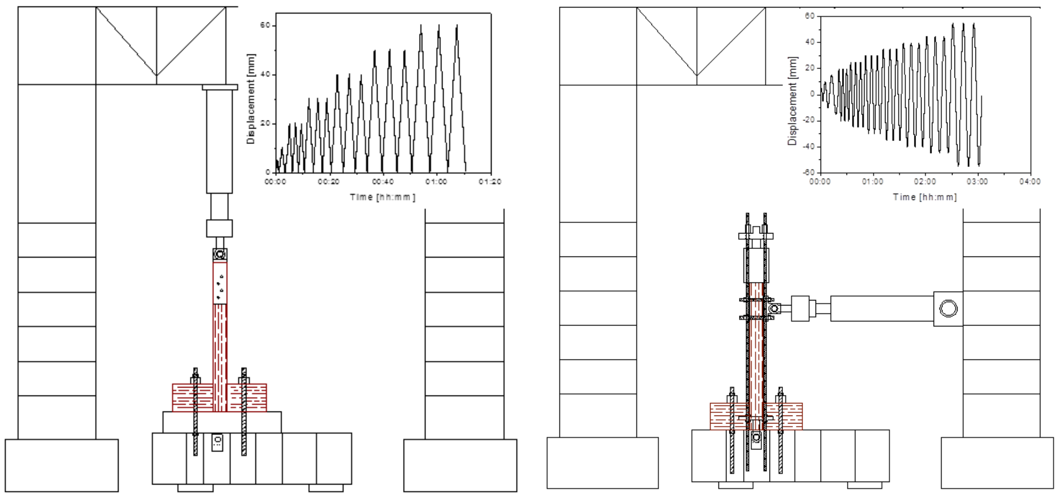



**Fig. 3** Test setup and procedures adopted: pull-out test (left) and in-plane test (right)

In order to compare the cyclic behaviour of the walls with the main characteristics of the mechanical behaviour of the connections, in-plane static cyclic tests have been performed on the connection, with a setup similar to that used for the walls (Poletti and Vasconcelos 2015), see Fig. 3. A constant vertical load was applied to the post by means of a hydraulic jack with the same system of hinges and rods **that guarantees that the jack follows the movement of the post**. Two vertical loads were considered during the tests, one of 25kN and one of 50kN. The horizontal load was applied by means of a hydraulic actuator with the aid of two 2-dimensional hinges in order to allow rotations during the test **(Fig. 3).**

**Table 1** Specimens adopted for each test and nomenclature

| TEST TYPE | **UNREINFORCED SPECIMENS** | **RETROFITTED SPECIMENS** | |
|---|---|---|---|
| **PULL-OUT TESTS** | MONO<br>URT1<br>URT2<br>URT3<br>URT4 | GFRP_T<br>SCREWS<br>STEL PLATE<br>NSM | GFRP sheets (T disposition)<br>Self-tapping screws<br>Commercial steel plates<br>Near Surface Mounted steel bars |
| **IN-PLANE TESTS** | MONO_25 | GFRP_T_Pr_25 | GFRP sheets with prosthesis |

| | | | |
|---|---|---|---|
| | CYC01_25<br>CYC02_25<br>CYC03_25<br>CYC04_25 | StPl_Pr_25 | Steel plates with prosthesis |
| | MONO_50<br>CYC05_50<br>CYC06_50<br>CYC07_50<br>CYC08_50 | GFRP_T_Pr_50<br>StPl_Pr_50<br>NSM_Pr_50<br>NSM_StPl_50 | GFRP sheets with prosthesis<br>Steel plates with prosthesis<br>NSM bars with prosthesis<br>NSM bars with prosthesis and lateral steel plates |

The procedures used for these tests were adapted from standard EN 12512 (2001). For both pull-out and in-plane cyclic tests, a preliminary monotonic test was carried out in order to obtain the yield and ultimate displacement.

For the pull-out test, an ultimate displacement of 50mm **was assumed**. This value, obtained from the monotonic test (Poletti 2013), confirms what was observed experimentally in the walls, for which the maximum uplift of the posts was approximately 50mm. Since no standard is available for pull-out tests, keeping in mind this ultimate displacement, it was decided to adopt 6 steps going from 10% to 100% of the ultimate displacement, introducing also stabilisation cycles.

Four pull-out tests were performed on unreinforced half-lap connections. The specimens were designated as URTx, standing for unreinforced timber connection followed by the number identifying the specimen (Table 1).

For in-plane cyclic tests, a yield displacement of 19.9mm was obtained from the preliminary monotonic tests **performed for both vertical load levels** considering yield values for a load-displacement curve without two well-defined linear parts (EN 12512, 2001), assuming an initial secant stiffness and a secondary one with slope corresponding to 1/6 of the initial stiffness. Ten displacement steps were applied, from 25% up to 275% of the yield displacement, **in order to obtain similar drift values to those obtained for the walls for comparison purposes (Poletti and Vasconcelos 2015) and to clearly obtain the post-peak behaviour of the specimens**. **4 specimens were tested for each vertical load level. The specimens were designated as CYC_x_25 and CYC_x_50, standing for in-plane cyclic followed by the number identifying the specimen and the vertical load level (Table 1).**

For the retrofitted specimens during pull-out tests even the simplest retrofitting solution greatly increased the stiffness and strength of the connection when compared to the unreinforced one, therefore the cyclic procedure had to be updated. Very small initial steps were considered, starting from uplifts of 0.10mm up to 30mm. This was done in order to capture the whole non-linear behaviour of the connection, as some retrofitting solutions would fail for low values of uplift.

All specimens were monitored with linear voltage displacement transducers (LVDTs) in strategic positions in order to record the most significant deformation of the connections (Poletti 2013); their position **is** shown on the graphs presented.

## 2.3 Strengthening techniques adopted

After the unreinforced tests, the specimens were repaired and retrofitted in order to (1) understand the influence of the retrofitting solution on the behaviour of the connection and (2) to compare different retrofitting solutions understanding their strong points and shortcomings. The retrofitting solutions were mainly the ones adopted for the walls, but some alternate solutions were considered.

The joints were retrofitted with some of the same strengthening solutions adopted for the tests performed on timber frame walls (Poletti et al. 2014, Poletti et al. 2015), namely using steel plates and Near Surface Mounted (NSM) steel rods. Additionally, self-tapping screws and Glass Fibre Reinforced Polymers (GFRP) sheets were also used to retrofit the connections to understand the efficiency of their adoption in walls.

The NSM technique proved to be efficient for the retrofitting of walls (Poletti et al. 2015). This technique is widely used to strengthen beams and slabs and is used in this case to retrofit vertical elements against horizontal actions. Steel rods of class 8.8 with a diameter of 10mm were used both in the horizontal and vertical direction (Fig. 4), forming a cross (Poletti et al. 2015). **The effective influence of the joint was considered to be the width of the element plus 8 cm on each side (total effective influence length is 18cm in the horizontal direction and 21cm in the vertical direction). To guarantee a good anchorage of the bars**, an anchorage length of 15 times the diameter is suggested (Poletti et al. 2015), but due to the lack of space in the vertical direction and in order to still ensure the appropriate anchorage length, the rods were linked by means of a half overlap and they were additionally welded together. The same structural timber glue used for the walls was adopted (MapeWood Paste 140 (MAPEI 2002)).

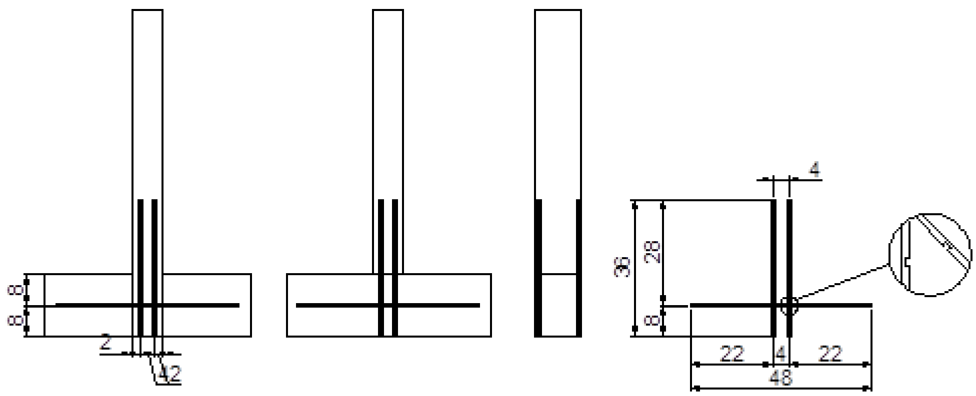



**Fig. 4** NSM strengthening performed on joints: **(from left to right) front view and positioning of bars, back view, lateral view and length of bars and detail of welded point**.

