
Full-scale experimental validation of a new crack stitching system for URM

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ABSTRACT

Crack stitching is a well-established technique for repairing unreinforced masonry (URM) structures cracked by thermal expansion, differential settlement or earthquake. This study presents the experimental evaluation of a crack stitching system consisting of flexible steel stitch element bonded with specified epoxy adhesive. The system was designed to improve mechanical interlock and distribute stresses along the bond length, achieving high pull-out performance at reduced development lengths.

An experimental campaign was conducted at both component and full-scale levels. Component-level pull-out tests were conducted on masonry wallettes in accordance with BS EN 845-1:2013, and full-scale cyclic shear-compression tests were performed on double-leaf clay brick unreinforced masonry (URM) walls. Walls were tested in as-built, damaged, and repaired configurations, with the repair sequence consisting of the stitching system first applied in combination with timber strongbacks, and followed by removal of the strongbacks to evaluate the stitching system independently.

Results confirmed reliable load transfer from the stitching solution to the masonry. Full-scale tests showed that the combination of strongbacks and stitches improved considerably the damaged condition as well as the as-built strength. Stitch-only repairs also increased the damaged strength even after experiencing severe damage. Both repair solutions substantially improved ductility with respect to as-built conditions. Stiffness degradation for the stitch-only configuration followed a similar trend to the strongbacks and stitches system, indicating that the stitching solution was effective in withstand external loads without brittle failure even after severe damage. The proposed solution provides a practical, adaptable, and minimally invasive repair strategy for URM structures requiring crack repair.

1 INTRODUCTION

Cracks are the most common defect observed in masonry construction (Drobiec et al., 2008), often requiring repair for structural or aesthetic reasons (see Figure 1). Cracks can reduce the structural capacity of load bearing unreinforced masonry (URM) piers or spandrels and disrupt load paths in seismic resisting elements. Several common repair techniques are used for repairing or retrofitting URM, including reinforced concrete or plaster overlays, addition of external framing, core grouting or near surface mounted insertions (Binda et al., 2005; Earthquake Engineering Sectional Committee, 2009; Kouris & Triantafillou, 2018). Among these solutions, crack stitching solutions are widely adopted due to its minimal visual impact and ease of application. Crack stitching consists of inserting tension-resisting linear elements into mortar bed joints bonded with adhesives, aiming to restore continuity by stitching cracked URM portions together, limit slippage along mortar joints, and enhance in-plane shear behaviour by absorbing tensile stresses (Valluzzi et al., 2005). Experimental studies indicate that crack stitching is capable of restoring the original capacity of the URM and improving the ductility, while limited increments of strength from the as-built capacity have been observed (Valluzzi et al., 2005; Drobiec et al., 2008; Ismail et al., 2011; Demaj et al., 2022). Commonly used crack stitching techniques can be classified as: 1) reinforcing steel bars, 2) rigid high strength plain bars (Corradi et al., 2020; Corradi et al., 2022), 3) helical bars (Ismail et al., 2011; Licciardello et al., 2020), and 4) steel wire rope (Misnon et al., 2021). Despite its widespread use, only limited information and recommendations on the effectiveness of crack stitching is available, and interventions are often based on local practice, product availability, and engineering judgement (Drobiec et al., 2008). In this study, a crack stitching system is proposed, consisting of flexible steel stitch element (provided by PYTHON Fixings®) designed to enhance mechanical interlock with the adhesive, avoiding slipping of the wire, and installed within masonry bed joints using epoxy resin. This paper presents an experimental campaign designed to evaluate the effectiveness of the proposed solution, with three URM wallettes tested to assess the system at a component level and a full-scale wall tested to evaluate the effectiveness of the complete system in realistic service conditions.



