Full-scale experimental validation of the use of plywood panels for the in-plane strengthening of URM walls

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ABSTRACT

The susceptibility of unreinforced masonry (URM) walls to collapse under seismic loading has been repeatedly observed and documented across a multitude of earthquakes worldwide. Various seismic retrofit techniques exist. However, there continues to be a developing need for experimentally validated, simple, and cost-effective solutions that also consider the impact on building tenants, aesthetics, and the heritage fabric of a structure. Because of this, timber-based retrofit strategies have been gaining popularity both in academia and industry as a method of achieving the developing performance, sustainability, and cost requirements of URM retrofit projects.

The retrofit technique studied herein is the use of plywood panels for in-plane strengthening of URM walls. With the use of mechanical fixings, this retrofit becomes a fully reversible, cost effective, and low impact solution.

The in-plane behavior of as-built, retrofitted, and repaired masonry walls was investigated by conducting full scale in-situ semi-static in-plane shear tests on as-built, damaged, repaired, and retrofitted masonry walls. The outcomes of this testing regime include quantification of improvement in seismic capacity, force vs displacement behaviour of test specimens, and observation of critical failure modes.

1 INTRODUCTION

Timber-based strengthening strategies are gaining increasing attention in both academia and industry as they align with key principles of sustainable retrofitting, such as material renewability, rapid installation, and reversibility. In the context of masonry buildings, various solutions exist to enhance both local and global seismic performance, each providing different levels of effectiveness and invasiveness.

Among these strategies, timber strong-backs have been proven highly effective in preventing the out-of-plane collapse of face-loaded walls while remaining cost-efficient and minimally disruptive to occupants (Dizhur et al., 2017; Cassol et al., 2021). Additionally, the integration of thin wood-based sheathing panels and blocking elements transforms strong-backs into light timber frames, significantly enhancing the in-plane performance of single-leaf masonry walls (Damiani et al., 2020; Guerrini et al., 2021).

For thicker masonry walls, overlays made of cross-laminated timber (CLT) or laminated veneer lumber (LVL) panels, attached via distributed dry fixings or bonded steel rods, have demonstrated effectiveness in improving both in-plane and out-of-plane seismic performance (Rizzi et al., 2020; Pozza et al., 2021; Cassol et al., 2024). Full-scale onsite testing (Giongo et al., 2021) and extensive numerical modelling (Giongo et al., 2017; Cassol et al., 2021, 2023) indicate that CLT overlays, when mechanically connected to masonry with a sufficient number of distributed point fixings (> 3 fixings/m²), can significantly enhance the lateral load capacity of masonry walls and buildings. More extensive retrofit approaches involving timber-based endo- and exo-skeletons have also been explored (Valluzzi et al., 2021; Zanni et al., 2021).

This study presents an optimized addition to the family of timber-based retrofits: a multi-layer plywood overlay mechanically fastened to the internal surface of masonry walls. The objective is to achieve seismic performance comparable to CLT overlays while enhancing adaptability and practical application, particularly due to the ease of transport and flexible sizing of plywood sheets. This paper reports on an experimental campaign designed to evaluate the effectiveness of this novel solution.

2 EXPERIMENTAL PROGRAM

The experimental program was structured in two stages. A first stage aimed at characterising the performance of the timber-to-masonry mechanical connections and assessing the optimal configuration maximising the shear capacity of the connection. The second stage consisted of testing under lateral force a masonry wall prior and after the application of plywood retrofit overlays.

2.1 Connection testing

Timber-to-masonry connection was provided using 230 mm long mechanical fixings with a shank diameter of 8 mm. The fixing features a 160 mm long front-thread for use in masonry and a 50 mm long rear-thread designed for timber or masonry (see Figure 1). The testing was performed as shown in Figure 1, with the timber-to-masonry connections subjected to shear loading to failure by applying an increasing horizontal tensile force to a timber block composed of 2, 3 or 4 layers of F8 plywood sheets. This block represents a portion of the timber retrofit overlay and was fixed to the masonry wall surface with a single mechanical fixing. The tensile force was applied through a steel rod attached to a hydraulic actuator. It was aligned with the mid-thickness plane of the timber retrofit and acted parallel to the wall surface.