Steel plates were applied in the same way as done in the walls (Poletti et al. 2014), positioning two commercial steel plates, one horizontally and one vertically, on each side of the joint and connecting them with bolts and screws, forming a cross **(Fig. 5). Perforated plates (Rothoblaas plates PF703085 (140×400mm) (Rothoblaas, 2012)) made of steel S 250 GD with a thickness of 2mm were used. The plates were anchored using 4 bolts having a diameter of 10mm and a length of 160mm and 16 screws (type PF603550 (Rothoblaas 2012)) having a diameter of 5mm and a length of 50mm.**

An alternative solution was to strengthen the connections with GFRP sheets. A uni-directional fibre glass fabric (MapeWrap G UNI-AX (MAPEI, 2012)) was used together with an appropriate epoxy resin (MapeWrap 31 (MAPEI, 2013)) for impregnation of the fibre with a dry system. Two sheets were applied (Fig. 5), one horizontal and one vertical, glued over each other, forming a T. **The total thickness of the two layers impregnated with the epoxy resin was 1.5mm**. The GFRP sheets should be able to prevent both the uplift of the post and the out-of-plane opening of the connection.

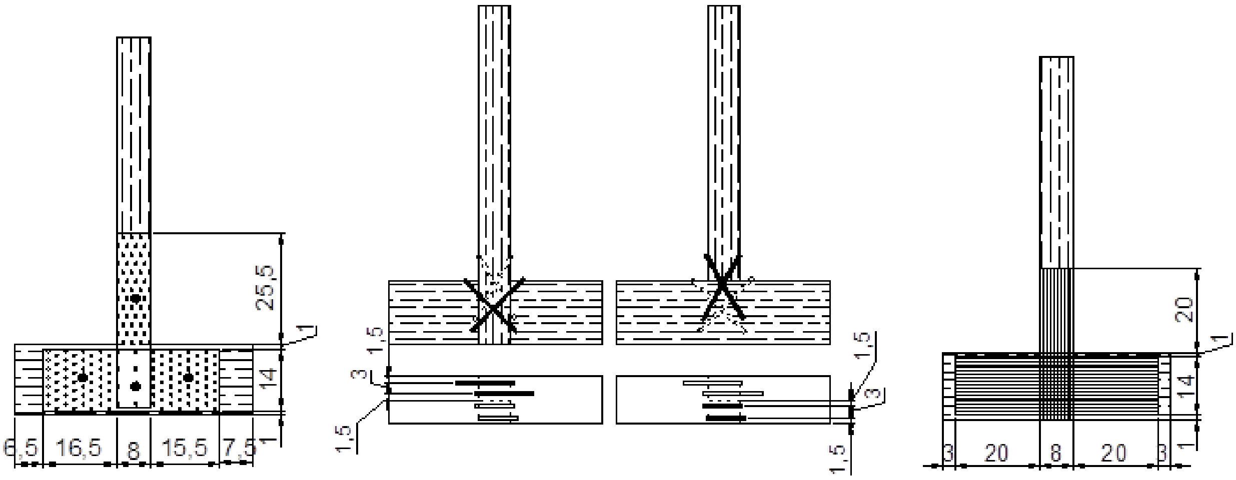



**Fig. 5** (left to right) Steel plates strengthening and positioning, disposition of self-tapping screws, GFRP sheets strengthening and positioning.

Another alternative solution was the adoption of self-tapping screws. This solution was only adopted for pull-out tests, to evaluate its effect on the connection against uplifting forces, but not against a cyclic lateral action. Four screws were inserted in the connection, two with an angle of 60° inserted from the post and intersecting the beam and two with an angle of 45° screwed from the beam and intercepting the notched part of the post (Fig. 5). Each two pairs were inserted one from each side of the connection. The screws, having a length of 19cm and a diameter of 8mm, were type VGZ9200 (Rothoblaas, 2012).

**Due to lack of specimens and time, only one specimen was adopted for each retrofitting solution and each load level. From past experience with these types of retrofitting, the scatter is minimal and therefore the single tests should be able to predict the joint behaviour sufficiently accurately**.

For the in-plane cyclic tests, various cracks were present in the notched part of the posts **from early values of drifts** and, **on** two occasions, total failure of that part occurred. Since applying the strengthening on such damaged part could alter its efficiency, it was decided to cut out the damaged part of the post and replace it with a prosthesis, since in an existing structure it is not always possible to substitute an entire element. To apply the prosthesis, different solutions could be adopted: the new piece of wood could be simply glued or glued-in rods could be used to restore continuity. The latter technique is used for example in restoration projects of historic structures (Tsakanika and Mouzakis, 2010). This solution, though, is highly invasive and already a strengthening per se. Moreover, it makes the application of some of the retrofitting solutions foreseen impossible, such as NSM steel rods or even steel plates, since the screws and bolts could come into contact with the rods.

For the purpose of the tests it was decided to adopt a simpler solution, i.e. a glued prosthesis, taking out a bigger part of the damaged post and creating an S shape contact connection by gluing the two pieces (Figure 6). Moreover, to improve adherence, two screws **(VGS9160 (Rothoblaas 2012) with a diameter of 9mm and a length of 160mm**) were used to better link the two elements of the post. The idea was to re-establish the continuity of the post. In this way, it was possible to apply all retrofitting techniques. The timber used to build the prosthesis was the same one used for the original specimens, while the structural timber glue used was the same adopted for all other applications (MAPEI, 2002).

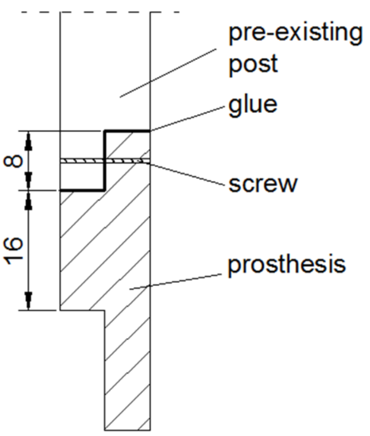



**Fig. 6** Prosthesis adopted for retrofitted specimens for in-plane cyclic tests

# 3 Results on unreinforced specimens

In this section the results of pull-out and in-plane cyclic tests will be analysed in terms of deformations and damage. **As mentioned above, results may be influenced by the presence of irregularities and gaps. For the tested specimens, gaps were measured in the half-lap joint varying between 0.5 mm and 2 mm.**

## 3.1 Pull-out tests

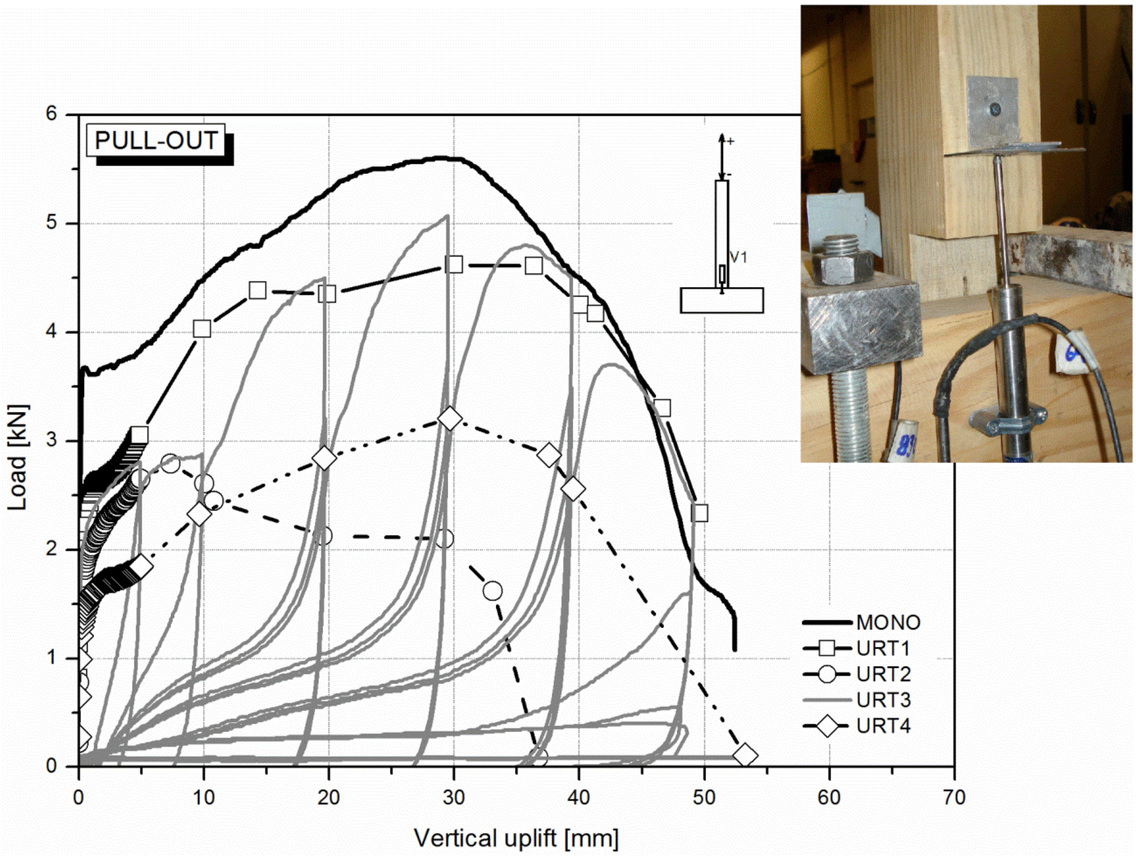
From the results of the pull-out tests it was observed that all connections behaved in a similar way, being the response characterised by out-of-plane opening when the post was pulled out and deformation of the nail. The connection stopped working when the nail was not effective, as it pulled out completely from the beam.

