Full scale experimental testing of retrofitting techniques in Portuguese "Pombalino" traditional timber frame walls

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Abstract

Traditional timber frame walls are constructive elements representative of different timber frame buildings that are well known as one of the most efficient seismic resistant structures in the world. Timber frame walls were also used in the reconstruction of buildings of the old town of Lisbon after the earthquake of 1755 aiming at improving their seismic global behaviour. As it is important to preserve these structures, a better knowledge about their behaviour under seismic actions is important and can give some indications about possible retrofitting techniques to be used to improve the seismic performance. Due to the great rehabilitation effort currently carried out in many countries, a better understanding of retrofitting techniques is also needed. Therefore, this paper aims at providing a study on possible retrofitting techniques adopting traditional solutions such as bolts and steel plates. Static cyclic tests have been performed on retrofitted **traditional timber frame** walls in order to study their seismic performance. The experimental results showed the overall good seismic performance of steel plates and the more ductile behaviour of retrofitted timber frame walls with bolts

Keywords: timber frame wall, cyclic test, retrofitting, steel plates, bolts, stiffness, energy dissipation

1 Introduction

In the past centuries all sort of natural materials have been used to build walls, such as mud, earth, straw, clay, cork and wood. Timber has often been associated to masonry as a complementary material to bind masonry. **Timber frame** walls combine these natural materials, creating a traditional structural element relatively cheap and which can be built with the materials locally available.

Timber frame walls are often adopted in seismic regions as shear walls, in order to resist to horizontal seismic actions. An example of this, is the adoption of **timber frame** walls in the reconstruction of vernacular buildings in Haiti [Vieux-Champagne, 2012], after the earthquake of January 2010. The particular geometry of the walls, with St. Andrew's crosses, contributes to the capacity to dissipate the energy generated by the earthquake motion. Besides, the timber structure that acts as a skeleton of the building does not encounter severe damages during the earthquake [Mascarenhas, 2004; Gülhan and Güney, 2000].

The origin of **timber frame** structures probably goes back to the Roman Empire, with what is called Opus Craticium by Vitruvius [Langenbach, 2007]. But examples were also found in previous cultures, such as in the Minoan palaces in Knossos and Crete [Tsakanika-Theohari, 2006], where, according to historians, timber elements were used to reinforce masonry [Tampone, 1996]. **Timber frame** constructions later spread not only throughout Europe, such as Portugal (edifícios pombalinos), Italy (casa baraccata), Germany (fachwerk), Greece, France (colombages or pan de bois), Scandinavia, United Kingdom (half-timber), Spain (entramados) etc., but also in India (dhaji-dewari), Turkey (himis and bagdadi), Peru (quincha), USA (balloon frame in Chicago) and Haiti (Gingerbread houses) [Langenbach, 2007; Cóias, 2007]. In each country different geometries for the traditional timber frame walls were used, but the common idea is that the timber frame can **confine masonry**, providing a better resistance to horizontal loads.

In Portugal these structures were adopted after the devastating earthquake that hit Lisbon in 1755 for the construction of residential and commercial buildings, known as Pombalino buildings. These buildings are characterized by external masonry walls and an internal timber structure, named *gaiola* (cage), which is a three-dimensional braced timber structure. The *gaiola* consists of horizontal and vertical elements and diagonal bracing members, forming the typical X of St. Andrew's crosses. The internal walls of the *gaiola* (*frontal* walls) may have different geometries in terms of cell dimensions and number of elements. The timber elements are notched together or connected by nails or metal ties. Traditional connections used for the timber elements varied significantly in the buildings: the most common ones were mortise and tenon, half-lap and dovetail connections.

Since these structures constitute an important historical heritage in many city centres in the world, their conservation is of essential importance and many restoration works are being done without having accurate knowledge on the behaviour of these structures before and after the application of the strengthening, which sometimes modifies significantly the original structure. Numerous Pombalino buildings in Lisbon have been retrofitted using FRP sheets in the connections of the frontal walls, creating a star-shaped strengthening [Cóias, 2007], or damping systems linked to frontal walls and to the outer masonry walls through injected anchors and providing additional bracing [Cóias, 2007]. In spite of this, little information is available on the actual behaviour of the strengthened walls under seismic actions. Cruz et al. [2001] performed diagonal tests on reduced scale wallets strengthened with Glass Fiber Reinforced Polymer (GFRP) rods and Glass Fiber Fabric (GFF) sheets. The walls were retrofitted embedding two GFRP bars in the outer timber members and GFF sheets were glued to the timber elements at the central connections. More relevant information is available on retrofitting techniques for traditional timber connections [Branco, 2008; Parisi and Piazza, 2002], which are also of great importance for the strengthening of traditional walls, since strengthening of these walls is almost reduced to the strengthening of the connections. Branco [2008] studied the cyclic behaviour of traditional timber connections (bird's mouth connections) and appropriate retrofitting solutions, which consisted of metal stirrups, internal bolts, binding strips and tension ties, being all considered as traditional solutions.

Higher amount of research is available on modern timber frame walls [Lam et al., 1997; Toothman, 2003], but for these walls strengthening is not usually considered. An improvement of their seismic behaviour is usually achieved through the adoption of different sheathing or through an alternative disposition of the frame [Varoglu et al., 2006]. Premrov et al. [2004] studied timber frame walls coated with carbon fibre-reinforced polymers (CFRP) strengthened fibre-plaster boards.

Following an experimental research on the evaluation of the in-plane cyclic testing of **timber frame** walls characteristic of Pombalino buildings, it was decided to strengthen the walls through two traditional techniques in order to assess the improvements on the cyclic performance, namely bolts and steel plates. The main aim of the work presented herein is to contribute to the state of the art and attain a better insight on the mechanical behaviour of these structural elements under horizontal loads with different retrofitting solutions.

2 Experimental campaign performed on timber frame walls

The objective of this work is to study the seismic performance of the Portuguese traditional timber frame walls, characteristic of Pombalino buildings built in the reconstruction of the old town after the great earthquake that hit Lisbon in 1755. The idea is that timber frame walls can improve considerably the seismic performance by contributing to the shear resistance of the building submitted to lateral loads. With this goal in mind, an intensive experimental program was designed based on static cyclic tests on real scale walls. Static cyclic tests are able to simulate, in a simplified manner, the seismic loads to which a shear wall is subjected during an earthquake [Tomaževič et al., 1996].

2.1 Wall specimens and types of retrofitting

The two adopted retrofitting techniques (bolts and steel plates) were applied on the previously unreinforced **timber frame** walls tested under cyclic loading [Poletti, 2013]. The tested walls were then repaired and retrofitted with distinct retrofitting techniques.

The sectional dimensions of all the members of the timber frame and the size of the cells were decided according to the dimensions of existing buildings found in literature [Mascarenhas, 2004]. The top and bottom beams have a cross section of 16×12cm^2 and all the other members have a cross section of 8×12cm^2. The total width of the wall is 2.42m and the total height 2.36m, resulting in a height to length ratio of approximately 1.0, even if in some cases this ratio can be higher. The cells are 86cm wide and 84cm high.

The connections of the main frame and between each two diagonal are all half-lap connections, , whereas the connection between the main frame and the diagonals is made by contact (Fig. 1). In all of the connections a nail was inserted.

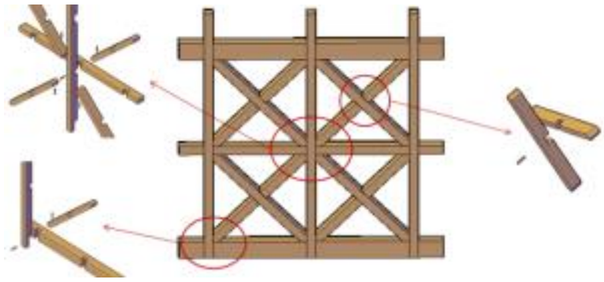


Fig. 1 Timber frame with detail of the type of connections adopted

The original Pombalino buildings presented timber frame walls filled with masonry (either brick or rubble), and thus part of the walls were filled with brick masonry, with a pattern suggested by a Portuguese company specialized in rehabilitation. The masonry pattern consists of double leaf masonry with transversal series of bricks every two rows of horizontal double leaf masonry, as detailed in Fig. 2. **Some specimens were kept without infill in order to assess the influence of its presence on the timber frame [Poletti, 2013].**

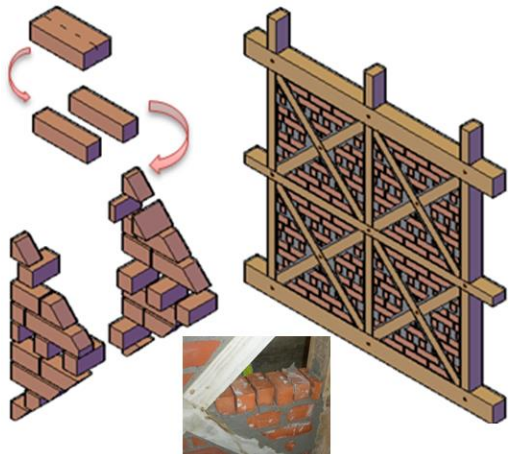


Fig. 2 Timber frame walls: masonry pattern adopted for infill walls

For the strengthening, two distinct types of traditional techniques were adopted, namely: (1) bolts and (2) steel plates. The selection of these techniques was based on the damages found in the unstrengthen walls [Poletti, 2013]. Both techniques use steel as material and are considered to be fast to be applied and removable (reversible), which represents an advantage, mainly concerning rehabilitation of cultural heritage structures. **The retrofitting was applied to the connections, as the damaged observed for both wall typologies were concentrated at the connections.**

The design of the retrofitting techniques was based on of the following parameters; (1) cross-sectional dimensions of timber elements involved, keeping in mind limitations on the minimum distances from the borders for bolts and screws [Eurocode 5, 2004]; (2) presence of knots or of pre-existing drying fissures, since they could create a preferential failure path. For this reason, visual inspection is suggested before strengthening the walls; (3) tensile strength and ductility of steel plates. It should be ensured that the plates have an adequate capacity to deform and that failure by tearing of the plate in tension does not occur, meaning that the tensile strength of the plate has to be greater than that of the component material.