Table 1: Timber-to-masonry connection test matrix

Plywood layup		**4 sheets**	**3 sheets**	**2 sheets**
Test ID	**Brick Type**	**n. test repetitions**	**n. test repetitions**	**n. test repetitions**
T1a	hard	3	4	3
T1b	medium	3	3	3
T2a	hard	-	3	3
T2b	medium	-	3	3

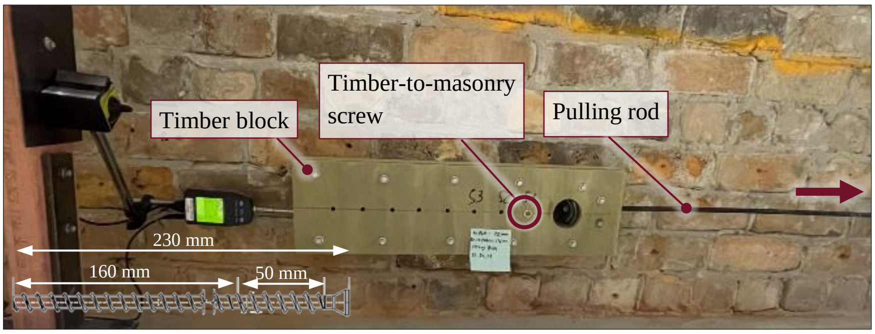


Figure 1: Timber-to-masonry connection test setup and fixing detail

Two main configurations, namely T1 and T2, were tested for every plywood layup (Table 1). The difference between the two is that in T2, the mechanical fixings were installed with a countersunk washer increasing the bearing surface underneath the fixing head and providing additional rotational restraint. According to Table C8.3 of MBIE (2017), the bricks selected for the testing fell either into the "Medium" or "Hard" hardness categories, to which correspond different probable compressive and tensile strengths. Therefore, the sub-configurations distinguished by the suffix "a" (hard brick) and "b" (medium brick) were introduced to keep track of such information.

2.2 Full-scale wall testing

The wall specimen was a three-leaf wall (1790 mm × 1440 mm × 380 mm, length × height × thickness) constructed using medium strength recycled vintage clay bricks and lime mortar. The wall material properties are given in Table 2. The bricks and lime mortar average compressive strength were obtained through testing performed according to test standards ASTM C67 and ASTM C109, respectively. Masonry compressive strength was then calculated according to NZSEE (2017). The timber strengthening was composed of three layers of 18 mm thick plywood sheets, totalling 54 mm in thickness. The plywood sheets were arranged so that their longer dimension (i.e., length) spanned the entire wall height. As most commercially available plywood sheets have a typical width of 1200-1250 mm, the sheets were staggered between the layers to form a continuous strengthening overlay along the entire masonry wall. The three layers were fastened together using 50 mm long (gauge 10) timber screws, spaced at 50 mm centres along the perimeter and 100 mm at internal junctions, as shown in Figure 2.

Table 2: Material properties

Material Description	Average Compressive Strength (MPa)	CoV	Test Standard
Bricks - medium strength (recycled vintage clay bricks)	13.7	0.4	ASTM C67
Lime mortar - medium strength	4.6	0.2	ASTM C109
Masonry	8.4	-	Calculated using NZSEE (2017)

The plywood strengthening overlay was attached to the masonry using 20 mechanical fixings with washers. The fixing density (7–8 per square meter of wall surface) aligns with previous research by Giongo et al. (2020), in which the same type of connectors were used to secure 60 mm thick cross-laminated timber strengthening panels to 340 mm thick masonry walls, resulting in up to a 40% increase in lateral capacity. Additionally, two hold down brackets were installed to counteract uplift forces, transferring them to the foundation and preventing rocking of the plywood strengthening overlay.