Analysing a typical **hysteretic** diagram of the response of the joint (Fig. 7a, specimen URT3), it is seen that the diagram is characterised by a high initial stiffness and an early non-linear behaviour. In the unloading branch, the connection has an immediate loss of strength and then acquires compression forces. This is associated **with** the reaction to the re-entering of the post to its original position in the beam, due to the plastic deformation developed in the nail. On the other hand, the reloading branches present a high amount of pinching, caused by the crushing of the wood surrounding the nail and consequent increasing gaps. **Significant** strength degradation is observed during the tests. This phenomenon is not only observed between two successive steps, but also in the stabilisation cycles. These two characteristics (pinching and strength degradation) were observed in the cyclic behaviour of timber frame walls without infill, as the behaviour of the connections affect the response of the wall (Poletti and Vasconcelos 2015). In timber frame walls, the unloading branch was characterised by the same high strength degradation observed in the connections.

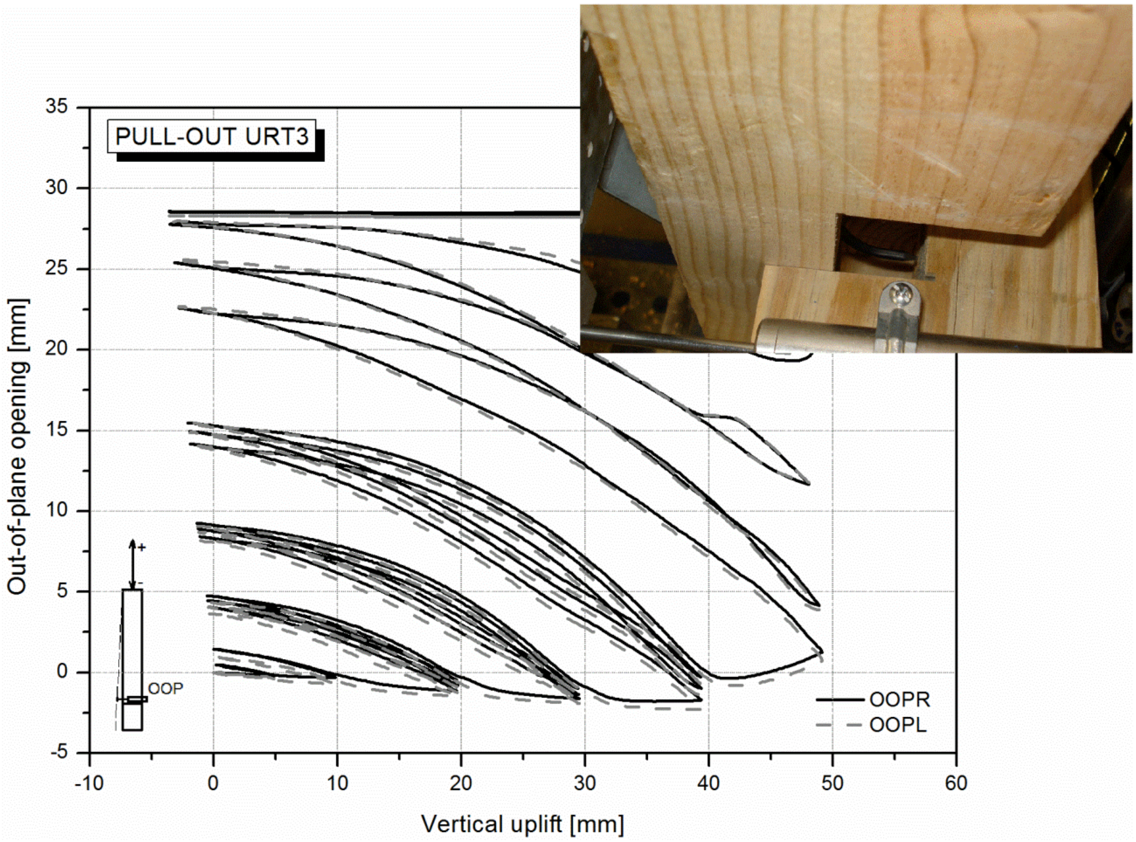
Considering the out-of-plane opening of the connection **measured in correspondence of the nail as the relative opening between the post and the fixed beam** (Fig. 7b), it is observed that the connection remains closed as the post is being pulled, **with** the existing gaps eliminated. During unloading, due to the difficulty for the post to reach its original position due to the plastic deformation of the nail and its impossibility to re-enter the beam, the connection exhibits increasing opening values for increasing vertical uplift levels, and thus for a higher deformation of the nail. This behaviour was also observed during the wall tests (Poletti and Vasconcelos 2015).

Residual or permanent out-of-plane displacements can be observed for very high values of displacement as well as **large** nail deformation. At the end of the test a permanent opening was observed in the connections, being usually higher at the bottom of the post and lower at the top of the post, indicating its rotation. The **size** of the opening varied between 20 and 45mm, depending on the level of vertical uplift reached and the level of gaps.

Even though the general behaviour of the specimens was similar, it was noticed that the maximum load and stiffness of the connection depended greatly on its level of interlocking, **as can be seen from the variation of the envelope curves presented in Figure 7a for the remaining specimens**. It was observed that for high gaps present in the connection, the load capacity of the connection decreased and the out-of-plane opening progressed more rapidly.




(a)




(b)

**Fig. 7** Results of pull-out tests: (a) typical load-displacement **hysteretic** diagram **(URT3)** and envelope curves for **remaining** specimens; (b) out-of-plane opening **(OOPR=out-of-plane opening right side, OOPL=out-of-plane opening left side)**.

Comparing the envelope curves of the tested specimens (Fig. 7a), it can be noted that specimen URT3 reached a maximum load of 5.07kN, whereas URT2 only reached 2.79kN, meaning that the resistance decreased by 45%. Moreover, the two specimens with the higher gaps (URT2 and URT4) failed earlier, since the nail pulled out of the beam at the first cycle of the step of 50mm, contrarily to what happened in the other specimens.

When the flexural behaviour predominates in the lateral response, it can be concluded that the non-linear behaviour of the bottom connections influences the overall behaviour of the walls: (1) the unloading of the walls is influenced by the difficulty of the post to recover its original position due to the plastic deformation of the nail and it corresponds to the plateau characterising the unloading branch of the force-displacement diagram (Fig. 1); (2) the same deformational features were observed in the walls and, as the vertical uplift increased, the stiffness of the wall decreased,

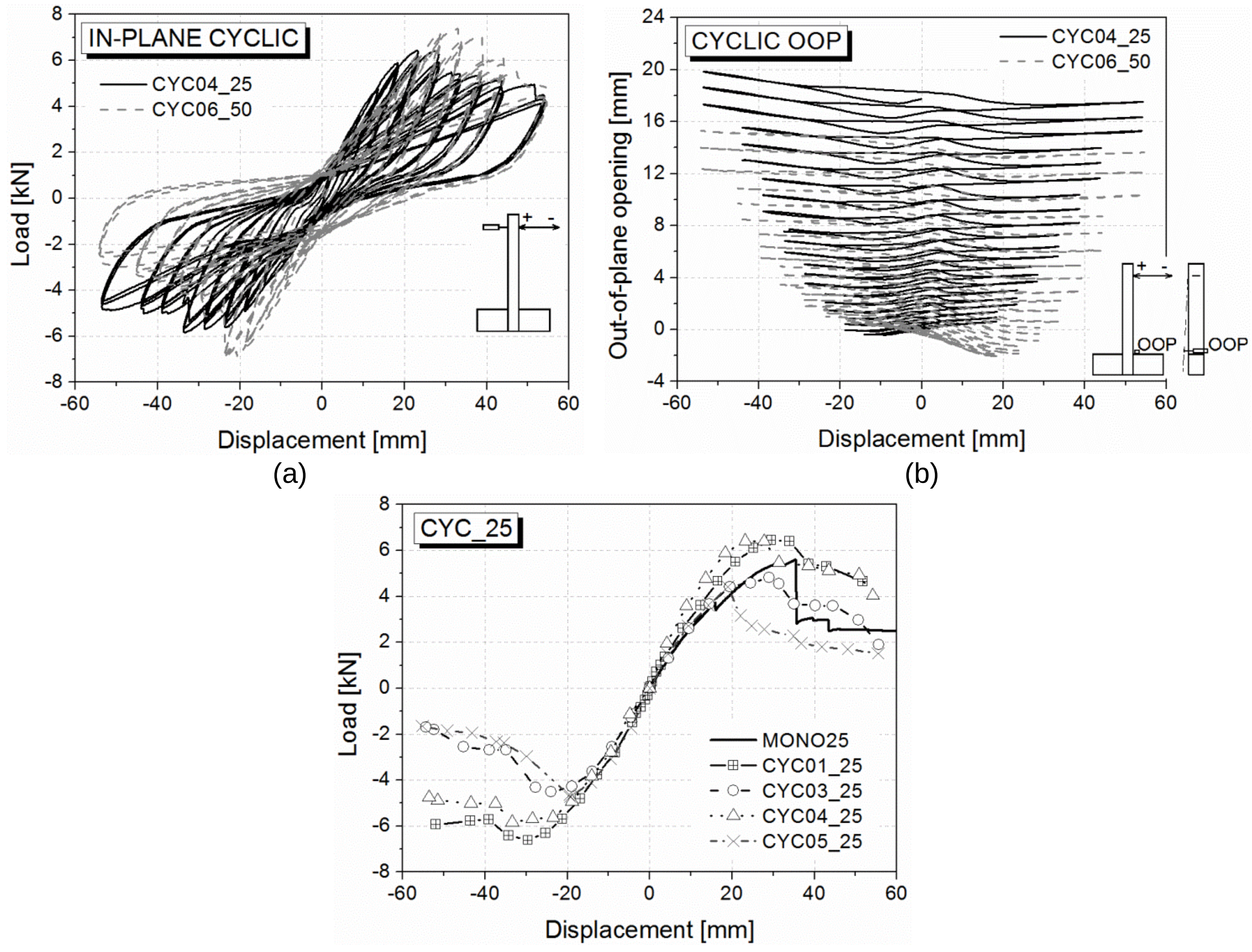
**accentuating** its rocking behaviour; (3) the pinching behaviour observed in the joints is then observed in the wall response.

