Experimental proof of the effectiveness of timber panels in the seismic retrofit of URM walls

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ABSTRACT

Recently there has been a growing interest by both academia and the industry in the possibility of using timber as a strengthening material for the seismic retrofit of unreinforced masonry (URM) buildings. This possibility was made genuine by the increased availability of new engineered wood products and connection systems which enable effective interaction between timber and masonry, as proved by the first preliminary numerical studies reported in the literature. Therefore, obtaining reliable experimental validation becomes the next step to turning the possibility into reality. This paper summarises the experimental evidence collected over the last five years by the research team at the University of Trento, Italy. The focus is given to solutions that use timber panels connected to the masonry walls with spread fasteners. The testing activity reported herein addresses all components of the strengthening system at different scales: from small-scale timber-to-masonry connections up to full-scale walls tested onsite in a century-old building.

1 INTRODUCTION

That timber can help masonry resist seismic shaking is not news. All around the world, from the Roman *Opus Graticium* to the *Gajola Pombalina* in Portugal through the *Dhajji-dewari* in India, the examples of traditional construction technologies successfully surviving even the strongest of earthquakes thanks to symbiotic interaction between timber and masonry, are numerous (Langenbach 2007).

The recent development of fasteners and connections specifically designed for use in quasi-brittle materials (i.e. concrete and masonry) and the increased availability of engineered wood products (e.g. Cross Laminated Timber CLT and Laminated Veneer Lumber LVL) kicked off new possibilities for using timber to strengthen unreinforced masonry (URM) structures. A quite straightforward and cost-effective solution to prevent the disastrous out-of-plane (OOP) collapse of URM walls often observed by post-event surveys could be applying timber strongbacks (Dizhur et al. 2017, Cassol et al. 2020). The technique sees vertical rib-like timber elements connected to the face of the masonry walls at regular spacing, enhancing the inertia and the wall's out-of-plane capacity. The next step would be creating a light timber frame structure, with the addition of horizontal blocking elements and sheathing panels, to improve also the in-plane wall capacity (Guerrini et al. 2021). By replacing the light frame structure with solid panels made of CLT or LVL (Borri et al. 2021, Giongo et al. 2021, and Scotta et al. 2021) the performance of the system can be pushed even further. Parallel to the structural performance, careful attention has been given by researchers to the thermal performance of timber-based retrofits, developing integrated solutions with clear advantages in terms of energy efficiency (Busselli et al. 2021, Valluzzi et al. 2021, and Zanni et al. 2021).

The present work summarises the experimental evidence obtained by the authors over the last few years on the effectiveness of solid timber panels used for the seismic retrofit of URM walls. The chapter sequence follows the research phases, from testing single connections to full-scale walls.

2 TESTING OF TIMBER-TO-MASONRY CONNECTIONS

2.1 Dry connections | Shear

The system selected to connect the strengthening timber panels to the existing URM walls consists of metallic dowel-type fasteners spread over the panel. The first category of dowel-type fasteners studied experimentally were dry screw anchors, which ensure fastening without the need for grout or adhesives. The performance under shear loading conditions was examined via monotonic, cyclic, and semicyclic testing. The monotonic tests were used primarily to calibrate the displacement amplitude for the cyclic test loops. The tests were carried out in a historic URM hotel building of the nineteenth century in northern Italy (Comano Terme bath area in the Trentino province). The experimental apparatus used for the campaign is shown in Figure 1a. A 75 kN load cell was used to measure the load applied by a two-way manually activated hydraulic actuator, while the displacement of the timber panels was captured by a linear variable displacement transducer (LVDT). A combination of multiple variables was considered, totalling 64 tests: five different screw anchors (varying for type, length, diameter, and steel grade), two types of masonry (clay bricks and irregular stone blocks), three types of timber panels (60 mm cross-laminated-timber CLT made of spruce and 40 mm laminated-veneer-lumber LVL made of spruce and beechwood) and three different load-to-grain directions (0°, 45° and 90° to the direction of the grain in the outer panel layers). More details on the tested materials and their reference mechanical properties can be found in Riccadonna et al. (2019). The shear capacity of the connection appeared to be mainly governed by the local failure of the masonry material and by timber embedment strength, whereas the fastener type was found to have a small impact in determining the connection strength and stiffness. For this reason, a condensed summary of the cyclic test results is reported in Table 1, separating the various combinations of masonry support and timber panel.