(a) Thermal expansion cracking



(b) Earthquake cracking



(b) Differential settlement cracking

Figure 1: Examples of cracked masonry

2 EXPERIMENTAL PROGRAM

2.1.1 Component testing

The presented research focuses on evaluating the structural effectiveness of an innovative masonry crack repair technique. A first stage aimed at characterising the performance of the stitch at a component level and

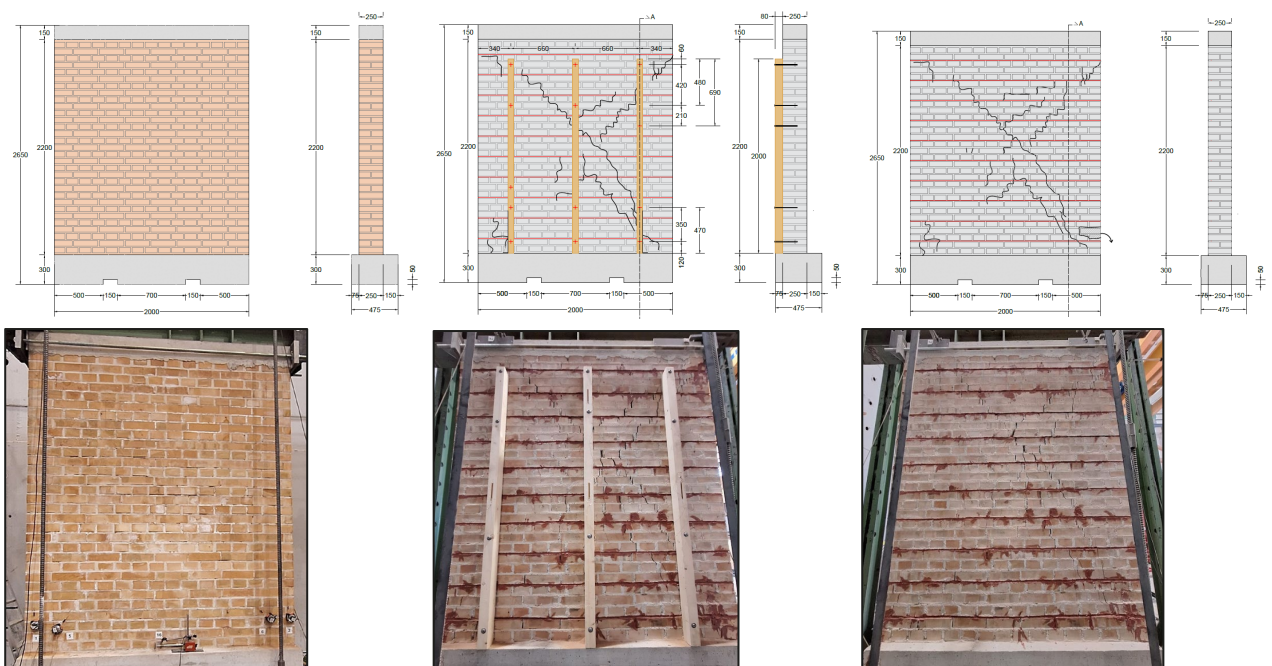
obtain the tension capacity and failure modes. For this purpose, three URM wallettes were constructed using heritage clay bricks and lime mortar. The wallettes were single-brick length built with two brick courses separated by a 15 mm mortar joint to accommodate the stitch with 230 mm development length (see Figure 2). After 28 days of mortar curing, a 30 mm deep groove was cut into the joint and cleaned to ensure effective bonding between the epoxy and the brick. A layer of epoxy was applied, followed by the installation of the stitch and finished with a second layer of epoxy (see Figure 2).