## 3.2 In-plane cyclic tests

Analysing the results of the in-plane cyclic tests, the specimens behaved in a similar way for both vertical load levels. Vertical cracks were observed in the notched part of the post, associated **with** shear stresses developed during the cyclic test, as the side of the notched part of the post not in compression slides and cracks. Moreover, cracks perpendicular to the grain developed at the interface between the post and the beam. The level and shape of cracking depended greatly on the grain alignment and the presence of knots.

Usually, damage **was** greater and developed more rapidly for the connections tested with the higher pre-compression level. The nails did not deform, as the vertical uplift was minimal and they were mainly behaving as a hinge around which the post would rotate. As for the pull-out tests, the post opens in the out-of-plane direction, **with the extent** of opening dependent on the presence of gaps and on the vertical load level.

Fig. 8a presents a typical hysteretic curve of a specimen for both pre-compression levels. **All specimens tested presented very little variation for each load level (Poletti 2013).** The connection has a linear initial response and then non-linearity appears nearer to the maximum load capacity. Strength and stiffness degradation was observed. The differences in the response observed **concerned** mainly the maximum load capacity and were attributed to the quality of the connection and of the material, as the presence of knots or of grain misalignment led to lower values of load. The average maximum load is 5.53kN while the minimum -6.61kN, with variations of 19% for the lower vertical pre-compression, while for the higher load level an average maximum load of 6.48kN and a minimum of -5.98kN were observed, with a variation of 10% and 14% respectively.




(c)

**Fig. 8** (a) Typical load-displacement **hysteretic** diagram of joint for two different load levels; (b) out of plane opening (**OOP**) **on the right side of** the joint **for two load levels; (c) envelope curves of cyclic in-plane tests for lower vertical load level**.

Similarly to what **was** observed in the tests performed on walls, the out-of-plane opening of the joint played an important role on the **cyclic** results, **particularly in regards to maximum strength and stiffness**; it varied among the tests depending on the level of interlocking **(Fig. 8c).** The increase in opening with increasing values of drift is approximately linear and it becomes significant only for displacement levels higher than the yield displacement. For the higher vertical load, the out-of-plane opening was lower (Fig. 8b)., as the compression stresses in the post are higher, resulting in the trend for the post to penetrate into the beam.

During the test, the post uplifted from the bottom beam, accompanying the rotation, which resulted in **a compressive** stress concentration at the opposite side to the application of the lateral load.

Even though the cyclic test on just one connection did not represent the dual cyclic-pulling out action exercised on the connection during the test of the wall, it is still representative in terms of local damage developed in the connections of the walls (at least at the bottom corners).

# 4 Results on retrofitted specimens

The tested specimens were retrofitted with the techniques presented in Section 2.3. In this section, the deformation and damage patterns of the retrofitted joints **are** analysed.

## 4.1 Pull-out tests

Results on retrofitted pull-out tests demonstrated that even the simplest retrofitting technique can help in decreasing the level of uplifting of the connection. Depending on the type of strengthening, the connections showed a great increase of initial stiffness and maximum load capacity, and the failure modes changed completely, sometimes being extremely brittle.

Retrofitting performed with GFRP sheets had a very high initial stiffness and reached its maximum capacity (15 times greater than that of the unreinforced specimen) for a low value of vertical uplift. The failure occurred in two phases: The envelope curve presents two peak values: (1) sudden debonding of the vertical sheet that was strengthening the connection on the side where the post is discontinuous; (2) after this point, the strengthening on the other side of the connection ensured some strength and stiffness, and it was actually able to recover some strength (Fig. 9). Debonding of the horizontal sheet **continued** until total failure of the fibres.

Strengthening executed with self-tapping screws was the **least** invasive on the connection, being able to greatly improve its strength (6 times over) and stiffness without showing a brittle failure, but actually being able to ensure a post-peak softening behaviour and therefore a great capacity to dissipate energy. For this retrofitting solution, failure was mild, since the damage was progressing throughout the test, with pulling out of the screws, causing slight damage to the beam. After the peak load, **the cyclic movement of the screws caused** grain disorganization. Moreover, at the end of the test, plastic deformations of the screws were observed. Notice that this test is characterised by severe pinching, typical of dowel-type connections. Self-tapping screws have proven to be effective in the strengthening of beams and connections for shear stresses and stresses perpendicular to the grain (Dietsch and Brandner 2015).

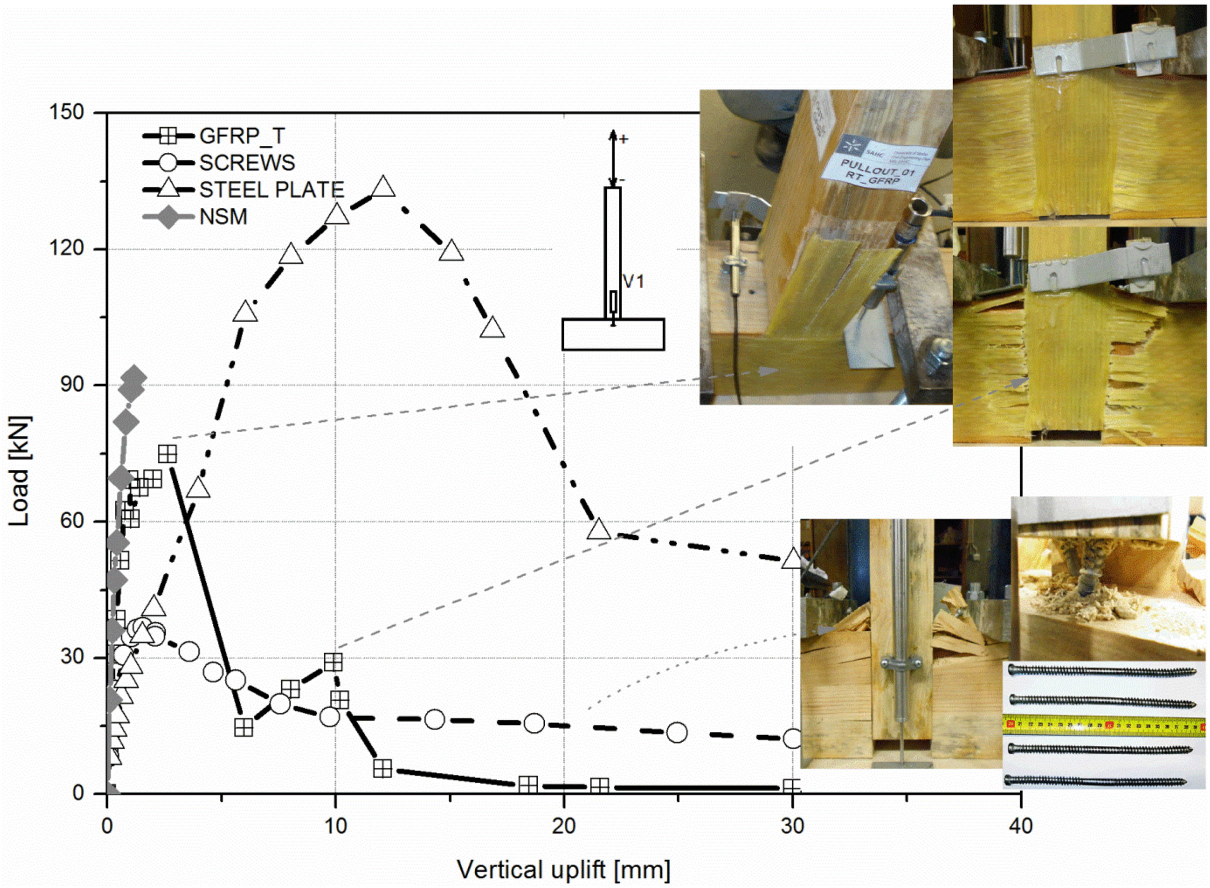



**Fig. 9** Envelope curves of pull-out tests on retrofitted specimens and damage.

Considering the test performed with steel plates (Fig. 10), the maximum load capacity exceeded the maximum load recorded in the unreinforced tests by over 46 times. Moreover, the stiffness of the connection increased greatly and a good post-peak behaviour was observed, since the steel plates were able to ensure a good residual strength even after peak load. For this case too, pinching plays an important role, particularly after peak load is attained. In fact, elongations of the steel plates were observed during the test, meaning that the bolts inserted in the connection ovalized the hole. At the end of the test, most of the screws on the post failed in shear, whereas the bolts deformed plastically.