For the first retrofitting technique one bolt was added in each half-lap connection of the main frame, see Fig. 3, aiming at tying together the two elements of the half-lap connection, namely vertical post and horizontal beam. **The choice of using one bolt for each connection was based on the spacing limits provided by Eurocode 5 [2004] concerning edge and end distances, both in terms of position and in terms of bolt diameter.** The bolts pass through the thickness of the wall. The selection of these connections was based on the trend for the out-of-plane detachment of the posts from the beam exhibited by unreinforced walls, especially at the bottom connections [Poletti, 2013], reducing thus the efficiency of the connection. The bolts had a diameter of 10mm and a total length of 160mm and were of the class 8.8 steel fasteners. They were inserted in pre-drilled holes, according to recommendations of Eurocode 5 [2004]. Washers were used to better distribute the stresses.

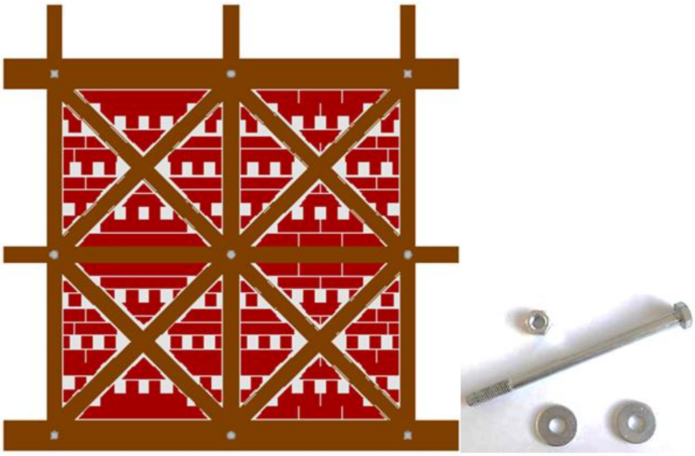


Fig. 3 Bolts strengthening: a bolt was used in each connection

The second type of strengthening consists of applying steel plates in all connections on both sides of the wall. Two types of steel plates were used; (1) custom plates were designed with a star-shape (Fig. 4a), **compatible with the geometry of the connection and aiming at not covering, neither confining the masonry infill**. The steel plates are secured with screws and linked with bolts going from one side to the other of the wall. The steel plates can link the diagonal bracing elements to the main elements of the connection (vertical post and horizontal beam). The plates were made **of** zinc-galvanized steel and had a thickness of 3mm. **Once again, the choice of using one bolt to link the steel plates in both faces of the connections was made based on the spacing limits recommended by Eurocode 5 [2004]. The adoption of a plate with a 3mm thickness was made to avoid rupture of the same plate, providing for the adoption of a thin plate**. Due to the high price of the custom plates, it was decided to use rectangular commercial plates for timber frame walls without infill, as shown in Fig. 4b, adopting two solutions for the steel plates, i.e. linking the diagonals to the main frame as done for **timber frame** walls with masonry infill and linking only the main members of the connections. Perforated plates (Rothoblaas plates PF703085 (140×400mm) and PF703035 (80×300mm) [Rothofixing, 2012]) made of steel S 250 GD and having a thickness of 2mm were chosen. **Moreover, as one plate was not sufficient to cover one connection, plates had to partially overlapped to create the superposition along the most stressed element, in order to provide additional strength. Apart from bolts, for which the same spacing limits adopted for custom plates were applied, screws were used to better distributed the stresses in the plates (type PF603550 screws from Rothoblaas**

[Rothofixing, 2012]**), having a diameter of 5mm and a length of 50mm. The screws present a round head with a cylindrical underhead and are especially designed to be used with these steel plates. The number of screws adopted depended on recommendations of Eurocode 5 [2004].** Both types of steel plates require low technical equipment and non-specialized workmanship.

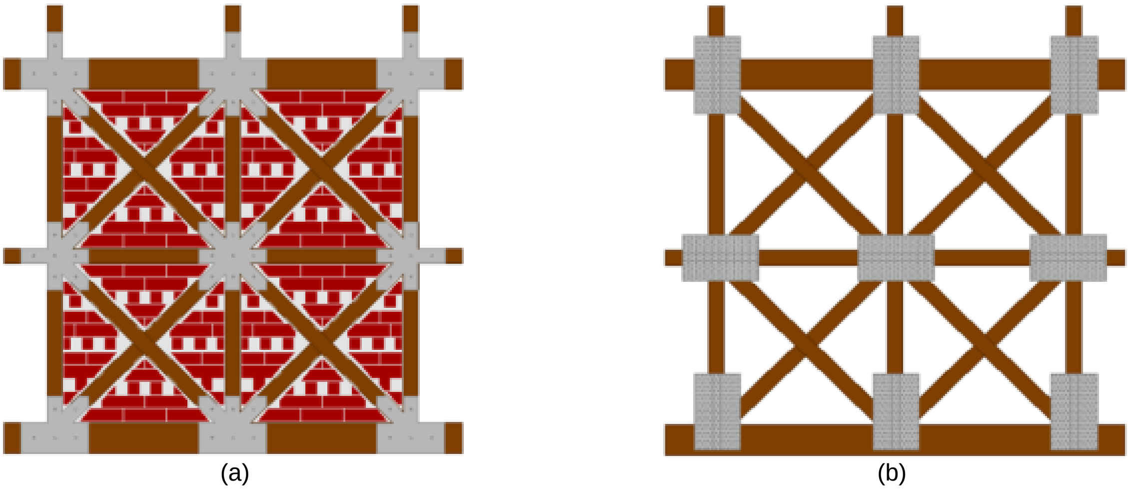


Fig. 4 Strengthening with steel plates: (a) custom plates for **timber frame** walls **with masonry infill**; (b) commercial plates for timber frame walls **without infill**

It should be stressed that both retrofitting techniques were applied after the specimens were previously submitted to lateral cyclic tests, meaning that the retrofitting was made for certain levels of damage induced in the walls. This situation simulates the retrofitting that can be made after the occurrence of an earthquake.

2.2 Material properties

In order to be able to better assess the behaviour of the walls during the cyclic tests, it is important to have an idea about the strength and mechanical behaviour of the materials, namely wood, mortar, masonry infill and steel adopted.

Wood, mortar and masonry had already been characterized for the previous tests [Poletti, 2013]. A summary of the results on the mechanical characteristics of the materials (wood, bricks, mortar and masonry) is presented in Table 1. Considering the materials used for strengthening, only steel plates were characterized in tension according to standard BS EN 10002-1 [2001]. It was assumed that steel bolts followed the requirements of steel class 8.8.

Table 1 Values of mechanical characteristics for materials used

Material	Compressive strength [MPa]	Bending strength [MPa]	Modulus of elasticity [GPa]	Standard
Wood	38,18 (// to the grain)	47,84	10,82 (global in bending) 12,02 (local in bending) 11,04 (in compression)	EN 408 [2003]
Mortar	4,01	1,58	-	EN 1015-11 [1999]
Bricks	34,5	-	-	EN 772-1 [2000]
Masonry	7,73	-	4,55	EN 1052-1 [1999]

Custom steel plates made of zinc-galvanized steel present an ultimate strength of 321.4MPa (c.o.v. 1.62%), with a percentage of elongation after fracture of 41.7% (c.o.v. 5.54%). The commercial rectangular steel plates made of S250GD steel present an ultimate strength of 477.2MPa (c.o.v. 1.85%), with a percentage of elongation after fracture of 3.5% (c.o.v. 14.29%). The difference in the elongation capacity of both types of plates is attributed to the holes of commercial plates, which constituted preferential points of failure in the plate, acting also in detriment of its deformation.

2.3 Test setup and instrumentation

The cyclic tests were carried out using the setup illustrated in Fig. 5. The application of the vertical load was made by means of vertical hydraulic actuators applied directly on the three posts. The actuators were connected to hinges welded in the bottom steel beam through steel rods, thus following the horizontal movement of the wall.

The horizontal displacement was applied to the top timber beam through a hydraulic servo-actuator with a maximum capacity in terms of displacement and load of 200mm and 250kN respectively. The out-of-plane displacements were prevented by a guide created in the upper beam through lateral steel rollers. For a detailed description of the test setup, see Poletti [2013].

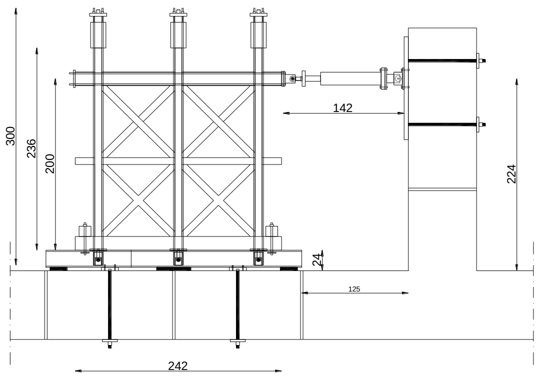


Fig. 5 Test setup used for cyclic tests

All of the walls were instrumented with linear voltage displacement transducers (LVDTs), placed in strategic positions to capture the global and local deformations of the walls, see Fig. 6. The horizontal displacements at the top (through TOP LVDT and control LVDT) and mid height beam (through MR and ML LVDTs) were measured on both sides of the wall. The vertical uplift of the three bottom connections were monitored through LVDTs BR, BM and BL.

