Combined Reinforcement of Rubble Stone Walls with CLT Panels and Steel Cords

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Abstract

Historic constructions are common in Europe, but natural hazards can produce significant damage. Shear reinforcement of historic walls, especially in seismic prone areas, is often necessary and new retrofitting methods have been recently proposed to restore or increase the lateral capacity of shear walls. In this paper the combined use of Cross-Laminated Timber panels (CLT) and steel cords is proposed to reinforce rubble stone masonry walls with the aim of increasing their lateral load capacity while improving the energy performance of the building envelope. An experimental campaign was carried out in the laboratory to assess the mechanical effectiveness of this retrofitting method. The results from a series of quasi-static cyclic shear tests are presented. Test programs are described and the analysis of test results is included. The potential benefits and limitations regarding the use of the proposed combined method for reinforcing masonry structures are discussed with an emphasis on the in-plane behaviour.

Keywords Seismic engineering, Brickwork & masonry, Timber structures.

List of notations

τ is the shear stress

τ_0 is the masonry shear strength

P	is the diagonal compressive load
P_{max}	is the maximum diagonal compressive load
A	is the area of the horizontal cross section of the wall panel
f_t	is the masonry tensile strength
Δl_{cA}	is the shortening of the compressed diagonal on panel’s side A
Δl_{tA}	is the elongation of the diagonal in tension on panel’s side A
l_{cA}	is the gage length of the compressed diagonal on panel’s side A
l_{cA}	is the gage length of the diagonal in tension on panel’s side A
ε_c	is the diagonal compressive strain
ε_t	is the diagonal tension strain
γ	is the angular strain
G_1	is the tangential elastic modulus of the masonry, given by the slope value of the secant line to the shear stress-angular strain curve between 10 and 40 % of the maximum diagonal load P_{max} and corresponding strain values.
G_2	is the tangential elastic modulus of the masonry, given by the slope value of the secant line to the shear stress-angular strain curve between 15 and 50 kN and corresponding strain values.

Introduction

There is growing evidence that natural materials and sustainable solutions are beginning to be taken seriously as viable materials to retrofit and repair old masonry structures (Righetti et al. (2016), Papayianni and Pachta (2017)). Old masonry structures (public and private buildings, schools, hospitals, etc.) and infrastructures (bridges, lighthouses, town walls, etc.) generally fulfil primary functions, and their reinforcement is a priority not only in seismic prone areas (Coburn and Spence (1992), Binda and Saisi (2005), Cardoso et al. (2005), Rota et al. (2014)).

Building two skins (wall leaves) of rubble stone masonry (double-leaf walls) has been the most popular method for constructing shear walls in many parts of Europe for centuries. However, these walls are often in need for repair or reinforcement and their response is particularly unsatisfactory when struck by an earthquake (Karantoni and Bouckovalas (1997), Bayraktar et al. (2007), D’Ayala and Paganoni (2011), Fiorentino et al. (2017)).

Several retrofitting methods have been proposed and used in the past. Reinforced Concrete (RC) jacketing is an established method of increasing the shear capacity of historic wall panels.

It consists in the application of a steel wire welded mesh (typically 150x150 mm), embedded into a concrete jacketing (typically, 40-60 mm thick). Although this method can be very effective in increasing the lateral capacity of shear walls, it is irreversible and the use of RC is not compatible with old masonry (Ashraf et al. (2012), Ghiassi and Soltani (2012)). As with all interventions, this will need to be weighed against the advantages of improving the structural stability of the building (Venice Charter (1964)).

Cracked brickwork and stone masonry can be also repaired or reinforced by injection of low cementitious or lime grouts. When used for repairing, cracks are sealed using micro fine grouts, made of low viscous and thixotropic materials for narrow cracks (Binda et al. (1994), Vintzileou and Miltiadou-Fezans (2008), Corradi et al. (2008), Isfeld et al. (2016)). This method can be also used to reinforce un-cracked stone masonry when there is a sufficient volume of internal voids. In this latter, more cohesive pastes or mortars are typically used. This method consists in pressure grouting of small volumes of cementitious or lime grouts into masonry structures to enhance structural integrity. Pressure grouting is able to strengthen cavity or rubble wall construction, stabilise loose fill and prevent movement. However, not all stone masonry structures can be injected: when the volume of voids is insufficient, this method can be ineffective. Such work is often undertaken in conjunction with other reinforcement or repair interventions, including RC jacketing, installation of stitch ties or anchors, or brick and masonry replacement.