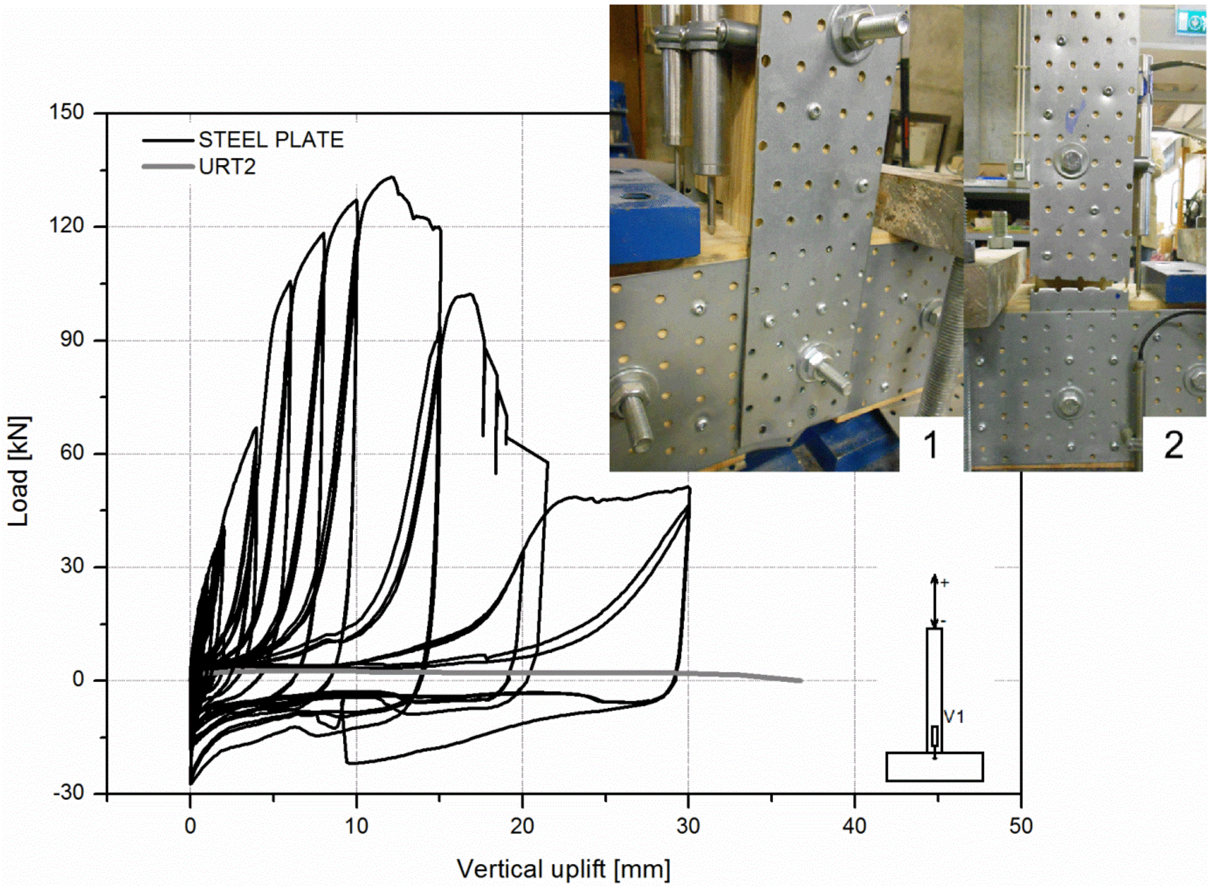



**Fig. 10** Pull-out test results of retrofitted connections: steel plates retrofitting

The connection retrofitted with NSM steel rods had a very high initial stiffness and no deformations were observed on the steel rods. Unfortunately, it was not possible to complete the test, as a problem occurred with the control LVDT. Nonetheless, the solution showed a good potential for it to be used in walls without failing due to insufficient anchorage length (Poletti et al. 2015).

Comparing the results of the different retrofitting solutions adopted (Fig. 9), two clear trends can be seen: (1) a very high initial stiffness is guaranteed by the strengthening with GFRP and NSM leading to a brittle failure for the former and (2) the increase in stiffness is not as severe (but still **large**) and a more ductile behaviour is observed in connections strengthened with self-tapping screws and steel plates. The latter constitute solutions characterised by a greater dissipative capacity. Brittle failure should be avoided, but it is possible that in a wall, where all connections are participating, the behaviour can be different. A further investigation would be required.

## 4.2 In-plane cyclic tests

The analysis of the results obtained in the in-plane cyclic tests on the connections show that their mechanical behaviour is greatly influenced by the condition of the prosthesis. Early failure of the prosthesis was observed for all specimens, indicating that the continuity of the post was not guaranteed.

Considering the specimens retrofitted with GFRP sheets, the strengthening was able to increase both initial stiffness and maximum load capacity for both vertical pre-compression levels by 43% and 52%, respectively. However, this behaviour was observed only up to a certain value of drift, after which the prosthesis failed, the lower part of the post remained vertical, whereas the upper part was rocking around the screws. This occurred for both load levels and pointed **to** the weakness of the prosthesis adopted. **In fact, the prosthesis and its connections to the post was weaker and presented** a lower stiffness than that of the retrofitting solution applied and was not able to promote the continuity of the post.

In case of retrofitting with steel plates submitted to lowest pre-compression load level an increase of 40% and 21% was observed for the maximum load for the lower and higher vertical level, respectively (Fig. 11). **Note that CYC25 and CYC50 represent the average envelope curves of the four specimens tested respectively for the lower and higher vertical load**. When the prosthesis failed the upper part of the post was simply rocking. However, for the higher level of pre-compression, this trend was not observed. In this case the post bended and deformations related to buckling were also observed in the steel plate (Fig. 11), similarly to what happened in the wall tests (Poletti et al. 2014).