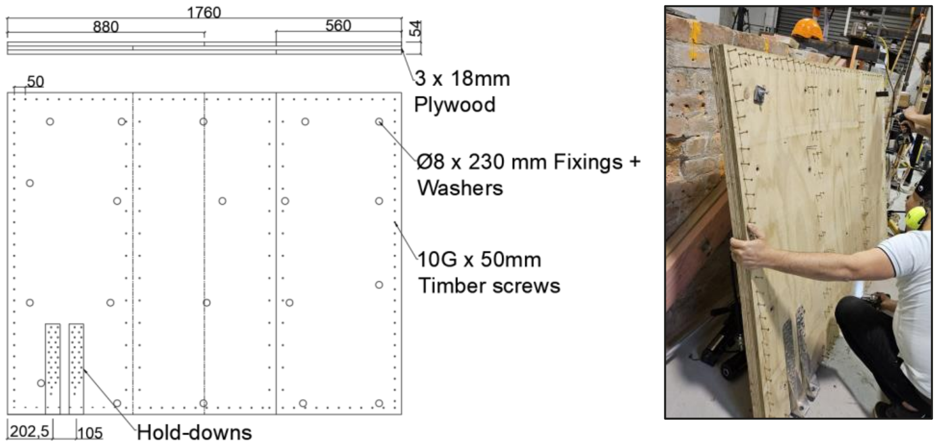


Figure 2 Plywood strengthening detailing

2.3 Test setup

The experiment consisted of applying an in-plane load to the top of the wall recorded by a digital acquisition system that also recorded in plane displacement, out-of-plane displacement of the plywood and diagonal displacement in both directions of the wall by means of displacement transducers and digital image correlation (see Figure 3). The horizontal load was applied by a 200 kN hydraulic actuator (see Figure 5a) pushing on a c-shaped steel spreader (180 mm × 75 mm PFC) located near the top of the wall front edge and supported on a 5 mm layer plaster to ensure a uniform stress distribution.

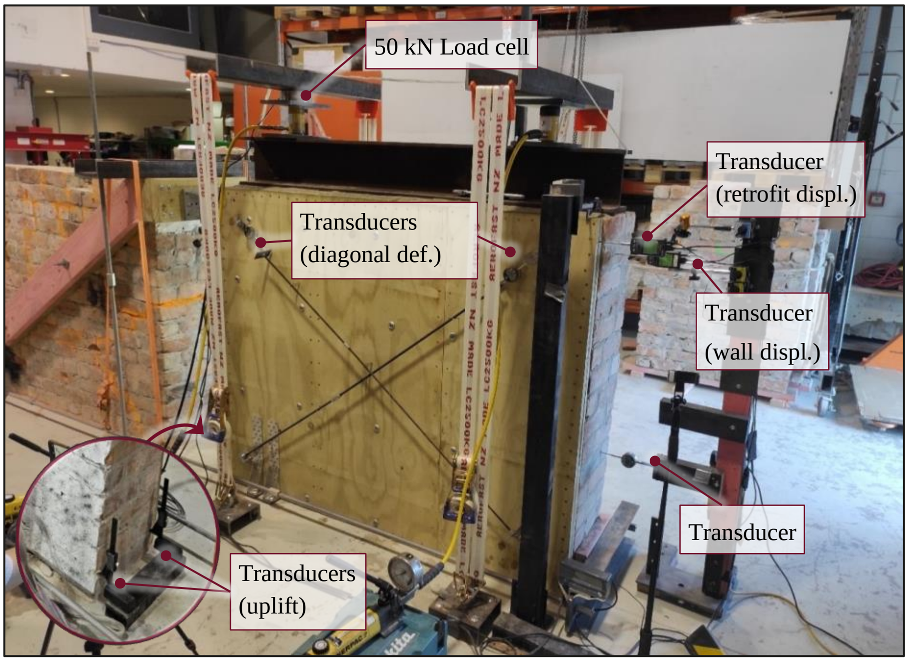


Figure 3: Instruments layout (retrofitted specimen)

The reaction system contrasting the actuator force was provided by a companion brick wall (2000 mm ×1440 mm × 380 mm, length × height × thickness) constructed near the specimen to be tested. Compressed diagonal timber struts on both wall sides and tension vertical steel rods closed the force triangle and helped the reaction-wall transfer the actuator force to the foundation of the laboratory (Figure 4).