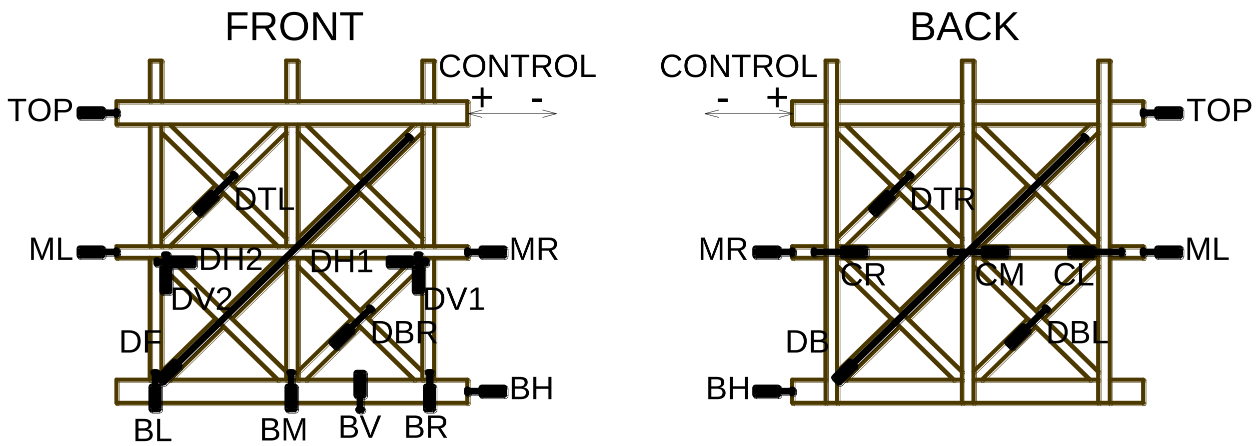


Fig. 6 Instrumentation in walls tested

The displacement in the diagonals is measured with LDVTS DF and DB at the front and back of the wall, in order to evaluate the effect of the compressive and tensile cycles as well as differences in both sides of the wall. The rest of the LVDTs were positioned strategically to measure the local opening of the connections.

In order to understand the efficiency of the strengthening materials and their mechanical behaviour during the tests, strain gauges were used in strategic places on the steel plates.

2.4 Vertical loading, cyclic procedure and number of specimens

The pre-compression loads applied were the same ones used in the unreinforced tests [Poletti, 2013], namely 25kN/post and 50kN/post, corresponding to the load calculated based on a building of three floors above the wall (4 floors in total) [Eurocode 1, 2002], and to an additional vertical load level respectively. The application of different vertical load levels aimed at assessing the influence of this variable in the lateral response, considering a possible change in use of the structure.

The cyclic procedure used for the retrofitted tests was the same one adopted for the previous tests [Poletti, 2013], following the recommendation of standard ISO DIS 21581 [2009]. In order to better capture the highly non-linear behaviour of the walls, additional steps were added in the procedure, considering an increment in the applied displacement of 10% (see Fig. 7).

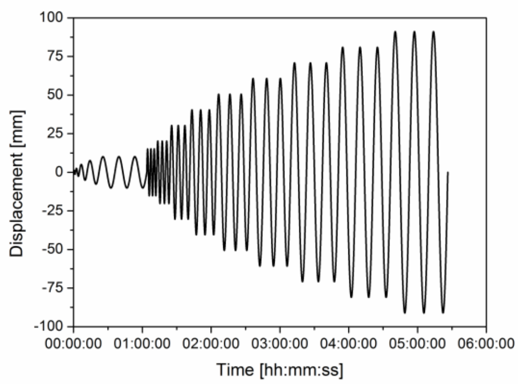


Fig. 7 Test procedure used in the experimental campaign of timber shear walls

In total, eight strengthened walls were tested, distributed in two distinct groups, see Table 2, according to the type of infill: (1) walls named as RIW with brick masonry infill; (2) walls named as RTW in which no infill was considered. The number 25 or 50 used in each designation is associated to the vertical load applied. The type of strengthening is given by the last letter and is divided in three groups: (1) bolts (letter B); (2) custom steel plates (letter P); (3) commercial steel plates (letters P_M).

To make the comparison with unreinforced walls easier, UIW designates unreinforced timber frame with masonry infill and UTW unreinforced timber frame walls, with the suffix 25 or 50 indicating the vertical load level. The decision to test only one specimen was based on the scarce availability of specimens and on the results obtained in the unreinforced tests, given the low variation in terms of lateral resistance and deformation.

Table 2 Typology of the specimens tested under cyclic loading

Specimen	Vertical load		Type of infill		Type of strengthening		
	25kN/post	50kN/post	Brick masonry	No infill	Bolts	Custom steel plates	Commercial steel plates
RIW25_B	✓		✓		✓		
RIW50_B		✓	✓		✓		
RIW25_P	✓		✓			✓	
RTW25_P	✓			✓			✓
RTW25_P_M	✓			✓			✓
RIW50_P		✓	✓			✓	
RTW50_P		✓		✓			✓
RTW50_P_M		✓		✓			✓

3 Analysis of test results

Cyclic test results performed on **timber frame walls with and without brick masonry infill** are here presented in this section and a discussion on the performance of the retrofitting techniques adopted is reported. The presentation and discussion of the results can be divided into three parts, namely: (1) discussion of the typical force-displacements hysteresis diagrams; (2) discussion of the main deformation features and typical failure modes; (3) assessment of seismic performance indicators.

3.1 Typical hysteretic diagrams

In this section the hysteretic diagrams of the retrofitted walls tested are presented, together with the vertical uplift of the bottom connections, in order to better understand the behaviour of the walls. In the unreinforced condition the walls had a strong flexural behaviour when tested with the lower vertical pre-compression load, characterized by rocking of the walls and consequent uplifting of the vertical posts. The walls tested with the higher pre-compression load exhibited a composite flexural-shear resisting mechanism [Poletti, 2013]. The retrofitting techniques adopted aimed at improving the performance of the walls under cyclic loading by limiting the rocking mechanism and improving their resistance, ductility and energy dissipation.

Comparing the hysteretic behaviour of the walls strengthened with bolts and the corresponding unreinforced **timber frame** walls with masonry infill, it is observed that there is no great gain in terms of ultimate capacity and stiffness. In fact, for the lower vertical load level, the gain in terms of maximum load was of 23.7%, whereas for the higher vertical load level it lost 5%. In terms of ultimate displacement, the walls gained 5.7% and 0.2% respectively (Fig. 8). The very low effectiveness of bolts as a retrofitting technique in **timber frame** walls can be attributed to the predominant flexural behaviour. In fact, bolts are not completely efficient in resisting the tensile stresses induced by cyclic loading at the bottom connections, being possible to observe practically the same damage patterns, i.e., tearing off of the beam-post half-lap connections.

In spite of this, for both load cases, the shape of the hysteretic loops experiences some changes. The plateau caused by the uplifting and recovering of the vertical post from the bottom beam [Poletti, 2013] is still present, but it is less pronounced and the unloading branch of the cycles is smoother. The vertical uplifting of the posts decreased of approximately 40% for both load cases, resulting from the lower predominance of the flexural resistant mechanism and from the contribution of a certain shear resistant component. Even in a reduced scale, the bolts contributed to the resistance to tensile forces developing in the bottom half-lap connections, and ensured a more remarkable post-peak behaviour enabling the connections to work until failure, contrarily to unreinforced walls, where after a certain lateral drift no contact was observed between the post and the bottom beam.

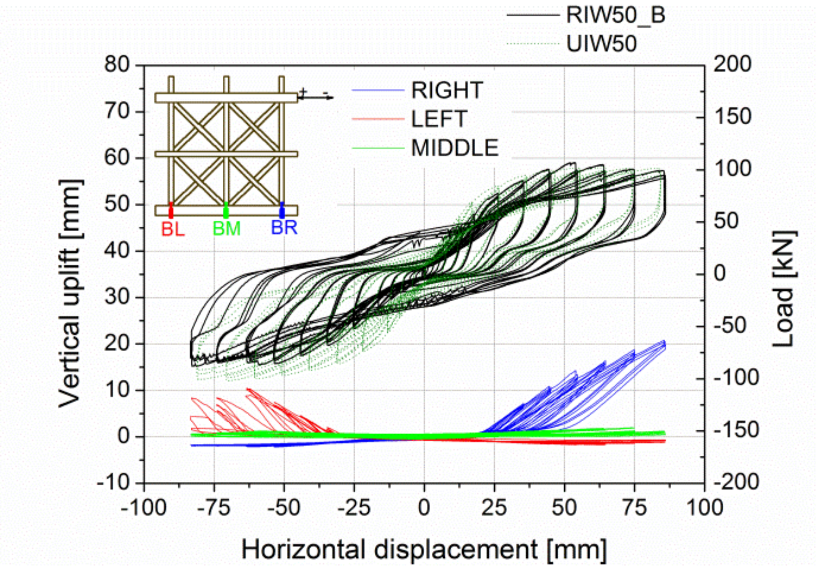
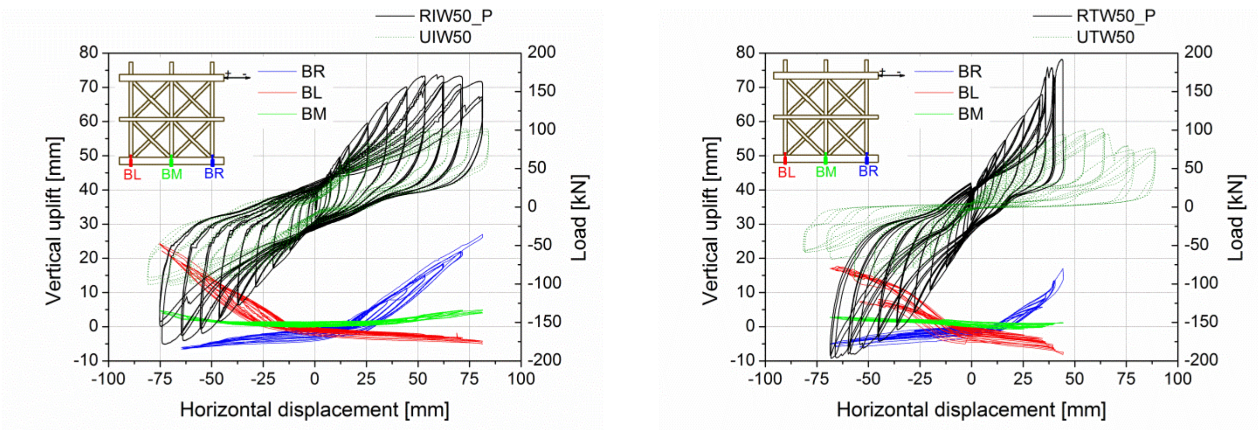