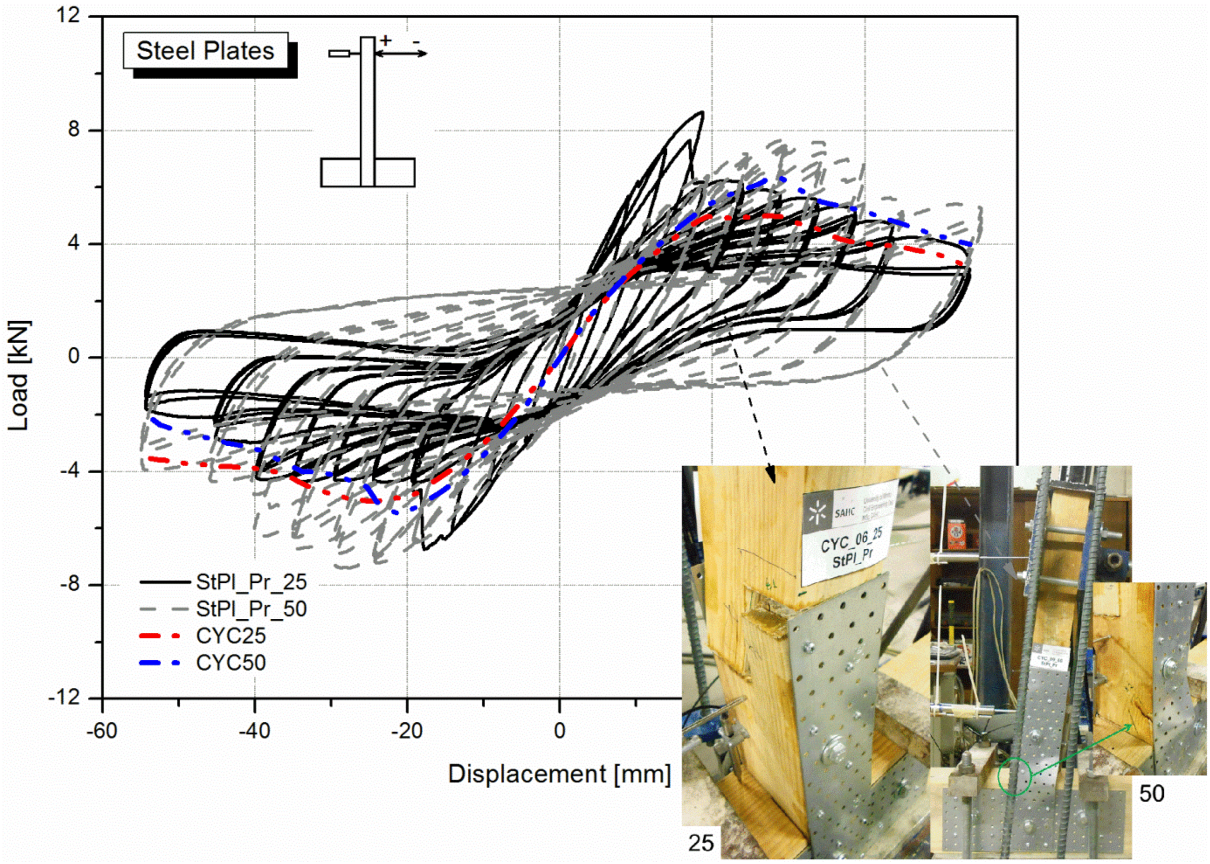



**Fig. 11** Hysteretic diagrams of retrofitted tests: steel plates retrofitting

To avoid failure of the prosthesis, the specimens retrofitted with NSM steel rods were tested directly with the higher vertical load (50 kN). Nevertheless for this kind of strengthening the prosthesis failed even for the higher pre-compression level. After experiencing an increase in load of 34%, the lower part of the post once again stopped contributing for the resisting mechanism. This was due to the fact that NSM retrofitting stiffens the post more and the high difference in stiffness between the two parts of the post caused the prosthesis to fail even for the higher vertical load.

Aiming at preventing this undesirable behaviour, commercial steel plates were screwed laterally to the post, linking the two parts in order to guarantee continuity. With this procedure, a better continuity was obtained, even if the post was still not completely monolithic. With this solution the maximum load increased by 45% in the positive direction and by 24% in the negative one, whereas the initial stiffness remained approximately the same, since the addition of the steel plates influenced only the continuity of the post, but not the stiffness of the connection (Fig. 10).

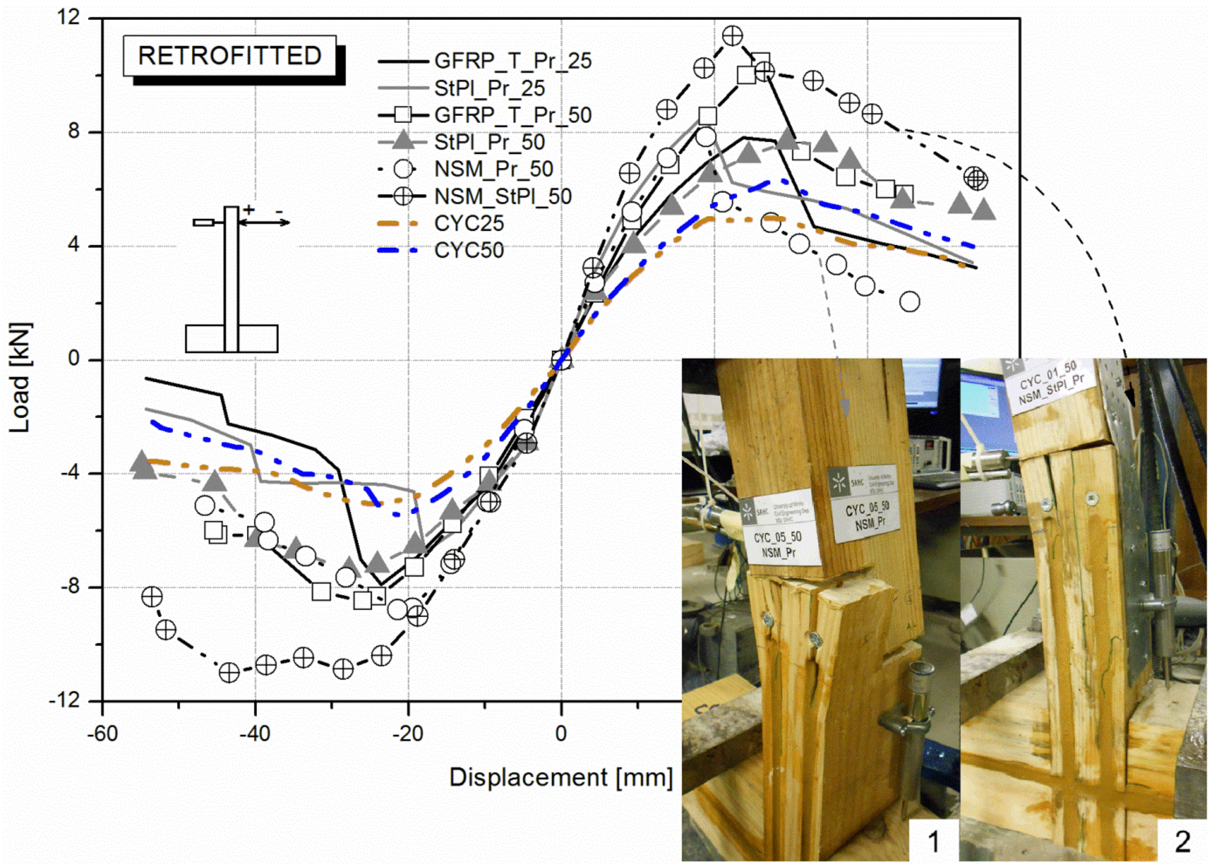



**Fig. 12** Comparison among retrofitting solutions and damages in NSM retrofitted joint, **envelope curves**.

Comparing the different retrofitting solutions adopted, on healthy connections steel plates and GFRP sheets could improve significantly the performance of the connection, even though here the results obtained were limited by the early failure of the prosthesis. For the higher vertical pre-compression level, results are clearer and it appears that GFRP and NSM are able to guarantee the highest stiffness while the connection with steel plates experienced the less degradation in terms of strength.

## 5 Evaluation of seismic parameters

To better evaluate and compare the seismic performance of the retrofitted connections, seismic parameters such as ductility, energy dissipation, cyclic stiffness and viscous damping were evaluated for the tested joints. The parameters were calculated considering the same formulae adopted for the walls (Poletti and Vasconcelos 2015).

## 5.1 Initial and cyclic stiffness

In this section the normal and lateral stiffness are calculated based on the pull-out and cyclic tests. The initial stiffness, $K$, is calculated according to European Standard ISO 21581 (2010), i.e. considering the ratio between 30% of the maximum load reached and the displacement between 40% and 10% of the maximum load (Eq. 1).

$$K = \frac{0{,}3F_{max}}{\delta_{40\%F_{max}} - \delta_{10\%F_{max}}} \tag{1}$$

where $\delta_{40\%F_{max}}$ and $\delta_{10\%F_{max}}$ are the displacement values measured on the envelope curve at 40 and 10% of the maximum load ($F_{max}$) respectively.

The cyclic stiffness, used to evaluate the stiffness degradation experienced by the walls, is calculated for each cycle as the slope of the line connecting the origin with the two points of loading corresponding to the maximum positive and negative displacements.

### 5.1.1 Pull-out tests

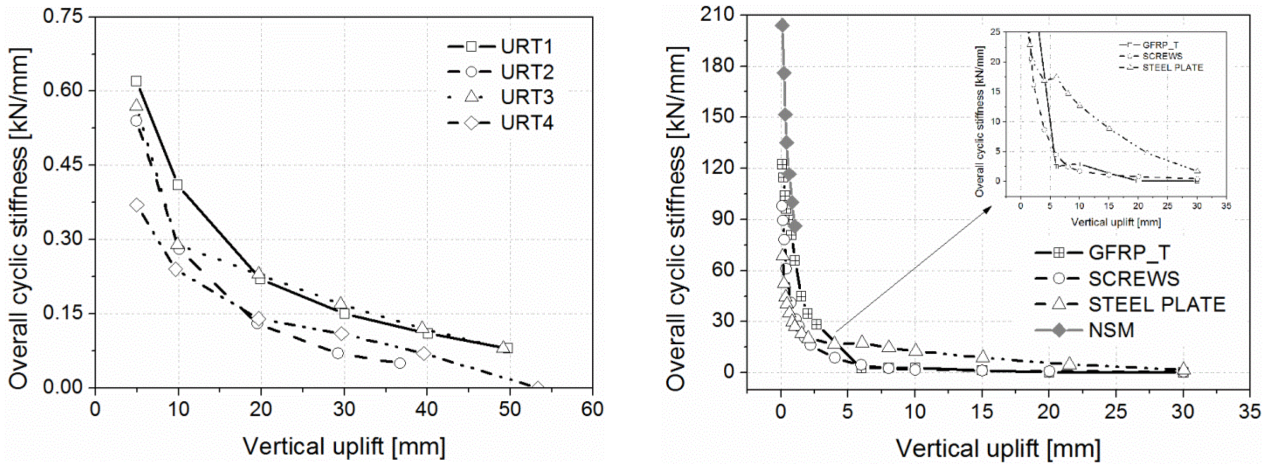
All unreinforced connections presented low values of initial stiffness with some variation among the results; the average value of stiffness is 3.94 kN/mm with a coefficient of variation (C.O.V.) of 24%. As already mentioned, this is due to the level of interlocking in the connection, since some specimens presented **large** gaps, which depends greatly on the workmanship of the carpenter.