Another traditional method of reinforcing wall panels is repointing. Repointing is the task of renewing the outer portion of the mortar joint, with new mortar (Alcaino and Santa-Maria (2008), Corradi et al. (2008)). A new development is to reinforce the new mortar used for repointing with steel elements (strips, cords, rods). In past investigations, Borri et al. (2014) and Castori et al. (2016) explored the effect of steel cord reinforcement on the structural response of in-plane loaded wall panels (Reticulatus method). They established that steel cords increased both the deformation capacity and shear strength of the reinforced panels. Repointing of stonework can be used to give structural support and cohesion to individual stone elements, either on its own or in conjunction with grouting. Repointing stone work provides a primary defence against water ingress, as well as having an important structural role for the stability of the building — it is also a critical aspect of the building’s maintenance schedule.

With regard to innovative retrofitting methods, recent investigation on shear reinforcement of masonry elements concentrated on the study of the behaviour of composite materials (grids) embedded into mortar coatings (FRCM: Fiber Reinforced Cementitious Mortars) subjected to a static, static-cyclic and dynamic loading (Ascione et al. (2015), Carozzi and Poggi (2015), Lignola et al. (2017)). Sciolti et al. (2018) performed static-cyclic shear tests on masonry panels, made with limestone and poor hydraulic mortar, until failure. The panels were tested in unreinforced configuration, and different FRCM reinforcement systems.

Another aspect should be also considered: one of the most critical flaws of traditional solid wall construction is its low energy efficiency. Several non-structural methods have been recently developed in response to improving technology and the implementation of new building regulations related to energy efficiency. Until the 1970s there was no maximum value for the thermal transmittance (U) of the layers that make up a building envelope. From the 1980s the U value was gradually reduced, requiring the introduction of insulation. With the release of the new Building Regulations in many parts of Europe (European Directive 2002/91/CE and 2010/31/UE; 2014 UK Building Regulations, Italian Act DM 26/6/2015, etc.) we face a situation where all the elements in the fabric envelope – the roof, walls and floor – need to have a very low U-value. In this situation the insulation of old solid walls is a challenging task. Rigid Polyurethane (PUR), polyisocyanurate (PIR) or phenolic foam are typically used for this purpose. Expanded (EPS) or extruded (XPS) are also employed to reduce the thermal transmittance of solid walls. All these materials are often under the form boards, fixed, glued or mechanically attached to the solid walls, without any structural function. Recently, wood fibre boards have been used for this purpose: wood fibre boards are also rigid and have a thermal conductivity similar to EPS.

Beyond the research efforts described above, the authors of this paper have proposed the use of composite grids embedded into a thermal-insulating mortar coating (Borri et al., 2015). This work experimentally studied the shear response of individual wall panels to help develop high walls’ shear capacity and reduced thermal transmittance.

With regard to the use of CLT panels, Lucchini et al. (2014) proposed the refurbishment of masonry buildings by means the installation of a CLT structure connected to the masonry, without altering the geometry of the masonry construction.

One additional study involving the application of a CLT structure connected to the masonry walls looked at issues of seismic resistance (Pozza et al., 2017). This work showed the need to properly prepare the connection of the CLT structure to the horizontal diaphragms (floor) of the masonry building. For interventions on heritage masonry buildings, the reversibility and the compatibility of new materials and techniques with historic ones is often required (Polastri et al. 2017). In this case, the reversibility of the proposed retrofitting intervention was guaranteed by the use of steel screws as a method of connection of the CLT structure to the masonry substrate.

Another important aspect has to be considered for its architectural importance: the preservation of the fair-faced aspect of the masonry. In many situations, brickwork or stone masonry walls were constructed in the past without lime plasters (fair face masonry). It is sometimes difficult to increase the load capacity of a wall without a surface reinforcement (i.e. RC jacketing, FRP sheets, etc.).