Table 1: Dry connections. Maximum strength and stiffness under cyclic shear loading (mean values)

Specimen	Number of tests*	F_{max} (kN)	CoV (%)	K_s (kN/mm)	CoV (%)
Brick – Spruce CLT	35	8.14	21.0%	0.97	52.1%
Stone – Spruce CLT	16	10.30	19.9%	1.46	94.5%
Brick – Beech LVL	16	10.43	39.8%	1.16	82.4%
Brick – Spruce LVL	16	7.76	25.5%	1.08	81.7%

* Tension and compression loading are taken into account separately

An observation worth mentioning regards the relationship between the fastener yielding moment and the connection energy dissipation under cyclic loading. Because of the intrinsic “brittle” nature of both masonry and timber, the cyclic load-displacement curves of the tested connection are characterised by a noticeable “pinching” effect due to the irreversible deformation experienced by both materials (Figure 1b). In the experimental campaign, the fasteners made of milder steel exhibited less pinching and consequently higher energy dissipation thanks to the formation of plastic hinges in the fastener shank at relatively low load levels.

Therefore, for low-strength masonry (similar to the tested clay brick masonry), it is advisable to use mild or low-grade steel to foster early activation of the plastic hinge and thus enhance the system energy dissipation.

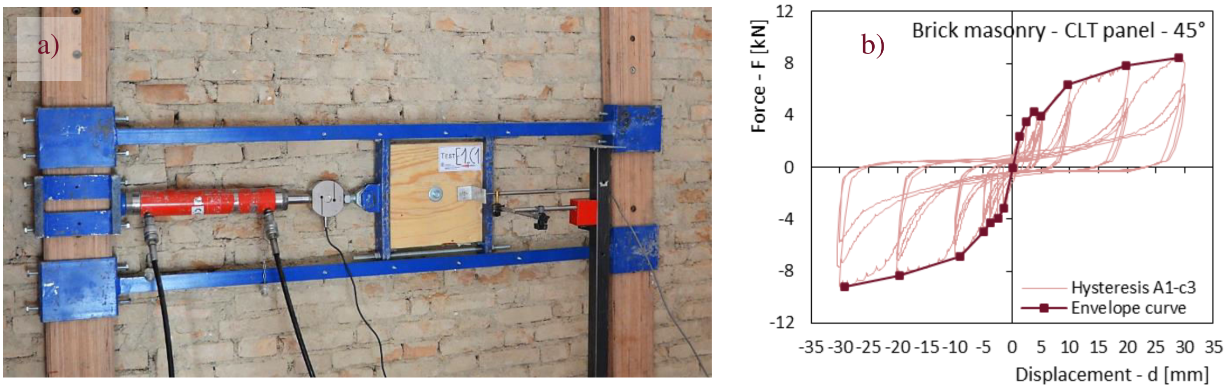

Figure 1: Dry connections. a) Monotonic and cyclic shear test configuration, b) Typical load-displacement hysteretic curve and associated envelope

2.2 Dry connections | Pull-out

A series of monotonic pull-out tests were performed to provide additional insight into the behaviour of dry connections when adopted for the seismic retrofit of URM walls. Information about the pull-out capacity of the anchors may prove helpful for studying the performance of the retrofit system against local out-of-plane failure mechanisms. In addition, contrarily to adhesive connections (Dizhur et al. 2013), limited information is available on the response to tension forces of screw fasteners installed into masonry. The pull-out tests were carried out on clay brick masonry in the same building used for the shear tests. The load was applied monotonically using a single-effect hydraulic actuator. Force and displacement were recorded using a 75 kN load cell and a linear variable displacement transducer (LVDT), respectively (experimental setup shown in Figure 2a-b).