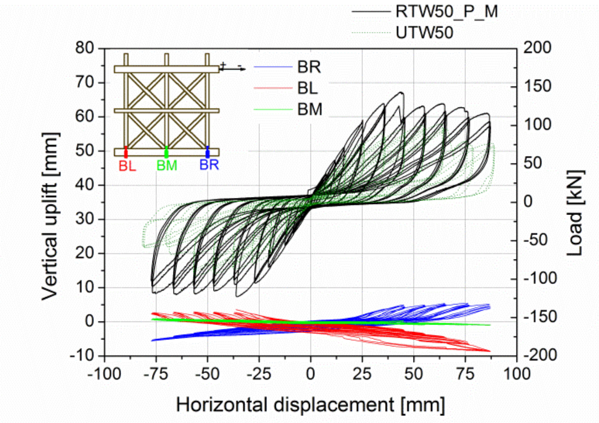
Fig. 8 Hysteretic diagrams for walls strengthened with bolts, higher pre-compression load

The comparison between the hysteresis diagrams found in unreinforced walls and after retrofitting with steel plates can be made through the analysis of Fig. 9, where results are shown for the walls tested with the higher vertical pre-compression. Walls retrofitted with steel plates experienced a similar behaviour independently on the vertical load level. For both timber frame walls with masonry infill, an important increase in terms of load capacity and stiffness was recorded: the maximum lateral load increased by 147% for the lower vertical load and by 60.4% for the higher vertical load. The initial stiffness of the walls increased by 30% when compared to the unreinforced solution for the lower vertical load level and by 14% for the higher vertical load level. The displacement imposed to the walls does not correspond to its maximum displacement capacity as it was not possible to obtain the complete failure mode of the walls, even if, due to the high levels of lateral load reached, the walls exhibited some out-of-plane displacement, reaching values of 6mm at mid height of the wall. The high stiffening effect of custom steel plates, linking the main elements of the connection (post and beam) to the diagonals, together with the slenderness of the wall led to this out-of-plane component, even if it was considered minimal. The ultimate state would be achieved if further lateral displacements were applied.

For this type of strengthening, the values of initial lateral stiffness are comparable for the two vertical load levels, meaning that for such a strong retrofitting technique, the effect on the amount of vertical load becomes secondary.



(a) (b)



(c)

Fig. 9 Strengthening with steel plates: (a) **infill timber frame** wall, higher vertical load; (b) timber frame wall with plates linking the diagonals, higher vertical load; (c) timber frame wall with plates linking main elements only, higher vertical load.

The lateral cyclic behaviour obtained for timber frame walls retrofitted with commercial plates linking the main members (post and beams) with the diagonals is shown in Fig. 9b.. Notice that the higher number of fasteners in a connection should be favourable from the point of view of ductility [EC8, 2004]. From the analysis of results, it is observed that this retrofitting configuration led to considerable out-of-plane behaviour, mainly in the positive direction, resulting in the instability of measurements of the in-plane response for both pre-compression levels**, even though the walls were restrained against out-of-plane movements with an additional testing device applied at the top beam**. This out-of-plane deformation was mainly due to the stiffening of the walls and to the remarkable increase of lateral strength resulting in higher levels of compressive stresses conducted by the diagonal elements, promoting the development of second-order effects. From the results obtained, it appears that this type of retrofitting is too **stiff and not ductile enough** for timber frame walls without infill, increasing significantly the lateral resistance (over 200% for the lower vertical load level and 97% for the higher vertical load when compared to the equivalent unreinforced wall) and the stiffness of the walls (77% for the lower vertical load and 50% for the higher one). This configuration adopted for the steel plates prevented severely the movement of the diagonals, which is a deformability feature of the unreinforced timber frame walls [Poletti, 2013].

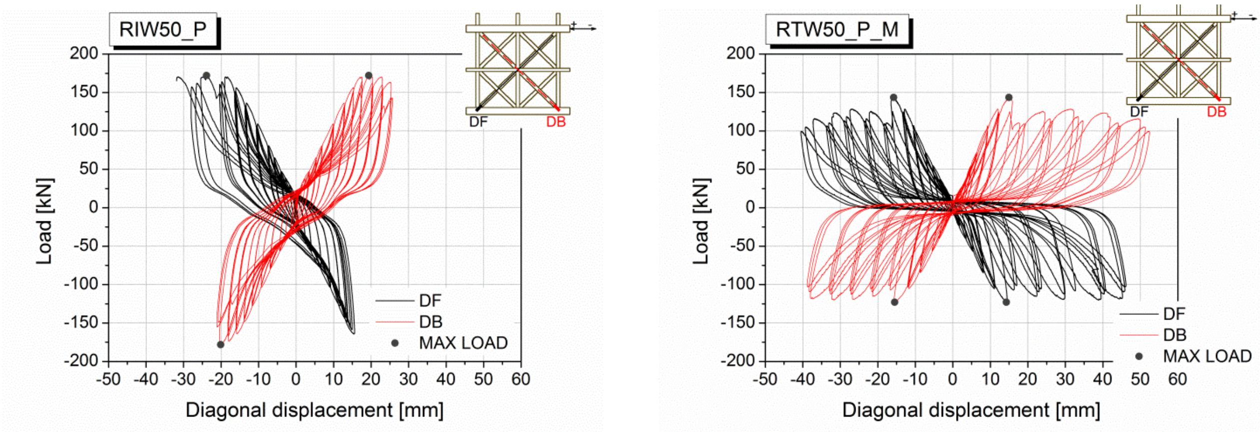
Therefore, in order to avoid this behaviour it was decided to adopt the same type of strengthening but without linking the diagonals, i.e. the steel plates were in the same position as in the previous tests, but the bolts and screws linked only the main members (posts and beams) allowing the free detachment of the diagonals, keeping the deformation to the diagonals free. This solution allowed the walls to gain significantly both in terms of stiffness and load capacity, without compromising the displacement capacity (see Fig.9c). In fact, in terms of maximum load, the walls gained 183% and 35% for the lower and higher pre-compression load respectively and experienced a minimal reduction of 5% and 3.5% in terms of ultimate displacement respectively. On the other hand, this retrofitting solution led to remarkable pinching in the timber walls. Similarly to the retrofitting with custom plates, the vertical load has only marginal influence in terms of maximum load, even if it influences the initial stiffness, being higher for the higher vertical pre-compression. This solution is therefore more appropriate for timber frame walls, since its stiffening effect is not overwhelming.

Comparing the two retrofitting solutions, bolts were able to improve the overall behaviour of the wall in terms of deformation capacity and post-peak behaviour, but it is not relevant in the increase on the lateral strength. On the other hand, the appropriate steel plates configuration is able to guarantee a better seismic response of the walls both in terms of stiffness and load capacity.

3.2 Deformational features of the walls

Besides the uplift of the posts analysed previously, some other deformational features are also here analysed in order to better understand the lateral behaviour of the distinct walls.

From the displacement measured on the diagonals (Fig. 10) it is possible to understand the stiffening effect of each retrofitting technique. For all walls, the diagonals deformed moderately until failure (diagonal displacements in the order of 10 to 15mm), similarly to what happened in unreinforced timber frame with masonry infill [Poletti, 2013]. For the stiffer retrofitting solution (steel plates linking the diagonals to the main frame), the deformation of the wall was moderate even after failure, achieving values up to approximately 30mm (Fig. 10a). Higher values of diagonals movement are reached only if there is complete failure of one element.

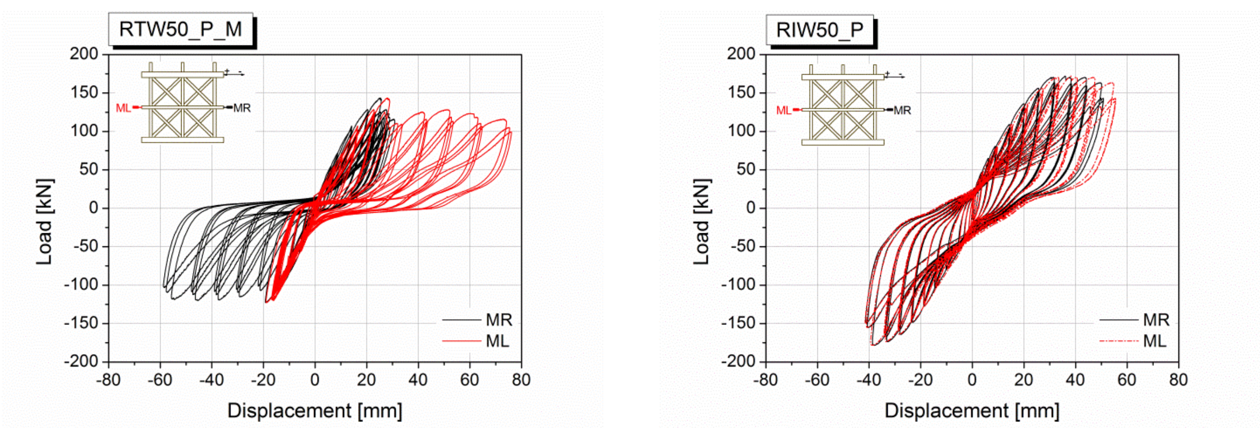


(a) (b)

Fig. 10 Diagonal displacement in timber frame walls retrofitted with steel plates for higher vertical load level: (a) with masonry infill; (b) without masonry infill (only main elements are connected with the steel plates).

For the less stiffening solutions (bolts and steel plates not linking the diagonals), a similar behaviour was observed. After failure of the central connection (sometimes the lateral one of the central beam also failed), an increase in the crack opening led to higher elongations of the diagonals, reaching values of about 45mm for bolts strengthening and 50mm for steel plates (Fig. 10b).

The same conclusions can be drawn from the analysis of the horizontal displacement at mid height of the wall (Fig. 11). For all walls where the diagonals were not linked to the main elements (i.e. bolts and second configuration of steel plates), the two sides of the walls experienced similar displacements up to failure, and an almost displacement linear profile was obtained. After failure, the displacements at mid height increased significantly and became asymmetrical when comparing both sides of the wall (Fig. 11a). In fact, with the shear crack opening at the central connection it is not possible to have a full displacement transfer between border posts.