The basic idea of this experimental work is to investigate the effect of a combined retrofitting method for shear walls. CLT panels and steel cords have been used as NSM (Near Surface Mounted) reinforcement. The aim is to develop a method able to reduce several critical problems of historic walls: their low shear strength, high thermal transmittance and to preserve the masonry fair face aspect.

2. The proposed reinforcement method

The two wall leaves of a stone wall panel were reinforced using two different methods. With the aim of preserving the fair-faced aspect, one wall leaf was reinforced using the Reticulatus method. This method was proposed by Corradi et al. in 2016 and consists of embedding steel cords in the mortar joints (both vertical and horizontal ones). By doing this, it is possible to create a nearly square or rectangular mesh, the nodes of which are connected to the reinforcement on the other face of wall panel by means of transverse steel bars, according to the schemes in Figures 1 and 2a. The number of transversal connections depends on many factors (type of masonry, its mechanical properties, wall thickness, etc.): an established number is 5-6 per m^2. Furthermore, the cords are connected to the transverse steel bars using standard steel eyelets in which the cords can slide: thus, it is possible to apply a moderate tension to the cords, so as to make them immediately functional (Figure 2b).

The ends of the steel cords were passed into holes and fixed on the other wall face. To prevent stress concentration and local failures, rounded steel angles were placed near the holes between the cords and the masonry (Figures 3c and 3d) and finally the steel cords were fixed to eyelets, anchored to the stones, using locking devices (Figure 3d).
The grid and the eyelets are covered with a final layer of new mortar, making it possible to preserve the fair-faced aspect of the masonry (Figure 3e).

The other panel's wall leaf was reinforced using a 60 mm-thick CLT panel. This had identical dimensions as the stone masonry panel (1200x1200 mm) and it was made of three 20 mm-thick layers of solid Douglas wood (*Pseudotsuga Menziezii*). According to the producer data sheet, the shear modulus of this type of wood is 810 MPa. This value can be considered compatible with the shear modulus of the masonry. New mortar was used to level the panel surface before the application of the CLT panel (Figure 4a). Furthermore, to protect the CLT (Figures 4b and 4c) from water ingress, a membrane made of a polyethylene film was interposed between the stone masonry and CLT panel. The polyethylene films can be considered as vapour barriers and as such detrimental to the CLT material durability due to moisture stagnation at the CLT-mortar interface surface. However, the water ingress has to be prevented. More analysis is necessary in order to assess the best method to prevent wood degradation at interface with masonry.

The transverse threaded round bars with 8 mm diameter were made of stainless steel. A steel eyelet was welded to the bar's end (Figure 2b). 12 mm-diameter holes were drilled into the wall panels for bar installation. The other bar-end was fixed to the CLT panel using a 50 mm-diameter washer and a nut (Figure 3d). By tightening the nut, it was also possible to apply a moderate tension to the steel cords.

2. Specimens Construction, Instrumentation and Test layout

2.1 The wall panels

The proposed combined method has been applied as a repair technique of cracked wall panels. A total of 6 wall panels has been tested in shear (Tab. 1): two wall URM (Unreinforced masonry) wall panels (Panels No. 1 and 2) were tested in a previous experimental investigation (Borri et al. 2014). These stone work panels, identical in dimensions and constituent materials with the remaining four wall panels, were used here to assess the effectiveness of the proposed repair technique by comparison between test results. Four cracked URM panels have been repaired using two different methods: for Panel No. 3, the repair method consisted in sealing the shear cracks (Figure 5a) only using a new cement mortar. Finally, Panels No. 4, 5 and 6 were repaired by sealing the shear cracks and reinforced with the proposed method (Figure 5c). It is worth noting that the diagonal load used to test in shear the repaired wall panels was applied along the uncracked panel's diagonal (arrows in Figure 5b).

The 1200x1200x400 mm wall panels were designed for usual mechanical resistance of URM stone masonry, using the historic construction method used in Italy for stone work masonry. Each wall panel was constituted of two adjacent masonry leaves: ashlar (rubble) stone masonry was used and there were no through stones connecting the two masonry leaves. The aim was to reproduce in the laboratory a typical double-leaf stone masonry wall. This type of masonry is common in historic constructions not only in Italy, but in several countries in Europe and Asia.