Figure 2: Wallette specimen

2.1.2 Full-scale testing

The second stage consisted of full-scale in-plane lateral testing of a pre-compressed two-leaf clay brick URM wall with 15 mm thick lime mortar joints (2000 mm × 2200 mm × 250 mm, length × height × thickness) prior and after the application of stitches and strongbacks. After 28 days of curing, the first specimen (1_URM, see Figure 3a) was tested to failure under cyclic shear-compression loading in as-built conditions. A second wall, identical to URM_1 for geometry and material properties, was tested similarly to 1_URM. After reaching failure, a single reference cycle was performed (1_URMt), aiming to reach the largest possible displacements in both directions without inducing collapse. This cycle represented the damaged behaviour of the wall in the pre-repair condition and served as the benchmark residual strength of the damaged masonry to compare against the repaired specimens. Subsequently, the damaged specimen was repaired with timber strongbacks and stitches installed every third course across the entire width of the specimen and tested to failure (2_RpSB&S, see Figure 3b).



(a) Benchmark URM pier (1_URM)

(b) Repaired pier with stitches and strongbacks (2_RpSB&S)

(b) Repaired pier with stitches (3_RpS)

Figure 3: Tested configurations

In the specimen 2RpSB&S, the stitches were inserted into the mortar bed joints with grooves of approximately 50 mm deep every third brick layer. Epoxy resin was then cast into the back end of the grooves, and the stitches were installed along the entire length of the wall and left to harden for over 12 hours. Then, the strongbacks were mechanically connected to the masonry using timber-to-masonry anchors, forming a composite system. The strongbacks were installed according to the configuration shown in Figure 3b. The intervention was completed with the installation of two hold-down anchoring devices at the base of the outermost strongbacks, designed to limit uplift and rocking phenomena during the subsequent loading phases. Finally, the strongbacks were removed leaving only the stitches (3_RpS, see Figure 3c) and tested for two cycles up to the point where safety was a concern.

2.2 Material properties

The three wallettes were constructed using heritage bricks with average dimensions of 230 mm (Length) x 110 mm (Width) x 75 mm (Height) with average compressive strength 17.8 MPa. The full-scale walls were constructed using new clay bricks with dimensions 250 mm (Length) x 115 mm (Width) x 60 mm (Height) with average compressive strength 23.9 MPa. The lime mortar used for the component testing was mixed with a 1:2:9 ratio of cement:lime:sand by volume with compressive strength 4.6 MPa. For the full-scale testing, natural hydraulic lime mortar was used, with an average compressive strength of 2.5 MPa. The crack stitching system was provided by PYTHON Fixings® and installed with epoxy resin with a compressive strength of 82 MPa and a tensile strength of 70 MPa.

The strongbacks installed in sample 2RpSB&S were laminated glued timber with strength class GL24h with a cross section of 80×60 mm and oriented with the long edge perpendicular to the plane of the wall. To reproduce in-service the boundary conditions and ensure an efficient transfer of vertical and horizontal loads, reinforced concrete beams were placed at the top and bottom of each wall. The beams were made of C25/30 reinforced concrete. The lower beam measured 475 mm in width and 300 mm in height, and the upper beam measured 250 mm in width and 150 mm in height (see Figure 2).

The wallette specimens were also used to evaluate the long-term resistance of the stitch under weathering exposure, subjecting the specimens to repeated thermal and moisture variations prior to tensile testing according to the principles of BS EN 13687-3 (2002).

2.3 Test setup

2.3.1 Component testing

The component testing was performed using three wallettes where the stitching system was installed and tested in tension following the principles of BS EN 845-1 (2013). A uniform vertical pre-compression load of 0.1 MPa was applied along the specimen length to replicate the weight of the upper storey in a two-storey typical URM buildings. The test configuration (see Figure 4) incorporated a reaction frame designed to prevent specimen rotation during loading as the stitch is installed eccentrically, and to avoid confinement effects around the wire rope. Pull-out tests were performed by applying an increasing load parallel to the mortar joint simulating the forces imposed on the stitch during crack opening. One load cell paired with a hydraulic jack was used to apply and measure the pulling force on the stitch, while a second load cell with another hydraulic jack measured the pre-compression applied along the specimen (see Figure 4). Displacement of the stitch relative to the wallette was recorded using two displacement transducers at each side of the stitch, and a wire clamp was used to lock the stitch and react against.