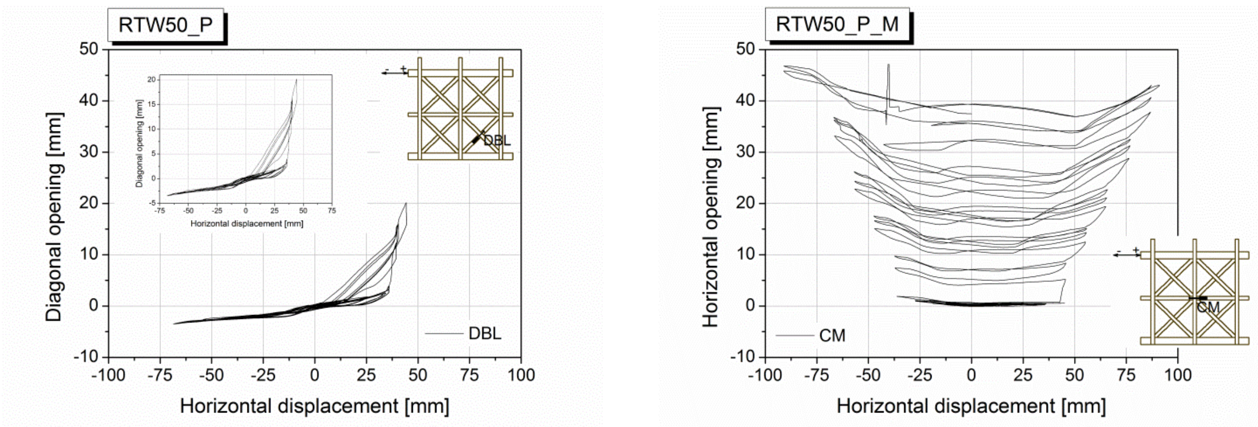
(a) (b)

Fig. 11 Displacement at mid height: (a) wall RTW50_P_M; (b) wall RIW50_P

For the stiffer solutions (Fig. 11b), the displacement on the two sides of the wall was similar, with a small tendency to deform more on one side after failure occurred. The displacements recorded were generally higher than half of the displacement applied to the top of the wall, meaning that the deformation of the walls does not result exclusively from the rotation of the walls but results from the deformation associated to flexure and shear of the wall.

The distinct deformation of the walls retrofitted with different configuration for the steel plates can be also observed in the openings recorded in other connections, see Fig. 12. The failure in the diagonal half-lap connection of wall RTW50_P (Fig. 12a) led to an opening of approximately 20mm (either in infill or timber frame walls). Instead, the displacement recorded at the half-lap connections of the main frame was minimal, not reaching 5mm, confirming the more rigid response of the wall.

The central connection of wall RTW50_P_M (Fig. 12b) experienced low openings until failure (up to 4mm) similarly to what was experienced in unreinforced **timber frame** walls [Poletti, 2013]. After failure the opening of the connection increased progressively, reaching an opening of 47mm. Nonetheless, the connection was still able to work since the steel plates kept the timber elements together. In order to see the damage level to which the wall had been subjected, it was necessary to take out the steel plates, since no damage was visible otherwise. A similar behaviour was observed in walls retrofitted with bolts, which experienced even higher openings (over 50mm), since the contribution of the steel plates was not present.



(a) (b)

Fig. 12 Opening of connections: (a) diagonal opening of half-lap connection in wall RTW50_P; (b) central middle connection of wall RTW50_P_M

To understand the efficiency of the retrofitting techniques adopted, in particular to understand the level of effectiveness, strain gauges were applied to the steel elements used in the reinforcement techniques. In general, strain gauges applied to the steel plates recorded small deformations in the plates, usually under 1.5‰ (Fig. 13). The main deformation in the steel plates consisted of the ovalization of the holes where the bolts were inserted and buckling of the plates which could not be prevented by the screws, since they failed in shear. This local deformation was responsible for the higher pinching behaviour found when steel plates were used.

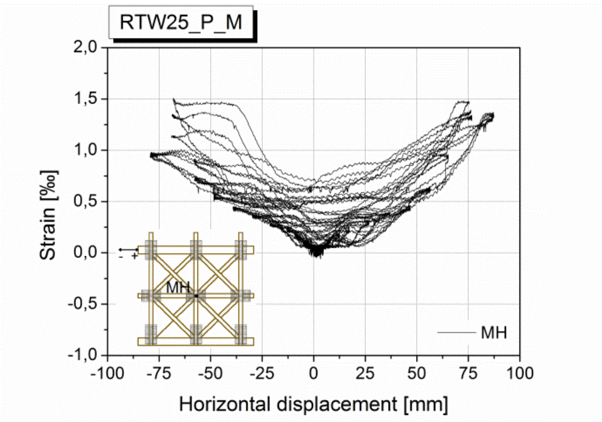


Fig. 13 Strain gauges recording at steel plate in central connection of wall RTW25_P_M

3.3 Typical damage patterns

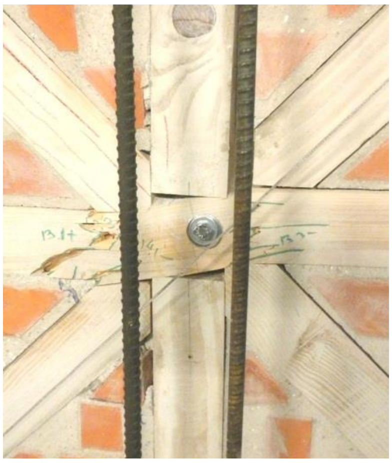
The distinct deformational features of the walls discussed previously resulted from distinct damage patterns exhibited by the different walls. The typology of strengthening is particularly relevant in the damage patterns when timber frame walls **with and without infill** are compared.

Walls retrofitted with bolts exhibited severe damages for both vertical load levels. The walls experienced damages in the central connections until their failure. The nailed diagonals detached from the main frame. The central beam tore off (Fig. 14a) in tension and the central post crushed due to the shear induced by the diagonals, similarly to what happened in the unreinforced tests [Poletti, 2013].

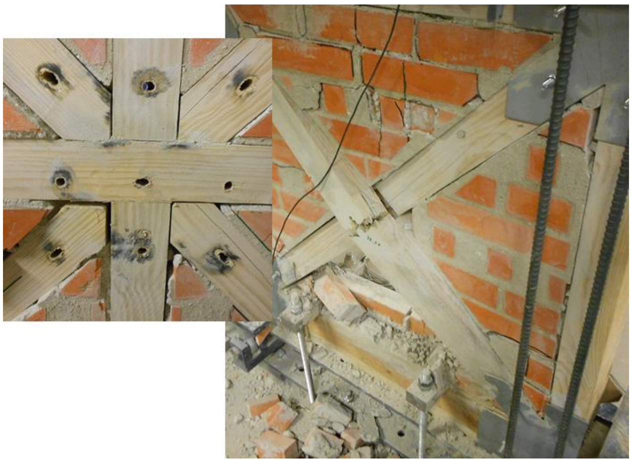
As already mentioned, in case of walls retrofitted with steel plates the damages observed were similar for all walls and they consisted on: (1) failure of the half-lap connection linking two diagonal members when steel plates linked the diagonals to the main frame; (2) failure of the central middle connection when the diagonals were not linked to the main frame through the steel plates.

The failure of the half-lap connection of the diagonal elements occurred in all specimens, independently on the type of plate, because this type of retrofitting stiffened excessively the connections, not allowing free movement to the bracing elements. The strong retrofitting of the post-beam half-lap connections in combination with the increase on the stresses carried by the diagonal bars resulted in the failure of the weakest zones of the wall, which were the half-lap connection of the diagonals. Notice that no damages were observed in the main wood members of the connection. An example of this type of failure is given in Fig. 14b for RIW50_P. An ovalization of the holes for the bolts in the diagonals was also observed, since these elements were particularly stressed.

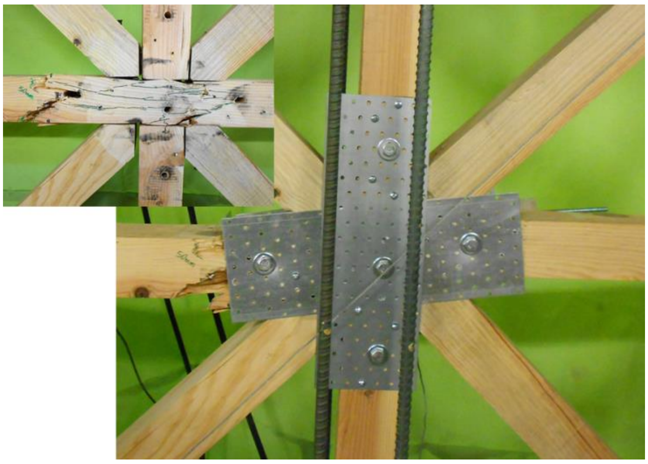
When the diagonals are free to move, the failure occurs in the main members of the frame. In both specimens tested with this retrofitting scheme, failure occurred in the central middle connection (Fig. 14c) due to the shear action imposed by the diagonal elements. For the higher vertical load, the failure propagated along the horizontal beam in alignment with the bolt, due to the presence of a knot. From Fig. 14c it is seen that the rotation of the steel plate during the test is clearly visible. The rotation was associated to the non-deformation of the steel plate and to the shear failure of the screws resulting from their shear resisting mechanism against this rotation.