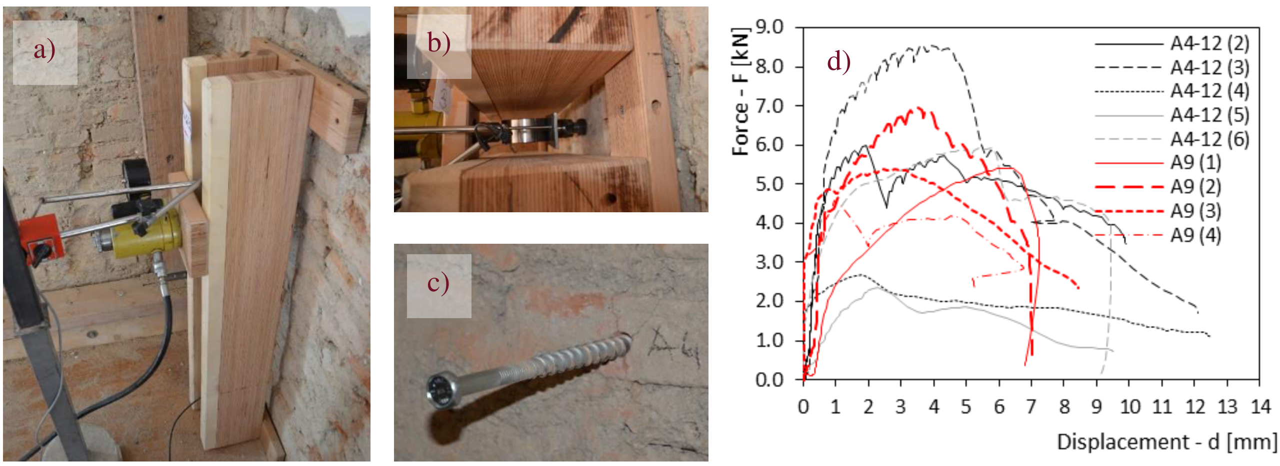

Figure 2: Dry connections. a-b) Monotonic pull-out test configuration, c) Damage at the end of the test, d) Load-displacement curves

Two among the fastener types tested in shear were selected for the pull-out tests. The fasteners have an equal core diameter (i.e. 9.4 mm) and thread length (i.e. 100 mm). The two fastener types differed in terms of thread diameter (12.0 mm for A4-12, 12.5 mm for A9), total length (180 mm for A4-12 and 160 mm for A9), and axial resistance (25 kN for A4-12 and 55 kN for A9). Such differences proved to have limited influence on the experimental results because of the negligible geometry variation and the little stress on the fastener when the connection failure was reached. Not surprisingly, the most recurrent failure mode was related to brick tensile cracking (Figure 2c). The lowest pull-out capacities (i.e. A4-12(4) and A4-12(5)) were influenced by the mortar crushing with a consequent rigid movement of the brick and by the withdrawal of the fastener thread due to the brick crumbling around the fastener. A total of 9 monotonic tests (Figure 2d) were performed, exhibiting an average peak capacity of 5.32 kN (coefficient of variation of 36%) at a corresponding displacement of 3.18 mm (coefficient of variation of 56%).

2.3 Adhesive connections | Shear

Adhesive connections (also known as "wet connections") were the second category of dowel-type fasteners studied with the onsite experimental investigation. Contrarily to the dry connections previously described, the bonding between the metallic fastener (e.g. threaded bar, rebar) and the masonry substrate is chemical. Wet connections were investigated to provide fixing opportunities in cases where: a) chances of having ineffective dry fasteners are high (e.g. in the presence of irregular masonry texture with thick air-lime mortar joints); b) transversal connection between masonry wythes is absent.

The performance under shear loading was examined via monotonic and semicyclic tests. The monotonic tests were used primarily to calibrate the displacement amplitude of the semicyclic test loops. Wet connection testing was performed on irregular stone masonry in the same building used for the dry connection testing. A 75 kN load cell was used to measure the load applied by a manually operated hydraulic actuator (Figure 3a), while the displacement of the timber panel was monitored with a linear variable displacement transducer (LVDT). The timber specimens were made of three-layered 60 mm thick spruce CLT for all tests, and the embedment depth of the steel rods was constant at 30 cm. Testing variables were the chemical composition of the adhesive (i.e. epoxy resin and hybrid epoxy-vinyl ester resin), the steel anchor type (8.8-grade threaded bars and B450C rebars) and diameter size (14 mm and 20 mm). The diameter used for pre-drilling the masonry was 22 mm for the 14 mm rods and 30 mm for the 20 mm rods. The CLT specimens were also pre-drilled, adopting the hole sizes used for pre-drilling the masonry. After removing the drilling debris and carefully cleaning the hole, a metallic sleeve (Figure 3b) was inserted for the full hole depth up to the interface with the timber panel to prevent leakage of the liquid adhesive during injection and curing.