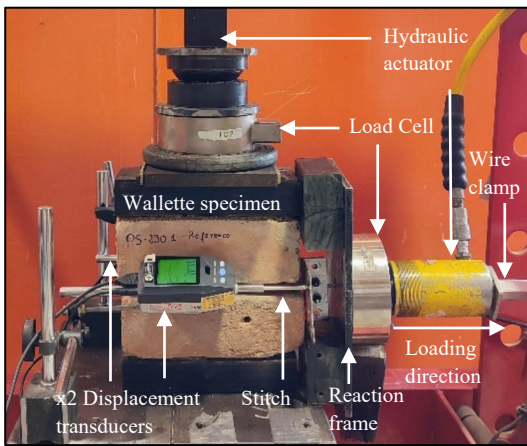


Figure 4: Tension test setup

2.3.2 Full-scale testing

The experimental setup was designed to replicate the boundary conditions typically observed in a three- to four-story URM building, which was simulated by applying a constant vertical pre-compressive load over the pier of 0.4 MPa. The specimen was supported at its base by a steel reaction system consisting of four double-T steel beams anchored to the laboratory reaction floor via post-tensioned rods. The reinforced concrete base beam of the wall was connected to the steel framework using custom-made steel angles (see Figure 5). Additionally, the base beam was post-tensioned with four 20 mm diameter threaded bars to improve anchorage efficiency, provide confinement, and increase the overall stiffness of the base region. A system of steel beams installed at the top of the wall ensured load transfer to the specimen. Two HE200 steel beams, each equipped with welded stiffening ribs, were placed on the top reinforced concrete beam to achieve uniform load distribution. The lower beam was connected directly to the hydraulic actuator and the specimen via angular brackets anchored with two post-tensioned, 20 mm diameter bars. To prevent out of plane movement of the wall during testing, the upper beam was held in place by two vertical steel columns. A roller-guided interface between the two beams enabled simultaneous, uniform transmission of vertical and horizontal forces to the specimen while also allowing for relative movement. The vertical loading system consisted of two hydraulic jacks that applied a constant compressive force through a steel beam connected to post-tensioned vertical bars. A hydraulic actuator connected to the specimen via a specially designed joint system applied horizontal loading. Figure 5 shows an overall view of the shear-compression test configuration.