All retrofitted specimens presented very high values of initial stiffness. With this respect, it should be noticed that the specimen strengthened with the NSM technique cannot be considered as the final one, since the maximum load was not reached. Steel plates presented the lowest value of initial stiffness (14.96 kN/mm) followed by self-tapping screws (67.96 kN/mm), while GFRP sheets provided a high stiffness (128.53 kN/mm).

Self-tapping screws are driven into the timber elements and they tighten the connection, since they create their own precisely fitted hole, therefore possible gaps between the post and the beam, which influence the vertical uplift, are eliminated and the contact of the horizontal interface is improved, as well as the friction of the vertical interfaces. In the case of strengthening with steel plates holes have to be drilled to accommodate the bolt, therefore small gaps could be created decreasing the initial stiffness of the connection. This is one of the advantages of self-tapping screws, since they allow a direct entrance of the element ensuring a perfect adherence to the material. The strengthening with GFRP sheets led to a high initial stiffness, but its failure was quite brittle.

As far as stiffness degradation is concerned, all unreinforced connections presented a similar logarithmic trend. In this case, specimens with higher gaps exhibited the highest degradation in stiffness, pointing out the importance of good interlocking in the connection (Fig. 13a).

Apart from the specimen retrofitted with steel plates, all the other retrofitted connections had a similar trend in terms of **stiffness** degradation (Fig. 13b). Even though the specimen retrofitted with steel plates presented the **lowest** initial stiffness, its degradation was the slowest and at the end of the test its residual stiffness was almost the double of the solution adopting self-tapping screws.




(a) (b)

**Fig. 13** Stiffness degradation found for pull-out tests: (a) unreinforced specimens; (b) retrofitted specimens

### 5.1.2 In plane cyclic tests

Connections subjected to in-plane cyclic tests presented lower variations in terms of cyclic stiffness for the unreinforced specimens, even if for some specimens an asymmetry existed between the two directions, mainly attributed to the asymmetry in damage patterns. For the lower vertical load, an average value of initial stiffness of 0.33 kN/mm (C.O.V. 13%) was obtained, while for the higher load a value of 0.34 kN/mm (C.O.V. 19%) was obtained. For the joints, similar values of stiffness were obtained for both vertical load levels, contrarily to what observed in timber frame walls, for which a higher vertical pre-compression resulted into a higher initial stiffness.

All types of strengthening increased the values of initial stiffness of the connections with the exception of already damaged specimens. The strengthening with GFRP increased the initial stiffness by 45% and 44% for the lower and higher vertical load, while steel plates increased the initial stiffness by 103% and 58% for the lower and higher vertical load respectively. NSM retrofitting increased the value of initial stiffness by 75%.

Taking into consideration the cyclic stiffness degradation, the unreinforced specimens presented a similar trend in stiffness degradation and the values did not vary greatly for the two load levels (Fig. 14).

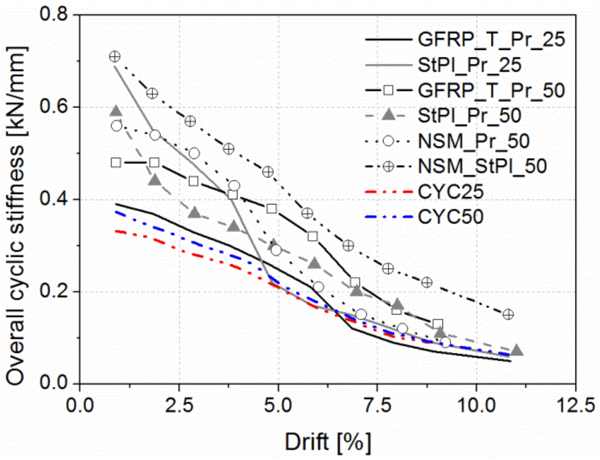



**Fig. 14** Cyclic stiffness degradation of connections obtained in in-plane static cyclic tests: **comparison between retrofitted specimens and average results for unreinforced specimens**.

The stiffness degradation of retrofitted connections was heavily influenced by the prosthesis and by the previous damage. For the lower vertical load level, the damaged connections had the same cyclic stiffness **as** the unreinforced specimen throughout the test (Fig. 14). Only the steel plate retrofitting **produced an** increase the values of cyclic stiffness **and, after** the prosthesis failed, the stiffness became similar to the one recorded in the unreinforced specimen. The GFRP strengthening applied on a specimen with prosthesis increased the values of cyclic stiffness only slightly prior to the failure of the prosthesis.

For the higher vertical load level, all retrofitting solutions had an increase **in** cyclic stiffness (Fig. 14), **although** it quickly degraded as the prosthesis became ineffective. The specimen retrofitted with NSM rods and additional steel plates to make the prosthesis effective, showed higher values of cyclic stiffness and a higher residual stiffness indicating that, with an appropriate prosthesis or even in an undamaged connection, the retrofitting solution should behave appropriately. The other connection where the prosthesis partially worked (steel plates) showed a similar **rate** of degradation. For all connections, the degradation trend was approximately linear.

## 5.2 Energy dissipation and viscous damping

The dissipative capacity of the connections was also analysed. This issue is of great importance since the walls dissipate energy mainly through their connections and it allows the contribution given **of** retrofitting technique **to be clearly understood**. The energy dissipated in each cycle is computed by calculating the area enclosed by the loop in the load–displacement diagram. The energy dissipated in subsequent cycles is added as drift progresses.

**Equivalent viscous damping is calculated according to Eq. 2 (Magenes and Calvi 1997):**

$$\zeta_{eq} = \frac{E_d}{2\pi(E_e^+ + E_e^-)} \tag{2}$$

**where $E_d$ is the dissipated hysteretic energy and $E_e^+$ and $E_e^-$ are the elastic energies of an equivalent viscous system calculated at maximum displacement for each direction of loading.**

### 5.2.1 Pull-out tests

The dissipated energy during pull-out tests carried out on unreinforced specimens was minimal, especially for connections with inadequate interlocking, meaning that friction does not play a role in the dissipation of energy.

The dissipated energy increases considerably once strengthening against uplifting is applied (Fig. 15a). The stiffer solutions **greatly** increased the dissipative capacity of the connections. The **most effective** dissipative strengthening technique is the steel plates solution, which increases the energy dissipation by over 8 times. It is not possible to give indications on NSM retrofitting, since it was not possible to complete the test. Retrofitting with screws was able to promote a good dissipative capacity of the connection, with an increase of 175% in relation to unreinforced specimen. The strengthening with GFRP leads to a good dissipative capacity for low values of uplift, but after failure the energy dissipation is lower than the value found for the equivalent unreinforced connection. Therefore, it is evident that brittle failures should be avoided, since the strengthening becomes inefficient.