(a)



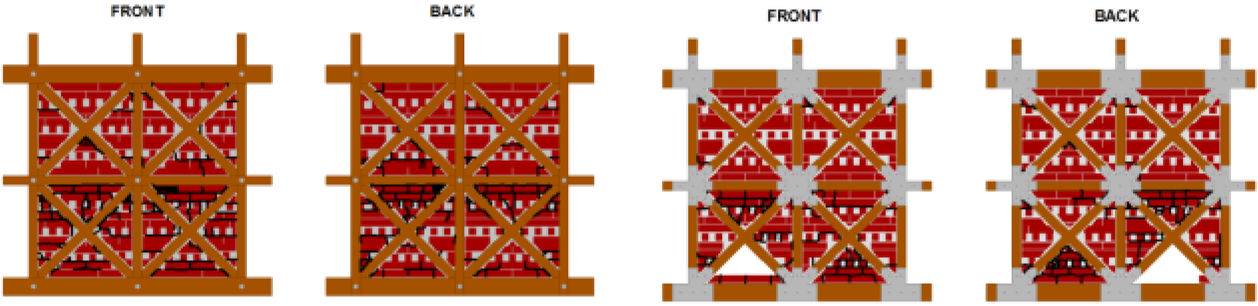
(b)



(c)

Fig. 14 Typical damages in walls: (a) tearing off of central beam RIW25_B; (b) failure of half-lap connection in bottom cell in RIW50_P; (c) failure of central connection in RTW50_P_M

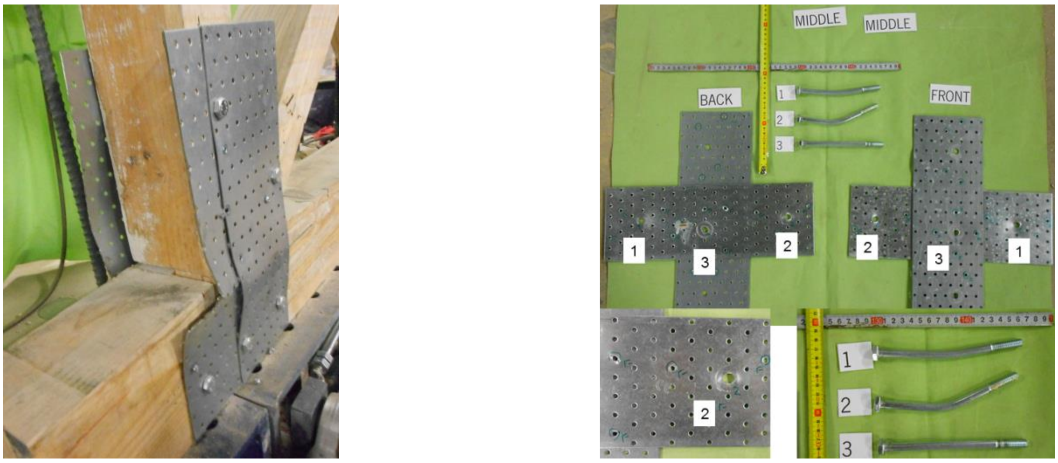
In all **timber frame** walls with masonry infill, damage was also observed in the brick masonry infill, with cracking mostly developed at the unit-mortar interface, detachment of masonry from the main frame and out-of-plane rotation of the masonry blocks, see Fig. 15. The damages were concentrated at the bottom half of the wall, as happened in unreinforced walls [Poletti, 2013], but they propagated even in the upper part of the walls, particularly for walls strengthened with bolts (Fig. 15a). In case of **timber frame** walls with masonry infill retrofitted with steel plates the complete detachment of some masonry blocks was observed (Fig. 15b).



(a) (b)

Fig. 15 Crack pattern in **infill timber frame** walls: (a) bolts strengthening; (b) strengthening with steel plates

The steel plates, both custom and commercial, did not exhibit significant deformations during tests. Only the plates located at the bottom connections tended to buckle (Fig. 16a) due to the elongation and compression during the test to which they were subjected. However, the holes accommodating the bolts were ovalized (Fig. 16b) due to bolt deformation during the test. In fact, all bolts used in the bottom and central connections, either as a standalone solution or together with steel plates, presented important deformations. It has to be noticed that for all bolts the deformation happens at a length of approximately 6cm, which is exactly the thickness of half connection, i.e. where the two elements are in contact. When analysing the connections after demolishing the walls, the holes in the timber elements of the posts, i.e. the elements that were uplifting, presented severe ovalization.



(a) (b)

Fig. 16 Deformations in strengthening elements: (a) buckling of bottom steel plates; (b) damages in commercial steel plate used in wall RTW50_P_M in the central connection with ovalized holes and deformed bolts.

3.4 Seismic performance

In the seismic design of new timber structures or in the rehabilitation of existing structures, including historic timber frame walls, the study of the seismic performance is of paramount importance. Since the seismic response of timber structures is very complex and time dependant, a better understating of the factors governing the problem is important for a safe and economical seismic design or for the adoption of the most adequate retrofitting measures. Parameters such as ductility, energy dissipation, overall cyclic stiffness, equivalent viscous damping ratio and lateral drifts characterize the behaviour of timber shear walls and are helpful in evaluating the performance of a structural element under cyclic loading. In this section, the main seismic parameters are presented and discussed for the walls previously analysed.

3.4.1 Bi-linear idealized diagrams and ductility evaluation

Aiming at obtaining the equivalent bilinear diagrams, which are a perfectly elasto-plastic representation of the actual response of the wall specimens, the monotonic envelopes for each wall tested were defined for both levels of vertical pre-compression, see Fig. 17. The monotonic envelope curves are defined as the curve connecting the points of maximum load in the hysteresis plot in each displacement level [ISO DIS 21581, 2009]. For both load cases, retrofitting technique with bolts provided the lower increase both in terms of stiffness and of load capacity.

The use of steel plates linking the diagonals to the main members of the wall gave similar results in terms of stiffness, load and displacement capacity, almost independently on the vertical load level and on the wall type (**timber frame wall with or without masonry infill**). This appears to indicate that the effect of retrofitting technique hinders the influence of both factors in the response of the walls. On the other hand, when the diagonals were not linked to the main elements of the frame (commercial plates), a lower stiffness was observed for both load cases, being more evident the coupling effect of the variation of the vertical loading and the application of the retrofitting. When comparing the two retrofitting techniques, it is clear that bolts strengthening does not improve the strength or stiffness of the wall but only its deformation. The improvement on the mechanical resistance when bolts are used as strengthening technique is more evident when single connections are tested [Poletti, 2013].

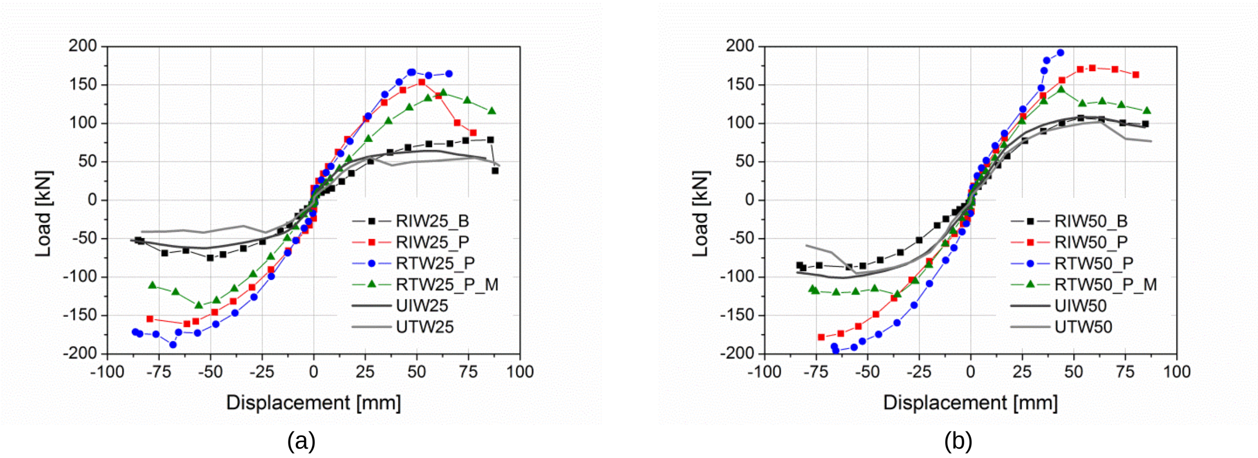


Fig. 17 Envelope curves of walls tested: (a) lower pre-compression load; (b) higher pre-compression load

According to Branco [2008], the improvement on the load capacity of a strengthened bird's mouth connection with bolts was of 147% and the maximum displacement was reduced of 19%. This seems to indicate that an improvement in the single connection cannot guarantee an equal improvement in the structural element where this connection is inserted. For the same type of connections, a strengthening with stirrups, comparable to the steel plates adopted for the walls resulted in an increase of the maximum load by 192% [Branco, 2008], a significant increase comparable to what occurred walls presented here.

In order to obtain the bi-linear diagram from the monotonic envelopes, the yield displacement has to be defined. **The curves were obtained using the method proposed by** Tomaževic [1999],

which considers the failure load as 80% of the maximum load and calculate the yield displacement from the equivalence of the energies enclosed under the bilinear and experimental envelopes (Fig. 18a). It should be stressed that for the majority of the walls, the ultimate displacement corresponds to the maximum displacement obtained experimentally, since only one of the walls lost more than 20% of the maximum load in the degradation process, namely wall RIW25_P. Therefore, in the present work, the ultimate displacement corresponds to the displacement reached in the last cycle imposed to the walls.

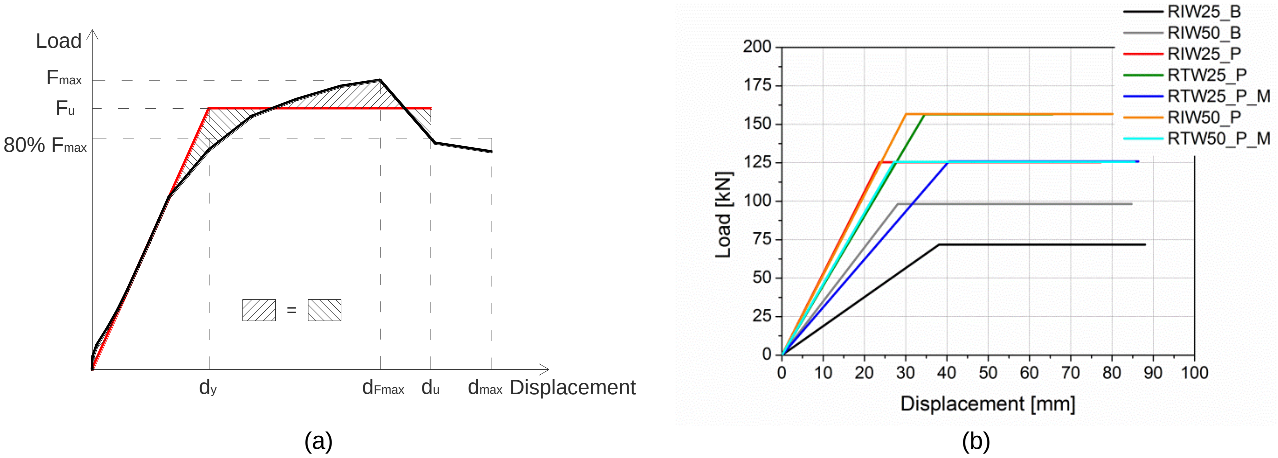


Fig. 18 Bilinear curves of walls tested: (a) method used to obtain bilinear; (b) bilinear idealizations

Fig. 18b presents the bilinear curves used to obtain the values of ductility for all walls. Only positive values are shown, since it was decided to take the positive displacements of the envelope as the reference for the calculation of seismic parameters.