Table 2 summarises all the semicyclic (tension cycles) test results in terms of strength and stiffness. The data are grouped based on the adhesive type and steel rod diameter, given that the type of steel anchor (i.e. 8.8 threaded rod or B450C rebar) had a minor influence on the connection performance under shear loading. As expected, larger fastener diameters led to an increase in both strength and stiffness, more evident in the case of the hybrid resin composition. The type of adhesive also had a significant impact on the mechanical response of the connection, with connections installed using epoxy resin showing greater strength and stiffness than those installed with hybrid resin. It is worth mentioning that hybrid resins require shorter curing time than pure epoxy resins (3 hours vs 24 hours) and are typically less expensive.

Table 2: Adhesive connections. Maximum strength and stiffness for semicyclic shear loading (mean values)

Specimen	Number of tests	F_{max} (kN)	CoV (%)	K_s (kN/mm)	CoV (%)
Hybrid resin Ø14	9	15.82	26.2%	2.82	66.6%
Epoxy resin Ø14	6	24.42	22.9%	8.76	42.8%
Hybrid resin Ø20	10	27.98	11.6%	5.10	41.4%
Epoxy resin Ø20	9	32.14	35.0%	9.43	61.4%

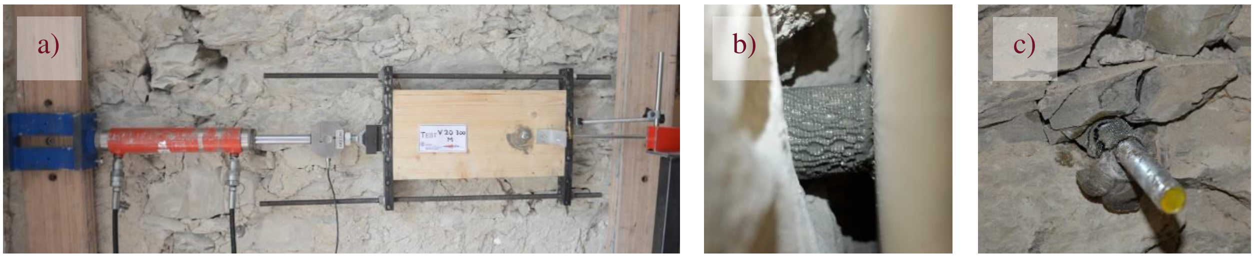

Figure 3: Adhesive connections. a) Monotonic and semicyclic shear test configuration, b) Metallic sleeve for preventing leakage of the adhesive, c) Damage at the end of the test

3 DIAGONAL COMPRESSION TESTING OF WALLETTES

The next step of the experimental campaign was moving from testing single connections to testing masonry wall portions strengthened with CLT fixed by multiple fasteners. Dolomite stone blocks and hydraulic lime mortar were used to build eight double-leaf 120 ×120 cm^2 40 cm thick masonry wallettes, with the goal of recreating conditions often typical in existing Italian masonry buildings. Each specimen was subjected to diagonal compression thanks to a self-balanced test setup based on the ASTM E519-15 (ASTM 2015) standard, using a 600 kN hydraulic actuator (Figure 4). The applied load was measured by a 600 kN load cell and the four diagonals of each specimen were instrumented using 100 mm wire displacement transducers. Four wallettes were tested in the "as-built" condition to provide a benchmark for evaluating the retrofit performance. The remaining four specimens were strengthened with a 60 mm-thick spruce CLT panel fixed to the masonry wallette through grade 8.8 Ø 14 mm threaded bars bonded to masonry and timber with epoxy resin. One connector was located at each corner of the CLT panel (minimum distance from the panel edges > 20 cm) plus one in the middle, for a total of 5 connectors per specimen (≈ 3.5 connectors/m^2). Care was taken to ensure that the load was applied only to the masonry without engaging directly the timber panel.