Stones were only barely cut into a parallelepiped shape, with the longest dimension of about 200-220 mm. These were made of solid, high-density (2000-2200 kg/m^3), white-coloured, sedimentary, calcareous rocks. The mortar joints were thick (10-20 mm) and made of a mix of sand and hydraulic lime (mix design 250 kg lime per m^3 of conglomerate).

Table 1. Test Program.

Test No.	
MP1-UR MP2-UR	Control (Unreinforced) Wall Panels
MP3-REP	Only Crack-Sealing Repair
MP4-RIN MP5-RIN MP6-RIN	Repaired by Crack-Sealing & Reinforced with the proposed combined method

The properties of the stones and the mortars used to seal the existing cracks, to repoint the joints (Reticolatus face) and to level the masonry surface at interface with the CLT panel are shown in Table 2. The three mortars have been labelled in Table 2 using the letter designation LE, RI and RA, respectively. The mechanical properties of the mortars were determined according to the EN 1015-11 standard (2007). The properties of the steel cords and the CLT panels are reported in Tables 3 and 4. These were taken from the technical data sheets of the producer.

Table 2: Mechanical parameters of mortars and stone.

	Mortar LE	Mortar RI	Mortar RA	Stone
Weight density (kg/m^3)	2129	1807	1717	2451
Volume Mix design (lime:sand:cement)	1:2:1	Pre-mixed	Pre-mixed	-
Sample dimensions (mm)	160x40x40*	160x40x40*	160x40x40*	100x100x100*
Compressive Strength (MPa)	38.41	14.41	10.76	34.8
Sample Size	12	18	18	5
Compressive Strength CoV (%)	14.8	38.3	19.3	13.1
Bending Strength (MPa)	6.13	4.89	4.01	-
Sample Size	6	9	9	-
CoV (%)	12.6	17.9	14.6	-

*nominal dimensions

Table 3: Mechanical properties of steel cords (producer data sheet) (EN 10088-1).

Type	Stainless steel (AISI 316)
Nominal cord diameter (mm)	3
Nominal cord filament (mm)	0.33
Number of filaments	49
Cross section (mm^2)	4.19
Tensile Strength (MPa)	1416 (characteristic value)
Young's Modulus (GPa)	81.5 (mean value)

Table 4: Mechanical properties of CLT panel (producer data sheet)

Wood species	Douglas (*Pseudotsuga Menziezii*)
Weight Density (kg/m^3)	500
Perpendicular to grain compressive strength (MPa)	2.9
Young's Modulus (Perpendicular to grain) (MPa)	430
Young's Modulus (Parallel to grain) (MPa)	13000
Shear Modulus (MPa)	810

2.2 Test arrangement

Wall panels were tested in shear using the metallic profiles shown in Figure 6a, according to the requirements of ASTM E519 (2010) and RILEM (1994) standards (diagonal tension test), and assuming an unconfined test layout (without compressive vertical loading), in line with common practice used for shear testing of masonry members in earthquake engineering.

Steel loading shoes were used for testing the wall panels. No modifications were made based on the dimensions provided in ASTM E519 to produce the loading shoes. Prior to testing each wall panel, the panel was left to dry to the ambient lab environment for a duration of a minimum of 30 days. The CLT panel was subsequently applied. The panels were not tilted to the conventional 45 degree angle for diagonal loading, but these were tested in the same position they were during their construction (Figure 6a). During the shear tests, wall panels rested over a timber pallet: according to Brignola et al. (2009) this did not have an effect on the structural response of the panels under shear loading.

Instrumentation for testing was provided by way of six displacement inductive transducers (LVDTs), set up on both faces of the panel as shown in Figure 6b. Four LVDTs were placed to measure the elongations and the shortenings of the panel's diagonals on both faces, two additional LVDTs were used to measure the horizontal out-of-plane displacements on the end-points of the unloaded panel's diagonal. The diagonal compressive force was provided by a 50t-capacity hydraulic jack.