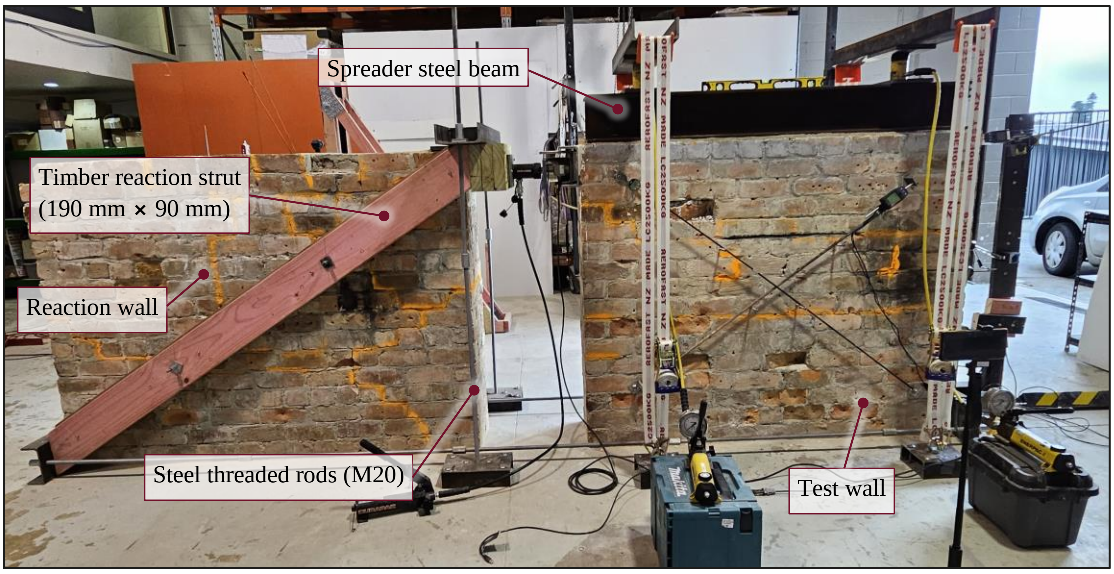
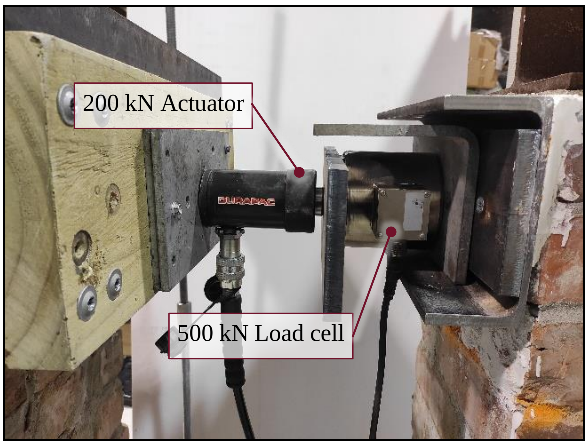
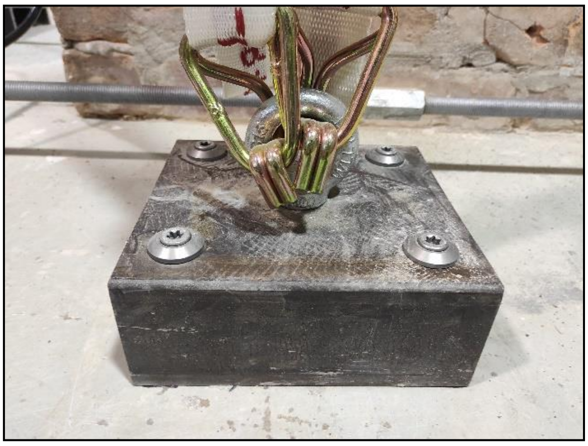


Figure 4: Test setup for the wall testing

The vertical loading applied on top of the wall simulated a two-storey building load of 0.15 MPa, which resulted in a total of 80 kN. To ensure an even distribution of the load, a spreader steel beam (250 mm UC section) was installed at the top of the wall as seen in Figure 4. The load was applied using two hydraulic actuators that pushed up tie-down straps connected to the floor with fixings (see Figure 5b).





(a) Horizontal load application

(b) Vertical loading anchor points to the floor

Figure 5: Close-up on setup details

3 TEST RESULTS AND DISCUSSION

3.1 Timber-to-masonry connections

All tested connections exhibited failure characterized by connection failure (under combined tension and bending), associated with local bearing failure of the timber and crushing of the bricks in the area surrounding the fixing shank at the timber-to-masonry interface (Figure 6). Unlike the consistent findings of Riccadonna et al. (2019), who performed similar tests on CLT/LVL-to-masonry fixing connections, no splitting of the bricks was observed. This is despite the probable tensile strength of the bricks in this study (ranging between 3.1 MPa and 4.2 MPa, MBIE, 2017) being aligned with that of the bricks tested by Riccadonna et al. (3.7 MPa). The reason might be the slightly smaller core diameter of the fixings (8 mm instead of 9.4 mm).