Ductility is an important factor for the evaluation of the seismic behaviour of structures, as it is directly related to the ability of the structure to deform nonlinearly without significant loss of strength. Displacement ductility is defined here as the ratio between the ultimate displacement (d_u) and the yield displacement (d_y) defined in the equivalent bilinear diagram. Ductility μ_1 was calculated using the values of secant stiffness.

In general, values of ductility found for retrofitted walls were lower than the ones observed in unreinforced walls (Table 3). This is mainly related to the increase on the lateral strength and to the consequent increase on the yielding displacement, even for higher values of lateral stiffness. On the other hand, the strengthening with bolts resulted in low values of ductility due to the decrease on the lateral stiffness.

Table 3 Values of ductility

WALL	μ_{1+}	equivalent unreinforced wall
RIW25_B	2.31	5.20
RIW50_B	3.02	3.62
RIW25_P	2.54	
RIW25_P*	3.27	5.20
RIW25_P_2	2.49	
RIW50_P	2.66	3.62
RTW25_P	1.89	4.57
RTW50_P	-	3.53
RTW25_P_M	2.13	4.57
RTW50_P_M	3.14	3.53

Notes: μ_{1+} = ductility in positive direction

* without ultimate displacement limitation

Notice that the evaluation of the ductility is directly related to the stiffness and to the lateral strength as the yielding displacement used in the calculation of ductility is dependent on both variables.

The conjunction of these two effects with the limited imposed lateral displacement, which did not correspond to the collapse state of the walls, resulted in the decrease of ductility. Many walls did not reach a clear softening in their response or, if they did, the loss of strength was less than 20%. For these reasons, the values of ductility could increase if the tests could go further in terms of lateral drifts, as it is believed that the walls would assure higher levels of lateral drift. Nevertheless, it was decided to provide the ductility of the walls, even if they should be viewed as indicative in some cases. For example, the wall RTW50_P_M had a loss of strength of 19% at the end of the test, and its ductility is similar to one obtained in the unreinforced specimen, indicating that its final ductility should be of this order. Similarly, the wall RTW25_P_M reached a strength loss of 17%. For the other walls retrofitted with steel plates the ductility should not be viewed as a true value since they clearly show evidences of being able to withstand higher levels of lateral displacement. For example, the wall RIW50_P reached a strength loss of only 5%. In any case, it can be seen that the ductility values obtained for the retrofitted walls are associated to low level of damage.

An increase on the ductility between 158% and 316% pointed out by Cruz et al. [2001] in timber frame walls was recorded due to the possibility of applying greater displacements than in the unreinforced configuration. However, it should be noticed that these tests were performed on distinct specimens (only one cell) and in very different boundary conditions.

3.4.2 Evaluation of initial stiffness and stiffness degradation

According to European Standard ISO DIS 21581 [2009], the lateral stiffness of the walls may be calculated according to eq. 1:

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

where $\delta_{40\%F_{max}}$ and $\delta_{10\%F_{max}}$ are the displacements obtained in the envelope curve at 40% and 10% of the maximum load (F_{max}) respectively.

The consideration of the initial displacement corresponding to 10% of the maximum force should be associated to the need of overcoming some type of initial nonlinearity due to possible clearances. Notice that this factor is particularly relevant in case of traditional connections, as considerable nonlinear behaviour was recorded at very small values of lateral drift, which should be associated to the accommodations of the wall connections at the beginning of the tests. In this work, to overcome the initial nonlinear behaviour and to obtain a more adjusted linear branch to the monotonic envelopes, it was also decided to calculate the secant stiffness taking into account the origin and the point corresponding to 40% of the maximum load, ($K_{1,s+}$). All values of stiffness were calculated for the initial cycle.

The values of the secant stiffness, $K_{1,in+}$, and $K_{1,s+}$ are shown in Table 4. As expected, the values found for the secant stiffness $K_{1,s+}$ considering a secant stiffness from the origin up to 40% of the maximum load are greater than those of the standard initial stiffness because it softens the effect of the initial nonlinearity due to the initial adjustment of the traditional walls connections. The values are nonetheless of the same order.

Strengthening made with bolts is considered to be a very soft intervention and does not reflect any improvement in the stiffness or in the capacity of the wall. The loss in terms of initial stiffness was of 46% and of 21% for the lower and higher level of the pre-compression load respectively with respect to the unreinforced timber walls. Notice that the retrofitted walls had already been tested and in this case the repaired walls did not reach the same condition as the initial wall. However, the retrofitting with bolts allows the full exploitation of the connection.

In all walls retrofitted with steel plates higher initial stiffness was recorded when compared to unreinforced walls [Poletti, 2013], particularly in case of timber frame walls without infill masonry. Indeed, this solution increased the initial stiffness of **timber frame** walls with masonry infill by 31% and by 14% when compared to the same unreinforced walls for the lower and the higher pre-compression load respectively. The gain was of 78% and 51% for timber frame walls submitted to the lower and to the higher pre-compression load respectively. With the use of commercial steel plates, not linking the diagonals to the main timber elements of the frame, the gain was lower, namely 30% and 28% respectively for the two load levels. The absence of confinement given by the infill in timber frame walls led to a high increase on of stiffness.

Table 4 Values of stiffness for walls tested with different retrofitting solutions

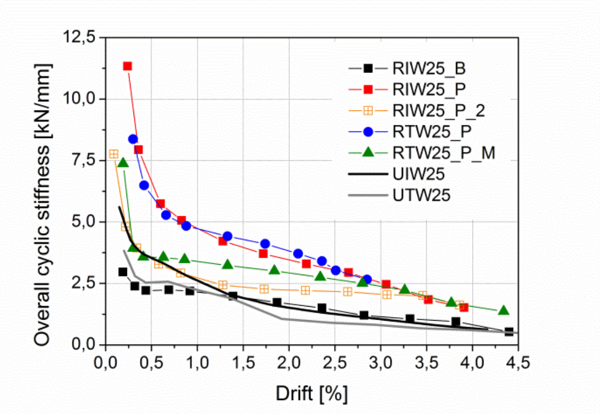
WALL	$K_{1,in+}$ [kN/mm]	$K_{1,s+}$ [kN/mm]
RIW25_B	1.63	1.89
RIW50_B	2.96	3.49
RIW25_P	3.98	5.30
*RIW25_P_2**	2.31	3.09
RIW50_P	4.28	5.21
RTW25_P	3.80	4.52
RTW50_P	4.76	5.69
RTW25_P_M	2.78	3.11
RTW50_P_M	4.06	4.62

Notes: *$K_{1,in+}$ = initial secant stiffness of first cycle in the positive direction; $K_{1,g+}$ = secant of first cycle in the positive direction; *test repetition*

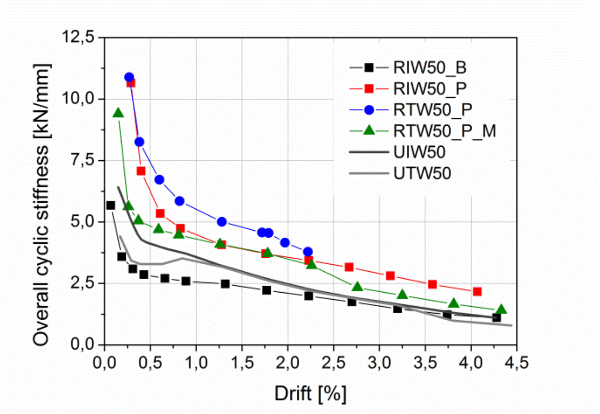
All walls retrofitted with steel plates exhibited similar values of initial stiffness, apart from wall RTW25_P_M, which was more in line with stiffness provided by bolts, as its retrofitting has a less confining effect and more dependent on the vertical load level.

In order to evaluate the degradation of stiffness experienced by the walls during the cyclic tests to a certain lateral drift, cyclic stiffness was calculated for each cycle considering the average of the slopes of the line connecting the origin and the two points of loading corresponding to the maximum (positive and negative) displacements, see Fig. 19. Due to the accommodations that occur in the wall for low values of drifts already mentioned, cyclic stiffness calculated for drift values lower than 0.15% was not considered reliable and thus it was not represented here. The lateral drift is calculated as the ratio between the lateral top displacement and the height at which the lateral load is applied. For all walls a dramatic loss of stiffness is found for values of drift lower than 0.5%, due to the accommodations in the walls at the beginning of the test.

For both vertical pre-compression loads, strengthening with bolts gave the lowest values of cyclic stiffness, as well as a lower rate of degradation. For low values of drift there was a decrease of cyclic stiffness of 47% and 12% for the lower and higher vertical load level respectively. For high values of drift, RIW25_B wall had a similar stiffness to the unreinforced one, while RIW50_B decreased its cyclic stiffness by 16%.