Time, magnitude of the diagonal load (P) and shortenings/elongations (Δl) of the LVDTs were measured during each shear test. More details regarding the stress analysis of a masonry wall panel under shear loading can be found in Calderini et al. (2010) and Menna et al. (2015). From these data it was possible to calculate the shear stress (τ), shear strength (τ_0), the masonry tensile strength (f_t), diagonal strains in tension and compression (ε_t and ε_c, respectively) and the angular strain (γ).

$$\tau = 1.05\frac{P}{A} \qquad 1.$$

where A is the horizontal cross section of the wall panel (A=1200x400 mm).

$$\tau_0 = \frac{P_{\max}}{3A} \qquad 2.$$

$$f_t = 1.5\tau_0 \qquad 3.$$

$$\varepsilon_c = \frac{1}{2}\times\left(\frac{\Delta l_{cA}}{l_{cA}} + \frac{\Delta l_{cB}}{l_{cB}}\right) \qquad 4$$

$$\varepsilon_t = \frac{1}{2}\times\left(\frac{\Delta l_{tA}}{l_{tA}} + \frac{\Delta l_{tB}}{l_{tB}}\right) \qquad 5.$$

$$\gamma = |\varepsilon_t| + |\varepsilon_c| \qquad 6.$$

where Δl_{cA} and Δl_{cB} are the diagonal shortenings along the compressed panel's diagonal on face A and B, respectively, and l_{cA} and l_{cB} are the corresponding gage lengths. The subscript t was used to identify the elongations and gage lengths of the stretched diagonal.

It was also possible to plot the shear stress vs. angular strain graphs and to gain important information about the stiffness and ductility response of both unreinforced and reinforced wall panels. The shear stiffness (G) of each panel was calculated using two procedures: as the slope value of the secant line to the stress-strain curve between 10 and 40% of the maximum load and corresponding strain values (G_1) and as the slope value of the secant line to the stress-strain curve between 15 and 50 kN and corresponding strain values (G_2).

3. Test results and analysis

Test results for both unreinforced, repaired and reinforced panels are shown in Table 5. Two unreinforced wall panels were tested in shear. The mean lateral load capacity was 134.3 kN, corresponding to a shear strength τ_0 of 0.99 MPa. Results in terms of shear moduli were similar for both G_1 and G_2 (2693 and 2442 MPa, respectively). Test results of URM panels were highly scattered, but this it is sometimes possible for rubble stone work masonry (Corradi and Borri, 2018). The structural response of the wall panels depends not only on the mechanical properties of the constituent materials (stone and mortar), but also on their dimensions and arrangement. This implies that the different masons hired for construction may also have an effect on the structural response of the panels. Stones of Panel No.2 were larger compared to the ones used in Panel No.1 and mortar joints were thicker for panel No.1.

The shear strength of repaired sample (MP3-REP) was 0.128 MPa. This was about 30% bigger compared to URM panels. We can say that, by sealing the shear cracks and by testing this panel along the other diagonal, it was possible to obtain a shear capacity similar to the one measured for URM panels. On opposite, the shear moduli were very different: for test MP3-REP, G_1 and G_2 were 791 and 1089 MPa, respectively. These values varied between 30 and 45%, compared to URM panels.

The application of two different retrofitting methods (having different shear stiffness's and strengths) for the two wall leaves produced different responses in terms of strains. During the initial elastic phase, the values of the deformations of the wall leaves are very similar. After the formation of the first cracks of the wall leaf reinforced with the steel cords, the shear stiffness of this leaf dropped and a re-distribution of the shear load between the two wall leaves, producing asymmetry in deformations and stresses. This asymmetry induced out-of-plane displacements of the wall panels that was clearly noted at the end of the tests (Figure 7). This was the consequence of the fact that the steel-cord-reinforced wall leaf failed before the CLT panel. The diagonal load, deformations and out-of-plane deflections vs. time plots are shown in Figure 8. The out-of-plane deflections were measured at a point located at the panel's edge along the unloaded diagonal. In Figure 8, a vertical line shows the moment in which cracks start developing. After the formation of the cracks, the structural response of the wall panels cannot be described using the theory of an in-plane loaded plate and equations (1)-(8) cannot be used for calculation of the mechanical parameters.