(a) timber bearing failure (b) local brick crushing (c) fixing head w/o washer (T1) (d) fixing with washer (T2)

Figure 6: Details at failure of the timber-to-masonry connections

Table 3 synthesizes the results of the timber-to-masonry connection tests in terms of maximum capacity. A more in-depth analysis of the connection behaviour will be provided in a future publication. Because no appreciable difference in the maximum capacity was observed between tests on “medium” or “hard bricks”, the results for the two sub-categories were merged.

Table 3: Timber-to-masonry connection performance

Plywood layup		**4 sheets**		**3 sheets**		**2 sheets**	
Test ID		**Max. Load [kN]**	**n. test repetitions**	**Max. Load [kN]**	**n. test repetitions**	**Max. Load [kN]**	**n. test repetitions**
T1	Avg [kN]	14,8	6	13,0	7	10,3	6
	CoV %	23%		14%		17%	
T2	Avg [kN]			24,3	6	19,5	6
	CoV %			22%		10%	

The results of test configuration T1 showed an almost perfectly linear correlation ($R^2 = 0.987$) between capacity and overlay thickness. Using the two-layer layup as a reference, a 26% increase in capacity was observed when an extra layer was added, and a 44% increase when two additional layers were added. The use of washers resulted in a maximum capacity increase of 89% for two-layered plywood overlays and 87% in case of three-layered overlays. The effect of the washers was also visible in the initial response of the connections, where an extremely stiff behaviour attributed to friction forces (of approximately 5 kN) was consistently observed (see Figure 7).

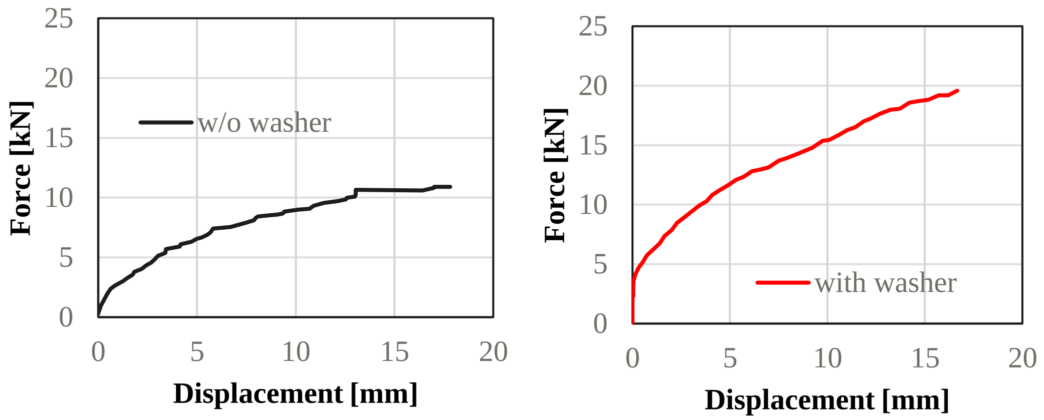


Figure 7: Representative force vs. displacement curves obtained from testing the mechanical fixings with and without washers (2 plywood sheets configuration)

3.2 Full-scale wall

Figure 8 presents the test results in terms of lateral applied force versus displacement measured at the top of the wall. As expected, in the as-built condition (black line), the wall's response was governed by rocking, which initiated at approximately 60 kN, closely matching the 57 kN predicted using MBIE (2017). The subsequent increase in capacity, reaching a peak of 100 kN, was due to the growing tension in the tie-down straps. Testing was halted when a lateral drift of 2.6% was reached, exceeding the 2.2% drift limit recommended by MBIE (2017) for three-leaf walls.

Inducing shear failure in the wall would require significantly higher vertical stress and complete rotational restraint at the top surface (e.g., simulating a strong spandrel–weak pier scenario). However, these conditions were incompatible with the available testing setup, which was constrained by the foundation capacity and the limited tension force that could be transferred to it.

The specimen was then retrofitted with the 54 mm thick plywood overlay. The retrofitted wall response (red line in Figure 8) did not show any sign of failure up the maximum force of 120 kN permitted by the testing setup that was reach at a displacement of ≈10.5 mm, corresponding to a drift of 0.74%. This resulted in a 37% capacity increase if compared to the performance of the as-built specimen at the same drift.