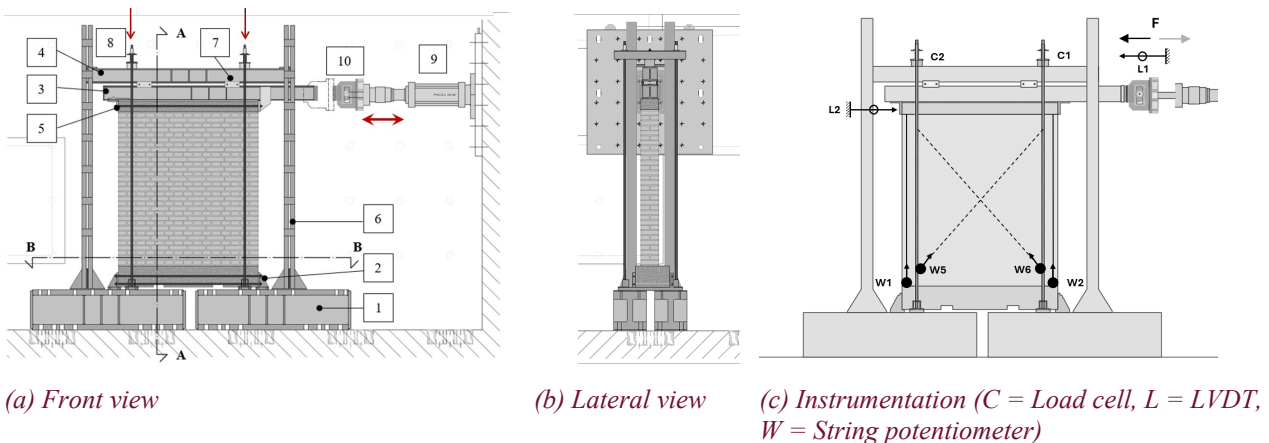


Figure 5: Full-scale test setup. 1- 4 double-T steel beams, 2 - custom-made steel angle plates + 4 threaded bars; 3 - lower HEB 200 beam; 4 - upper HEB 200 beam; 5 - angle brackets + 2 threaded bars; 6 - steel columns; 7 - roller-guided system; 8 - hydraulic jack + steel beam + vertical bar; 9 - hydraulic actuator; 10 - steel joint

The overall instrumentation layout is shown in Figure 5c. Horizontal (F) and vertical forces were measured using load cells (C1 and C2), while the total in-plane top displacements were recorded by linear variable displacement transducers (LVDTs L1 and L2). Vertical displacements along both sides of the wall were monitored using wire transducers (W1–W4). Other two wire transducers (W5 and W6) were located along the diagonal of the specimen on the unreinforced side to assess diagonal deformation. The out-of-plane deformations were controlled by two LVDTs positioned perpendicular to the face of the wall. Local stresses in the timber elements of the specimen repaired with strongbacks were measured using strain gauges.

A cyclic horizontal loading was applied under displacement control to simulate the seismic force. The loading protocol consisted of displacement cycles with progressively increasing amplitudes calculated by drift percentage in both directions (push and pull) until the ultimate capacity of the specimen was reached. The drift increments were of 0.1% up to 0.6% and of 0.25% increments up to 2%. From 2%, the drift increments were of 0.5%. Each drift level was repeated once except for the as-built specimen 1_URM that were repeated 3 times from drift level 0.5%.

3 TEST RESULTS AND DISCUSSION

3.1 Component testing

The test results and failure modes observed in the three wallette component specimens are reported in Table 1 and Figure 6. The stitching system installed in the wallettes was capable to withstand an average force of 12.78 kN at an average displacement of 1.70 mm with a development length of 230 mm. The governing critical failure modes observed in all the three specimens during the testing was a combination of mortar joint shear failure and partial cone failure (see Figure 6a,b).