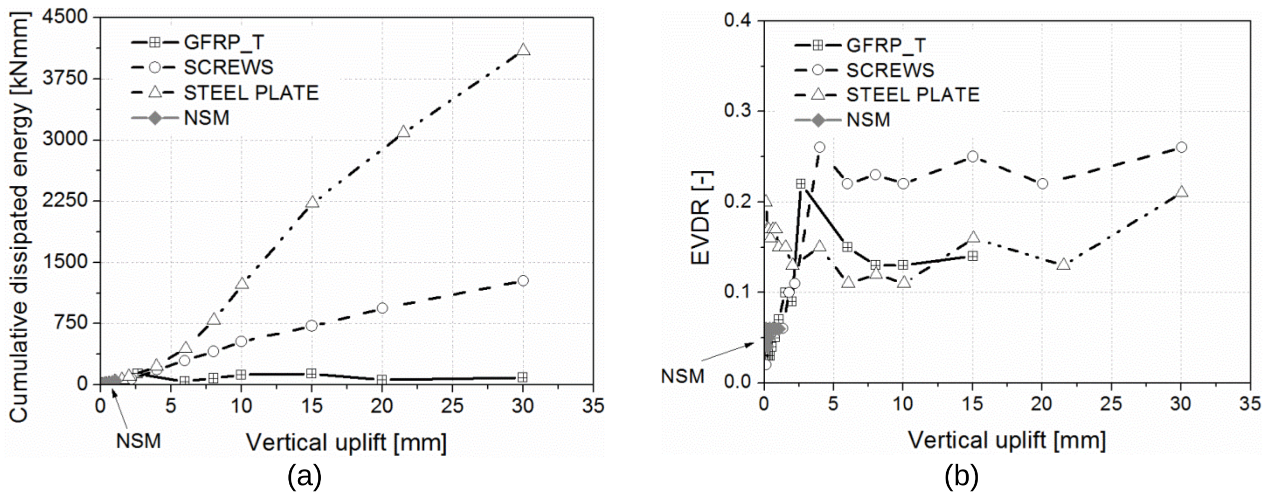


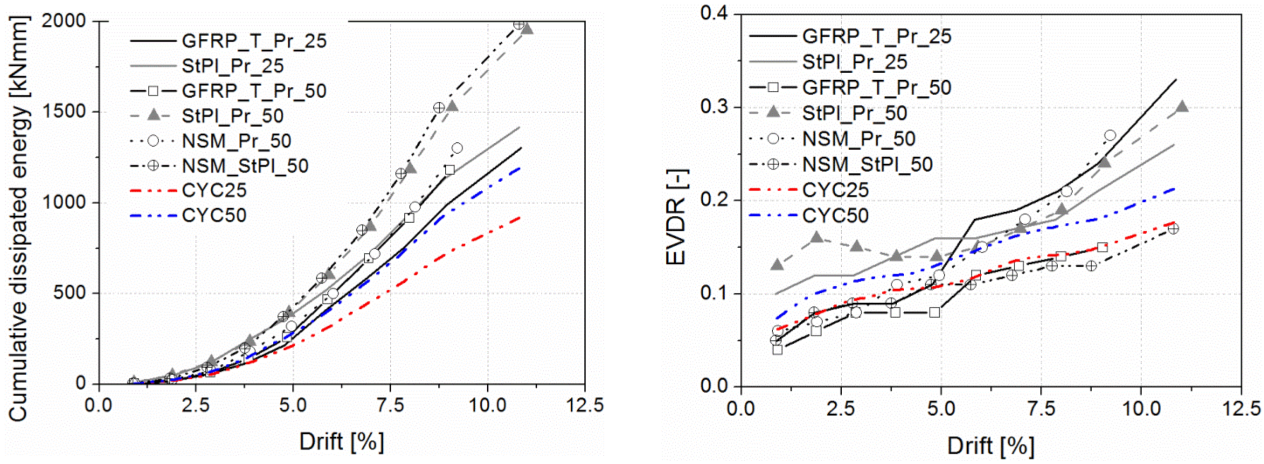

**Fig. 15** Pull-out tests: (a) Cumulative dissipated energy for retrofitted specimens; (b) EVDR for retrofitted specimens

Similar conclusions can be drawn on the equivalent viscous damping ratio measured for the connections. Values of damping of unreinforced connections are influenced by their level of interlocking (varying from 0.28 to 0.35 for low values of uplift); the values of viscous damping progressively decrease as the uplifting increase, with final values varying between 0.12 and 0.25. For retrofitted connections pinching plays an important role (Fig. 15b). In fact, connections retrofitted with screws presented the higher values of damping (0.28), as the effect of pinching was less severe than what observed for example in case of retrofitting with steel plates, for which energy dissipation was diminished by pinching, decreasing its ratio with input energy, and the value registered throughout the test was approximately 0.18.

Note that retrofitted joints tended to have higher values of damping than the ones in the wall tests, particularly for retrofitting performed with screws. It is however difficult to associate these results to walls, since the dissipative capacity of the walls depends on all connections.

### 5.2.2 In plane cyclic tests

In case of the connections tested under in-plane cyclic lateral loading, the unreinforced specimens with higher strength were able to dissipate more energy (Fig. 16a). In general, specimens tested with the higher pre-compression level were able to dissipate more energy, in average 29% more. Considering the retrofitted connections, all solutions were able to improve the dissipative capacity of the joint (Fig. 16a). The failure of the prosthesis influenced the results, as it is clear that energy dissipation was greater for specimens where the prosthesis was more efficient (i.e. steel plates with higher vertical load and NSM with lateral plates).




(a) (b)

**Fig. 16** In-plane tests: cumulative dissipated energy during in-plane cyclic tests: **comparison between retrofitted specimens and average results for unreinforced specimens**; (b) EVDR for retrofitted specimens

All joints tested present increasing values of viscous damping ratio (Fig. 16b). The retrofitted connections generally did not overly improve viscous damping, but results are to be taken with caution since the prosthesis failed.

# 6 Influence of joint response on wall behaviour

Correlating results extracted from joint tests to the global behaviour of a timber frame wall is complex, since the latter has a set of connections all interacting with each other and the joint tested is only representative of some of these. However, some observations can be made.

Similar crack patterns to those registered at the joints tests were observed at the bottom joint of the walls; vertical cracks would appear at these connections (Poletti and Vasconcelos 2015), leading up to the nail. There was also a contribution of the diagonal element to the cracks, but it was not taken into consideration in the tests presented here.

Pull-out tests are representative of the **large** vertical uplift observed during the cyclic tests on timber frame infill walls, which led to the rocking response of the walls (Poletti and Vasconcelos 2015). In this case, when the post is in tension, the diagonal does not influence the uplifting. The same out-of-plane opening of the connection observed here **occurred** in the walls tests, with the plastic deformation of the nails. The retrofitting solutions adopted are suitable to prevent this uplift, and thus have a shear response of the wall, keeping always in mind not to over-stiffen the connection and that the retrofitting has to be compatible with the lateral cyclic movement. Further studies are required in the response of self-tapping screws to horizontal actions, but this solution has the potential to be an easy and cheap intervention that good greatly improve the structural response.

Retrofitting with GFRP sheets can be a possibility to improve the behaviour of the bottom connection of the walls under uplift movements. This technique exhibited a good behaviour in terms of resistance, but it is considered that bi-axial sheets should be **preferred** in order to avoid **early debonding**.

Considering the effect of steel plates on walls and joints, in both cases compressive stresses cause buckling of the plates. Moreover, in tension plastic deformation of the bolts and tensile failure of steel plates in correspondence to the holes was registered. This solution has excellent dissipative capacities, it is easily implemented and relatively cheap, without requiring specialised labour. Further studies should be made in order to take into consideration the action of diagonal bracing members on the strengthened joint and the possibility to connect them to the plates, since this could overly stiffen the wall (Poletti et al. 2015).

Regarding the NSM technique, it should be stressed that configuration of the steel rods should be able to solve the insufficient anchorage length problem encountered in timber frame walls (Poletti et al. 2015). The welded cross guarantees a good dissipative capacity during the in-plane cyclic test (the prosthesis's influence notwithstanding) and it should be able to ensure adequate strength against uplifting actions. As a downside, the excessively high initial stiffness registered during pull-out tests could have a negative effect on the out-of-plane behaviour of the walls, but the free movement of the diagonals should limit this problem.

# 7 Conclusions

When considering **intervention** on a structure, it has to be kept in mind that an interrelation exists on interventions at all levels, from the whole structure, to the structural elements, to the individual joints.

To better understand the behaviour of traditional timber frame walls, it is important to understand their correlation with the key factors influencing their structural response, i.e. the connections. To do this, pull-out and in-plane cyclic tests have been performed on unreinforced and retrofitted traditional connections to study the joint in more detail and try to correlate its behaviour with the behaviour of the walls. From the results obtained, the following conclusions can be made:

- Pull-out and in-plane cyclic tests on unreinforced connections were mainly influenced by the quality of interlocking in the joint;
- The deformational features of the connections were in accordance with **those** found in the walls, where connections would open out-of-plane due to the asymmetry in thickness of half-lap connections;
- Retrofitting applied to specimens for pull-out tests **greatly** increased the initial stiffness of the connection presenting also a higher energy dissipation capacity. However, attention should be paid to the desired level of strength, in order to avoid brittle failure. Retrofitting

performed with screws and steel plates was able to guarantee a post-peak softening behaviour;

- The good performance of self-tapping screws to uplifting forces should be complemented by a study of its response to in-plane cyclic actions, since this solution has the potential of being a simple and cheap intervention;
- Uni-axial GFRP sheets are not appropriate for undergoing shear forces, due to their early rupture and debonding;
- The prosthesis solution adopted was inappropriate for the strengthening solutions used, since it created a weaker section in the post than some of the retrofitting techniques adopted in the actual connection. It would be more appropriate to create a well strengthened prosthesis, for example with embedded rods, and apply additional strengthening, if needed, around it or use a bigger prosthesis with more screws;
- The NSM configuration adopted for the bottom connection appears to be able to guarantee sufficient anchorage length to the rods, without early failure of the welding;
- Additional information on the influence of diagonal bracing members on the cyclic response of the connections is needed;

**The strengthening presented here constitutes a local solution and concerns the behaviour of a single joint. It is then important to study the global behaviour of such strengthening. It was seen in previous studies that local strengthening to the joints greatly improves the seismic capacity of shear walls. If a weak prosthesis was added to a wall and early failure occurred, this would then influence the capacity of the whole wall, setting on an early failure of the same. A balance between a strong strengthening and the ability to maintain the ductility of the wall needs to always be considered.**

# 8 Acknowledgements

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