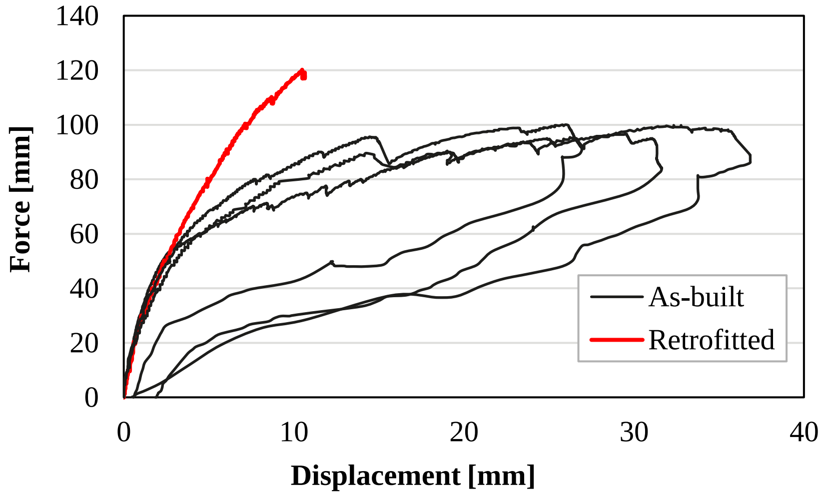


Figure 8 Test results: As-built vs. Retrofitted conditions

The plywood overlay was then removed and stair-stepped cracks were artificially created on both sides of the specimens (Figure 9) to simulate a wall damaged in shear by seismic loading. Figure 10 shows the pushover curve of the cracked wall (black line in Figure 10a), characterized by a maximum lateral capacity of 46 kN, consistently with the 40 kN of residual capacity predicted by MBIE (2017) for bed-joint sliding shear.