All specimens showed diagonal cracking failure, with the diagonal cracks parallel to the loading direction and the cracks opening almost always along the mortar joints (Figure 4b). The retrofitted wallettes exhibited a 14% increase in shear capacity with respect to the unreinforced configuration (Table 3).

Table 3: Diagonal compression tests. Peak capacity and equivalent shear strength (ASTM 2015, mean values)

Specimen	Number of tests	F_{max} (kN)	CoV (%)	s_s (MPa)	CoV (%)
As-built	4	209.55	10.8%	0.31	10.9%
Retrofitted	4	238.15	12.9%	0.35	13.6%

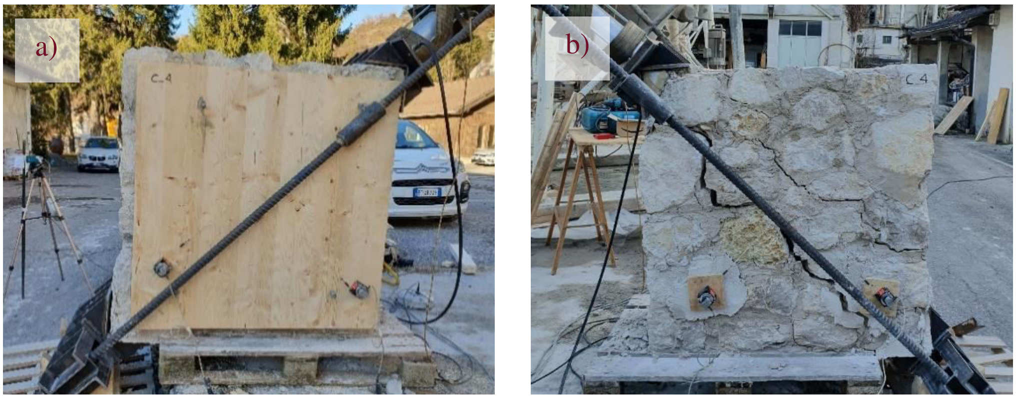

Figure 4: Testing of wallettes. Retrofitted specimen: a) reinforced side; b) unreinforced side

A scale effect related to the small number of connections (Rizzi et al., 2020) is thought to have lessened the impact of the retrofit. Furthermore, Rizzi et al. (2020) have compared the results from the wallettes retrofitted with the CLT panels with wallettes from the same construction batch but retrofitted with composite reinforced mortar (i.e., lime plaster reinforced with a glass fibre mesh) and observed that the CLT retrofit produces a more significant increase in the strain ductility.

4 ONSITE TESTING OF FULL-SCALE WALLS

The last phase of the experimental investigation saw three full-scale wall specimens of 1800 mm x 1800 mm subjected to cyclic lateral loading. The specimens were isolated from the internal walls of the clay-brick vertical addition constructed in the 1920s on top of the three-storey masonry hotel mentioned in the previous sections and currently scheduled for demolition.

4.1 Testing framework and experimental setup

The tested walls are three-leaf clay-brick walls with an average thickness of 340 mm (brick size ≈ 200 x 100 x 50 mm), separated from the surrounding masonry via through cuts. Brick and lime-mortar samples were extracted from the area closest to the wall specimens. Details on the obtained material properties are reported by Giongo et al. (2020). Preliminary numerical studies have shown that the key parameters in determining the performance under in-plane loading of masonry walls retrofitted with timber panels change based on the behaviour of the unreinforced walls. Slender walls mainly responding in rocking see the primary role of the panel anchoring system, while squat walls responding in shear see the predominant role of the timber-to-masonry distributed connection. Given the aspect ratio, the boundary conditions, and the mechanical properties of the masonry specimens, an additional vertical stress of ≈0.2 MPa to be applied on top of the specimens was deemed necessary to ensure activation of the shear failure. Such stress value can be considered representative of the stress level at the base of a three-storey building.