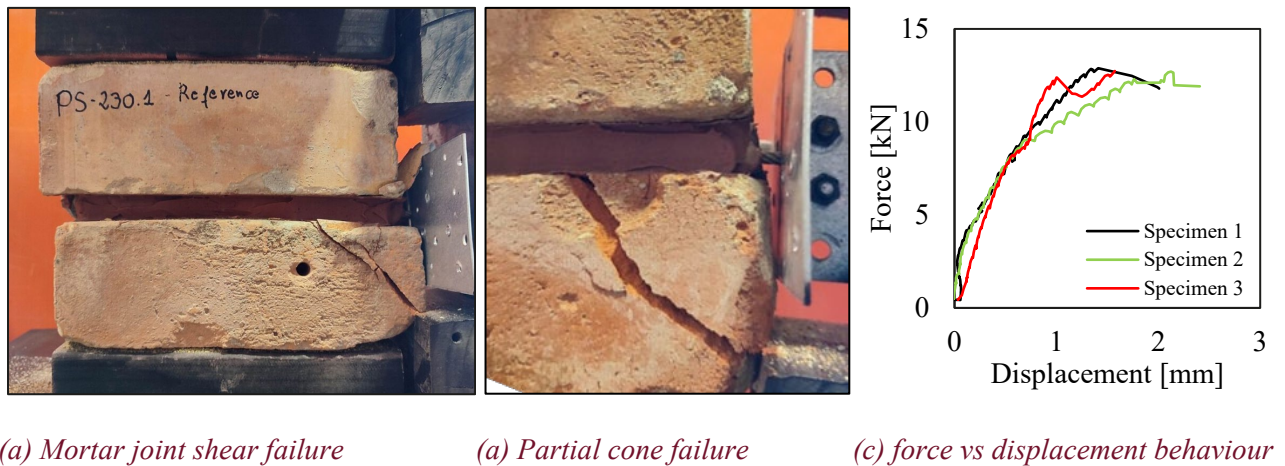


Figure 6: Failure modes and force vs displacement plots

Table 1: Pull-out test results from component testing wallettes

	$F_{\max}$ [kN]	Average $F_{\max}$ [kN] (CoV)	$d_{\max}$ [mm]	Average $d_{\max}$ [mm] (CoV)	Failure mode
Specimen 1	12.89		1.41		Mortar joint shear + partial cone failure
Specimen 2	12.71	12.78 (1%)	2.12	1.70 (22%)	
Specimen 3	12.74		1.57		

3.2 Full-scale testing

The results obtained from the full-scale tests, including maximum force, maximum displacement and failure modes are presented in Table 2 and Figure 7. At the end of the cyclic testing of the 1_URM and 1_URMt specimens, a clear x-shaped crack pattern was observed as visible in Figure 7a,b. The 1_URM test was interrupted when a capacity loss of 20% was reached, whereas specimen 1_URMt was tested beyond a “Severe Damage” limit state (see Figure 7d). After specimen 1_URMt was tested, the extensive cracking at the toe was repaired with epoxy mortar to prevent premature crushing failure during the 2_RpSB&S test. Specimen 2_RpSB&S was then tested up to failure, defined by a reduction in capacity of a cycle with respect to the previous cycle. Extensive diagonal cracking and significant damage were observed, particularly at the base of the wall (see Figure 7c). To evaluate the engagement and the force absorbed by the strongbacks during the 2_RpSB&S test, each strongback was instrumented with strain gauges that allowed to calculate the axial force based on the measured axial strains, the cross-sectional area of the strongbacks, and assuming an elastic modulus for timber equal to 11,500 MPa. The forces registered in the strongbacks show an active engagement throughout the test with the maximum force being 60 kN near the actuator at mid-height of the strongback, suggesting a strut and tie behaviour that resulted in the crushing of the repaired corners of the wall.

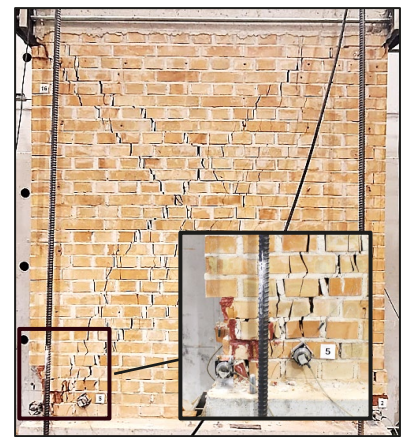
The stitches resulted in reduced damage to the face of the wall, allowing the wall to regain the capacity as much as the as-built 1_URM test with a reduction in stiffness. From the benchmark capacity (1_URMt) of 63.39 kN, a 258% capacity was improved in the 2_RpSB&S, and the capacity from the as-built model 1_URM was restored with a difference of 1.7%, achieved at a displacement of 2.1 times larger due to a reduction of stiffness (see Figure 7b).