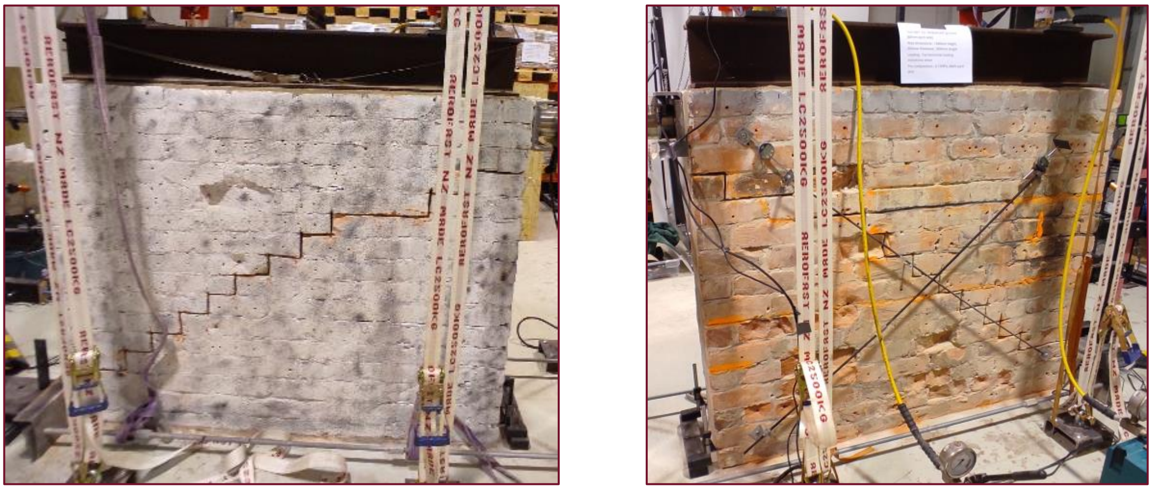


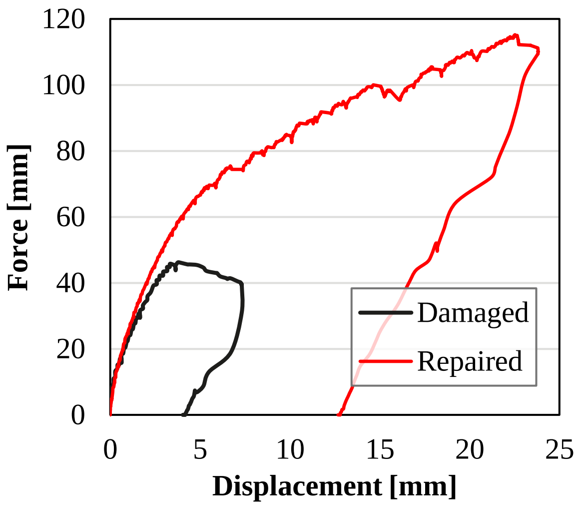
Figure 9: stair-stepped cracks

Once damaged, the specimen was repaired by applying the three-layer plywood overlay and then tested (red line in Figure 10a). At a displacement of 10.5 mm, the repaired specimen exhibited a lateral capacity of 88 kN, corresponding to 73% of that of the as-built specimen. The testing continued until the maximum force permitted by the setup was reached, with no signs of further damage to the wall or any damage to the retrofit. The capacity increased by 149% compared to that of the damaged specimen.

The stair-stepped crack separated the wall into two blocks. The nine fixings inserted in the top block were therefore responsible for transferring to the reinforcing timber overlay the approximately 75 kN of additional lateral force, which exceeded the cracked-wall residual capacity. This force level (i.e., ≈ 8.3 kN per fixing on average) was clearly compatible with the connection shear strength measured in the timber-to-masonry connection testing (expected average capacity of 24.3 kN). In fact, no fixing showed any signs of approaching ultimate conditions. To demonstrate that, based on the information obtained from the connection testing, this load could also have been successfully carried by "less performing" connections, the wall was tested again, this time with the fixings without washers (expected average capacity of 10.3 kN). As it can be seen from Figure 10b (continuous red line) no detrimental effects were observed in the overall performance of the repaired wall. Finally, the three-layer plywood overlay was replaced with a more efficient two-layer overlay (from the perspective of timber material use) fixed with the same number of fixings, now installed again with washer (expected average capacity of 19.5 kN). The repaired wall (dashed red line in Figure 10b) exhibited a similar response as before, with no damage to the retrofit system and only a minor reduction in stiffness.

The maximum angular shear deformation of the overlays, obtained from the diagonal variations, was 0.055%, confirming a "*solid panel*-like" response of the plywood overlay.

A final loading cycle was performed on the wall after removal of the repairing 2-layer overlay (black line in Figure 10b) to benchmark the residual capacity of the cracked wall after the testing with the various repair configurations against the previously obtained value. The residual capacity showed no decrease due to the repetition of testing cycles.



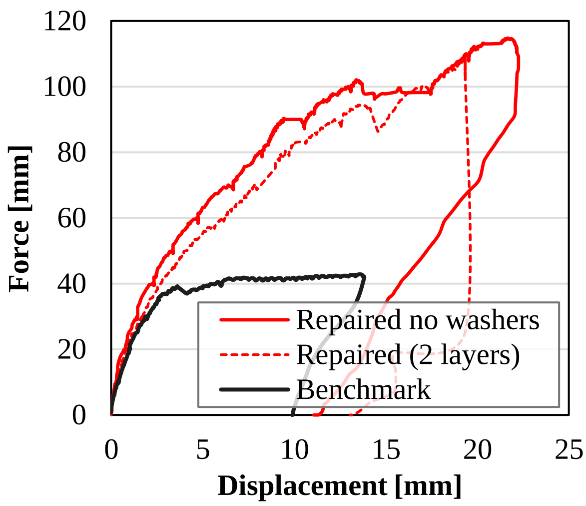


Figure 10 Test results: Damaged vs. Repaired conditions

4 FINAL REMARKS

This study experimentally assessed the effectiveness of plywood overlays for the seismic strengthening of unreinforced masonry (URM) walls, focusing on both timber-to-masonry connection behaviour and full-scale wall performance. Connection testing demonstrated that mechanical fixings with countersunk washers significantly enhanced the shear capacity of timber-to-masonry connections.

Full-scale testing further confirmed the effectiveness of the plywood retrofit, showing that at a drift of 0.74%, the strengthened wall exhibited a 37% increase in lateral strength compared to the as-built condition. However, due to test setup limitations, the maximum capacity of the strengthened wall could not be determined, as testing was halted before failure. To further evaluate the retrofit's effectiveness, the same wall was artificially damaged by introducing a stair-stepped crack to simulate seismic damage. When repaired with the plywood overlay, the lateral load capacity increased by 149% compared to the damaged condition, with no signs of further damage or retrofit failure. These findings suggest that plywood overlays, easily adaptable and reversible, can be a viable alternative to more invasive and less sustainable retrofits for improving both local and global seismic performance.

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