(a)



(b)

Fig. 19 Stiffness degradation: (a) lower pre-compression load; (b) higher pre-compression load

It is clearly visible that for strengthening carried out with steel plates linking the diagonals, the values of cyclic stiffness increased significantly, particularly for the lower pre-compression level. The amount of vertical pre-compression applied has little influence on the cyclic stiffness of the walls. In general, the stiffness was higher for walls with a higher pre-compression level, but the difference was minimal. For **timber frame** walls with masonry infill, the increase on the cyclic stiffness for low values of drift (from 0.2% to 0.4%) was of 102% and 66% for the lower and higher vertical load respectively, when compared to the unreinforced walls. For higher values of drift, the increase in the stiffness was of 140% and 96% for the lower and higher vertical load respectively, thus ensuring an important stiffness even when the walls are damaged. For timber frame walls with the alternative steel plate configuration (diagonals with free movements), the increase in the cyclic stiffness for initial values of drift was of 92% and 113% for the lower and higher vertical load and of 178% and 78% for higher values of drift.

3.4.3 Assessment of the ability to dissipate energy

Besides ductility and lateral drifts, one major parameter used for the assessment of the seismic performance of the seismic behaviour is the ability of a structural element to dissipate energy during cyclic testing. Here, the dissipation of energy per each cycle and the cumulative energy are considered. The energy dissipated by the walls at each cycle, E_D, is computed by calculating the area enclosed by the loop in the load-displacement diagram and it represents the amount of energy dissipated during the cyclic loading. The energy can be dissipated through friction in the connections, yielding of nails, yielding and deformation of the retrofitting bolts, steel plates and bars and permanent deformation accumulated in the walls as observed during the tests.

All retrofitting techniques adopted were able to guarantee greater energy dissipation during the tests (Fig. 20). The highest dissipative solution is provided by the retrofitting technique with steel plates linking the diagonals. **Timber frame** walls **with brick masonry infill** retrofitted with steel plates increased the total dissipated energy by 96% and 57% respectively for the lower and higher vertical load level. For the walls tested without linking the diagonals, the dissipative capacity was lower. In case of timber frame walls with this alternative steel plates configuration, the total dissipated energy increased by 132% and 38% respectively when compared to the equivalent unreinforced wall. The retrofitting with bolts showed results comparable to the ones obtained in unreinforced walls, improving only for high values of drift in case of the higher pre-compression load, given that the solution changed the failure mode of the wall, reducing the amount of pinching in the walls, guaranteeing a higher dissipative capacity of the wall (+14%).

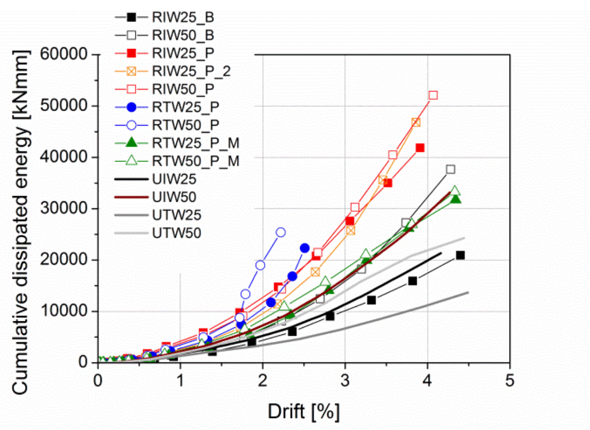


Fig. 20 Cumulative dissipated energy for all walls

3.4.4 Equivalent viscous damping

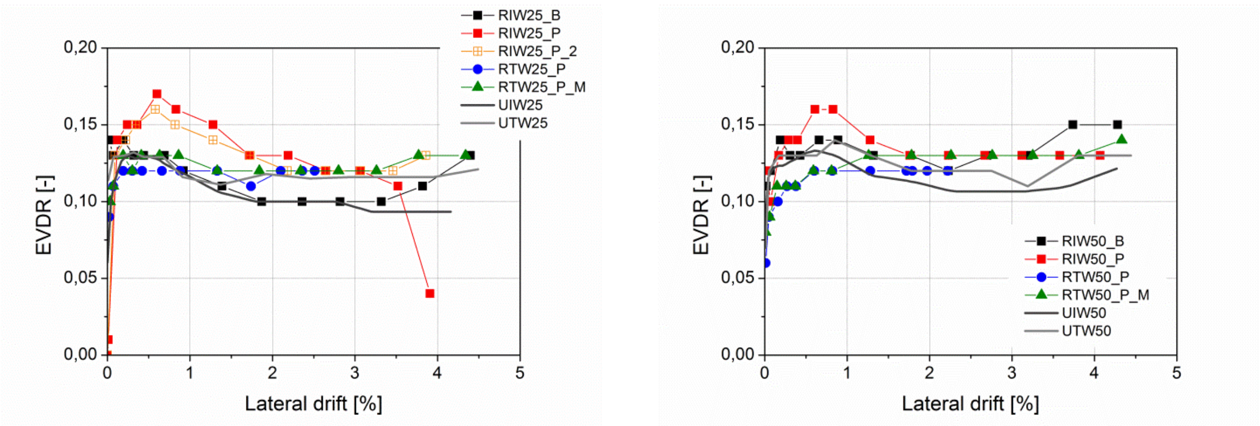
Damping is the process by which vibration steadily diminishes in amplitude [Chopra, 1995]. Damping diminishes the accumulation of energy of the structure through various mechanisms, such as, for the present case, friction in the connections and opening and closing of cracks. On the other hand, due to the hysteretic behaviour of the walls, it is possible that they dissipate energy during the cyclic response, particularly during the non-linear regime of the lateral behaviour. With this respect it is possible to calculate the equivalent viscous damping ratio (EVDR) correlating it to the energy dissipation in the nonlinear regime. The equivalent viscous damping (hysteretic 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 discussed above, E_e^+ and E_e^- are the elastic energies of an equivalent viscous system calculated as the area of the triangle formed at the maximum load and displacement in each loop for the positive and negative direction of loading respectively.

Comparing the results of equivalent viscous damping for the walls tested (Fig. 21a,b), the influence of the vertical pre-compression load was evident only for the strengthening with bolts. In the latter case the highest level of pre-compression leads to higher values of equivalent viscous damping than the wall submitted to the lower vertical pre-compression. In general the retrofitted walls present higher values of equivalent viscous damping. The walls retrofitted with bolts exhibit also higher values of hysteretic damping than the unreinforced walls for high levels of lateral drift in case of the walls submitted to the highest level com pre-compression. For the lower level of vertical load, the equivalent viscous damping is only higher for lateral drifts of 3%.

Walls with steel plates present a constant final equivalent viscous damping of 0.12 and 0.13 for the lower and higher pre-compression level respectively, with little variation among the walls. Similar values were found for cyclic tests on bird’s mouth connections [Branco, 2008]. This type of connections strengthened with bolts presents a value of EVDR of 0.11, while the connections strengthened with stirrups present a value of 0.15.



(a) (b)

Fig. 21 Equivalent viscous damping ratio: (a) lower pre-compression level; (b) higher pre-compression level

Comparing to similar tests conducted on strengthened concrete block masonry [Haach et al., 2010], where EVDR is increasing for high values of drift, in the case of timber frame walls the values tend to decrease or reach a constant value. Only walls strengthened with bolts encountered an increase in values when compared to the initial ones. This behaviour is due to pinching, which characterizes both timber frame and, in a smaller scale, **timber frame** walls **with masonry infill,** reducing the dissipative capacity of the walls.

4 Conclusions

Aiming at gathering a better insight on the influence of distinct retrofitting techniques applied on timber frame walls on the lateral behaviour of timber frame walls, characteristic of traditional construction in Portugal, and on the improvement of their seismic performance, an experimental campaign was designed based on static cyclic tests. Distinct parameters were considered, namely typology of the wall and vertical pre-compression load. Two distinct retrofitting solutions were adopted: (1) bolts and (2) steel plates. In case of steel plates, two distinct geometrical configurations were adopted. Besides, two vertical pre-compression levels were considered for each wall type.

From the detailed analysis of the experimental results it is possible to conclude that:

- The presence of masonry infill still influences the behaviour of the retrofitted walls, but not in the same level as in unreinforced tests [Poletti, 2013], since the retrofitting solutions play a predominant role on the lateral behaviour and hinder in a certain extent the influence of other factors, such as the vertical pre-compression.

- The increase on the vertical pre-compression load does not overmuch influence the behaviour of the retrofitted walls, mainly for the solutions that significantly change the stiffness of the walls. A dependency on the vertical load was observed only for the simplest and less intrusive retrofitting technique with bolts. For the other techniques, walls reached similar values of load, displacement and stiffness independently on the vertical load.
- The retrofitting with bolts improved the behaviour of the walls in the sense that it allowed to exploit the full capacity of the connections, changing the failure mechanisms and improving mainly the dissipative features of the walls, particularly for increasing lateral drifts, **with an improvement of 14% for the higher vertical load**. Additionally, it should be mentioned that this technique is the cheapest and the less intrusive when compared to steel plates. In any case, all retrofitted techniques have the advantage of being reversible.
- The retrofitting technique with steel plates increases considerably the stiffness of the walls, particularly when the diagonal elements were linked to the main frame. Besides, the use of this retrofitting technique led to an important increase in the lateral resistance of the wall (**between 50% and 200% depending on the wall type and vertical load level)**. The steel plates were able to guarantee a good behaviour of the walls even after peak load.
- There is a trend for the retrofitted walls to present a decrease of ductility for all walls. However, it should be stressed that for a great number of specimens the ultimate displacement capacity of the walls was not reached, indicating that it could be possible to obtain higher values.
- All retrofitting techniques ensured a higher dissipative capacity for the walls, with similar values for timber frame walls **with and without masonry infill**. The higher dissipative character of the retrofitted walls is revealed by both the cumulative energy dissipated for a given lateral drift and the equivalent viscous damping.
- **Retrofitting performed with steel plates appears to be more appropriate both for timber frame walls with masonry infill and without masonry infill. To ensure a better ductility of the wall, without compromising the dissipative capacity, it would be more appropriate**

not to link the diagonals to the main frame to prevent the excessive increase the stiffness of the connections.

- **Comparing the two types of retrofitting, the adoption of bolts as a per se is not recommended if a higher strength of the wall is needed. However, it should be noticed that this technique improves the post-peak behaviour of the walls leading to a more appropriate exploitation of the retrofitted connections.**

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