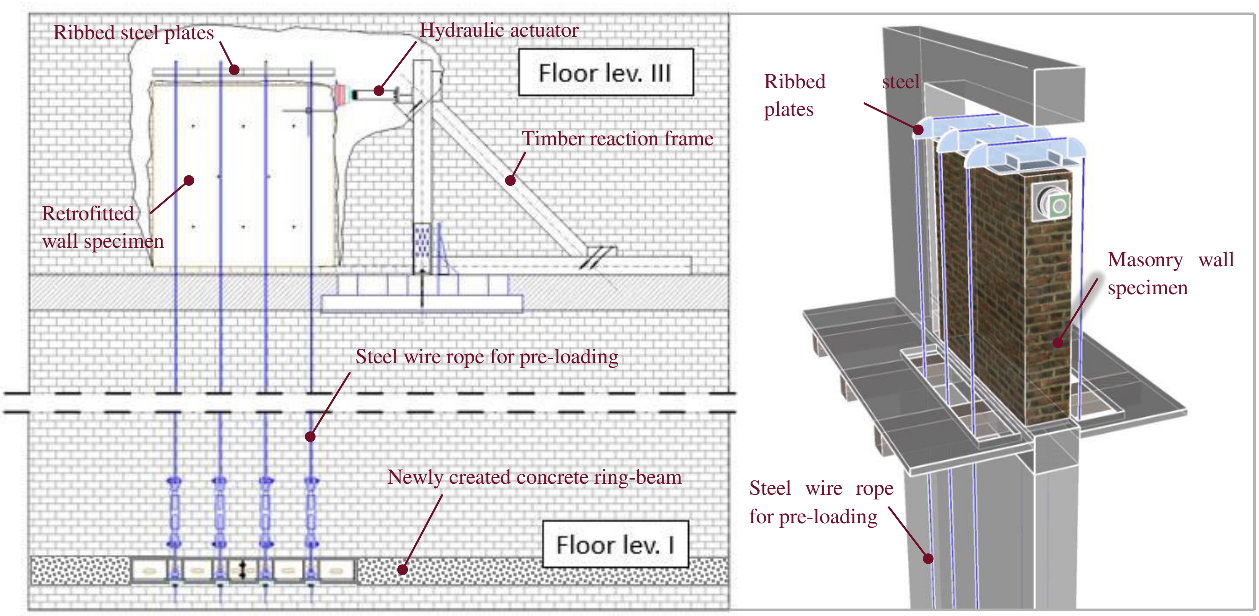

Figure 5: Schematics of the test setup

The vertical load was applied thanks to three steel wire ropes which circled the specimen's top surface (Figure 5b) and were anchored two storeys below the specimens to RC ring-beams explicitly constructed for this testing campaign on both sides of the tested masonry walls (Figure 5a). Turnbuckles at the rope extremities allowed a progressive tightening of the ropes with steps of ≈5 kN, up to a total load of ≈23 kN per wire rope. Ribbed steel plates with built-in rails to guide the wire ropes were positioned on top of each specimen (Figure6). The ribbed plates were supported on a layer of M5 strength-class lime mortar to ensure an even distribution of the vertical overburden. The semicyclic horizontal force simulating the earthquake action was applied with a hydraulic actuator positioned at ≈1600 mm from the specimen base. In the contact location between the actuator and the specimen, a 15 mm thick steel plate and a 20 mm thick layer of high-strength mortar were used to mitigate stress concentration and avoid local masonry crushing.

Figure 6: Details of the setup for the lateral testing of full-scale masonry shear walls

4.2 Specimens' details and test results

Specimens one and two (i.e. Sp.1 and Sp.2) were first tested in the as-built unreinforced condition (i.e. test AsB-1 and AsB-2) to create a benchmark for the retrofit performance evaluation. The testing was stopped soon after the shear failure mechanism was activated, and the formation of a stair-stepped diagonal crack was observed (Figure 7b). The specimens were then repaired with a 60 mm thick spruce CLT panel (Figure 7a) connected with 16 fasteners per specimen (≈5 fasteners/m^2). Partially threaded screw anchors coupled with washers were used for specimen one, whereas double-threaded screw anchors were adopted for specimen two (Figure 7c). The third and last specimen (Sp.3) was tested directly in the retrofitted configuration, with the reinforcing CLT panel connected to the masonry wall prior to any testing. A total of 25 double-threaded screw anchors were used for the timber-to-masonry connection (≈8 fasteners/m^2).