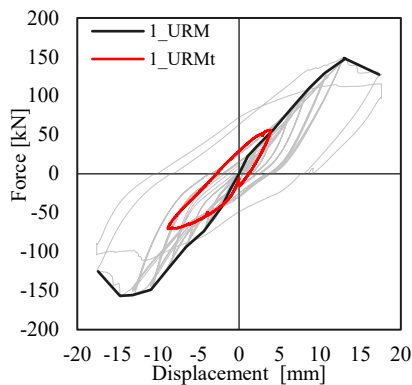
(a) 1_URM



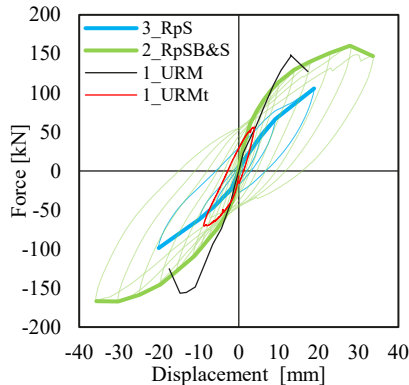
(b) 1_URMt



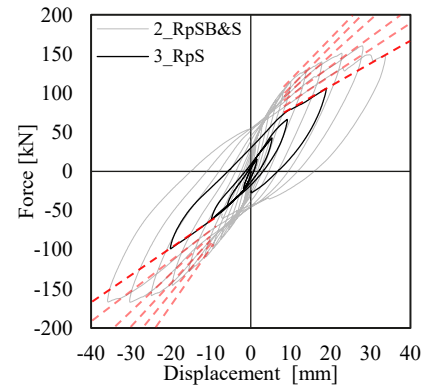
(c) 2_RpSB&S



(d) As-built vs damaged URM walls



(e) Comparison of all tests



(f) Stiffness degradation of repaired walls

Figure 7: Failure modes and force vs displacement capacity curves (+ values equal specimen being pushed)

Table 2: Summary of full-scale results (+ values equal specimen being pushed)

		1_URM	1_URMt	2_RpSB&S	3_RpS
$F_{\max}$ [kN]	+	148.53	56.18	160.33	105.58
	-	156.63	70.60	167.26	98.79
	Mean	152.58	63.39	163.79	102.19
$d_{\max}$ [mm]	+	12.95	3.89	27.89	18.85
	-	14.71	8.77	30.22	20.09
	Mean	13.83	6.33	29.06	19.47

Finally, the strongbacks were removed for testing of the specimen 3_RpS leaving only the stitches. Horizontal loading was applied up until safety measures permitted. The crack length continued to increase, up to the conclusion of the testing following the initial pattern, while the capacity of the wall reached 102.19 kN, which is an increment of 161% from the benchmark (see Table 2). The reduction in cyclic stiffness between the specimen with strongbacks (2_RpSB&S) and the specimen with stitches only (3_RpS) followed a similar degradation trend (see Figure 7f), indicating the effectiveness of the stitches after severe damage repair.

4 CONCLUDING REMARKS

This study experimentally evaluated the structural effectiveness of a newly developed crack stitching system for crack repair of unreinforced masonry (URM) walls, including its possible combined use with timber strongbacks. Tensile testing of the stitching system installed in masonry wallettes showed an average tensile capacity of 12.78 kN at an average displacement of 1.70 mm with 230 mm of development length.

The as-built full-scale URM specimen exhibited an average peak shear force of 153 kN with a mean maximum displacement of 13.8 mm. The benchmark specimen exhibited a residual mean strength of 63 kN, despite the severe damage it sustained during the previous test. The repaired specimen testing demonstrated that the combination of strongbacks and stitches significantly enhanced wall performance, providing a 7% increase in peak strength relative to the as-built condition and a 158% increase compared to the damaged specimen, together with a substantial improvement in ductility. This system proved capable of actively engaging the timber elements contributing to force redistribution and stitching effect between the damaged portions of the wall, as confirmed by the measured axial forces in the strongbacks and the capacity recovery.

When applied as a standalone repair, the wall repaired with only the stitching system reached 102 kN capacity, experiencing an increment of approximately 60% of the benchmark strength, even after severe damage from previous testing. The minimal stiffness degradation difference between the specimen with strongbacks and with only stitches showed the effectiveness of the stitches. Overall, the results of the experimental campaign indicate that the proposed crack repair methods represent an effective solution for the rehabilitation and seismic performance improvement of URM walls, restoring the strength of damaged walls to levels equal to or exceeding their original undamaged condition.

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