Figure 7: Full-scale tests: a) Reinforced wall with CLT panel ready to be tested [11], b) Stair-stepped failure observed after testing, c) Dry fasteners used for the CLT panel-to-masonry wall connection

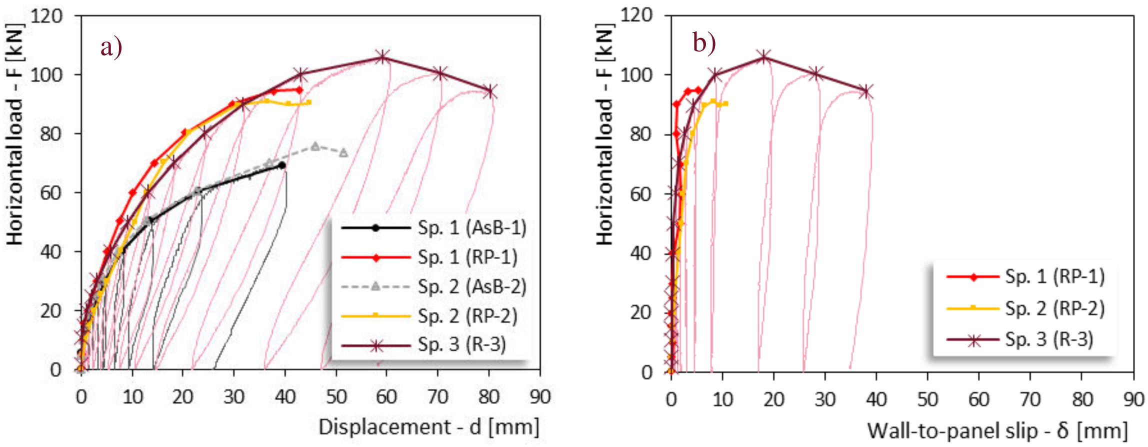

Figure 8: Full-scale tests. a) Comparison of load-displacement curves, b) Masonry-to-CLT slip measured at the top of the wall for the repaired and retrofitted tests

As visible from the graph in Figure 8a, all tests (as-built, repaired, and retrofitted) exhibited almost identical initial stiffnesses. Concurrently, repaired and retrofitted specimens showed a considerable increase in shear capacity (+20% and +40%, respectively). Such behaviour is particularly relevant to applying the retrofit technique in real scenarios, where selective strengthening of the most vulnerable walls can be pursued without altering existing force patterns. Figure 8b gives the slip values at the masonry-to-CLT interface, measured at the top of the specimen on the opposite side to where the load was applied. It can be observed that the peak capacity of the full-scale specimens (repaired and retrofitted) is associated with slip values in the range of 5 to 15 mm, consistent with the yielding of the fasteners derived from the connection shear testing (see Figure 1b). The post-peak softening of the load-displacement curve is governed by the sequential yielding of the fasteners located in less stressed areas of the panel. Finally, whilst the actual ultimate displacement capacity of the specimens could not be reached for obvious safety reasons, both repaired and retrofitted walls widely exceeded the drift limits by current standards, with Sp.3 reaching a remarkable drift value of 4.5%.

5 FINAL REMARKS

The paper reports a synthetic overview of the extensive testing activity carried out by the authors to provide experimental evidence to retrofit methods that use CLT-based claddings to improve the seismic behaviour of masonry buildings. The reported testing activity was focused on the in-plane response of the masonry walls and was mostly performed onsite. The testing addressed the entire retrofit system, from the distributed timber-to-masonry connections to the full-scale walls. The outcomes suggest that using CLT panels fixed to the masonry surface with spread dowel-type fasteners can considerably improve the in-plane response of the walls. The data presented here have already been used by the authors as the foundation for extensive numerical modelling and for developing design approaches that will be published soon.

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