Table 5: Results of tested carried out on stone masonry panels.

Test No.	Max Load P_{max} (kN)	$P_{max, reinforced}$ / $P_{max, repaired}$ (%)	Shear Strength τ_0 (MPa)	Shear Modulus G_1 (MPa)	Shear Modulus G_2 (MPa)
MP1-UR	161.6	-	0.117	3123	2759
MP2-UR	107.0	-	0.081	2254	2125
(mean)	(134.3)		(0.099)	(2693)	(2442)
MP3-REP	179.4	-	0.128	791	1089
MP4-RIN	282.8	158	0.191	1489	3382
MP5-RIN	226.9	126	0.157	1661	1485
MP6-RIN	292.1	163	0.193	1750	4365
(mean)	(267.26)	(149)	(0.180)	(1633)	(3077)

It can be noted that a significant increase in lateral-load capacity was measured for wall panels repaired with the proposed method (Test No. MP4-RIN and MP6-RIN). The shear strength of these wall panels was 0.18 MPa and this value is about 40% higher compared to the shear strength of the panel repaired by sealing the shear cracks with new mortar. It can be concluded that the combined reinforcement using the Reticulatus technique and a CLT panel is able to produce significant increases in lateral load capacity of URM stone masonry.

The average shear modulus of the reinforced wall panels was 1633 MPa, compared to 791 MPa measured for the repaired-only panel (MP3-REP). The shear stress versus angular strain plot is shown in Figure 9, for both unreinforced and reinforced wall panels. The shear stress versus angular strain curves of reinforced wall panels have been shown in two colors: red and grey. The initial part of the curves is red, while these are grey when out-of-plane deflections started to occur. This was done to better identify the different structural responses of the wall during in-plane loading.

As previously mentioned, equations (1)-(4) cannot be used for analysis of the non-elastic phase of the shear test. Comments and analysis regarding the ductility of the walls are difficult to make. Using a simplistic qualitative analysis, it can be noted from the shear stress vs. angular strain plot (Figure 9) that post-elastic behavior is different for unreinforced and reinforced panels: after the maximum lateral capacity was reached, we observed a subsequent reduction of the residual (post-elastic) lateral capacity for unreinforced panel. On opposite, the post-elastic phase of reinforced wall panels was characterized by negligible reductions of the lateral capacities. We even noted small increments of the lateral capacity during the post-elastic phase for tests MP4-RIN and MP5-RIN. This unusual response is likely the consequence of the use of timber (CLT panel), having high tensile strength and plastic behavior under compressive loads.

For the failure mode, a single crack developed along the compressed diagonal of URM and repaired specimens (Figures 10 and 11). This crack passed through the full thickness of the wall panel, following a zig-zag pattern along the mortar joints (Figure 11). In a similar way, the failure mode of the reinforced wall panels consisted in the opening of several parallel shear cracks on the Reticulatus-reinforced wall leaf (Figures 12 and 13). These cracks had a diagonal orientation parallel to the direction of the diagonal shear load. CLT panels did not exhibit a significant damage (no cracks were recorded). However, after the test, once removed the test apparatus, phenomena of embedment of the transverse steel bars in the wood were observed (Figure 12b).

Due to the different mechanical properties in terms of strengths and stiffness's of two reinforcement methods (Reticulatus & CLT panel), the subsequent detachment of the CLT panel from the other wall leaf was recorded at failure.

4. Conclusions

A new retrofitting method is developed using steel cords and CLT panels to increase the shear response of cracked stone masonry wall panels. Based on the findings of this investigation, the following observations can be drawn:

1. CLT-steel cord reinforced wall panels may provide effective repairing and retrofitting solutions for pre-existing buildings. First experimental results indicate that is possible to enhance the lateral capacity of wall panels using the combined method proposed in this study. However, there is a need for a broader experimental basis, using different masonry typologies (from squared stone masonry to pebbles, from brickwork to soft stone, etc.) and different types of CLT panels and connection methods in order to better study the behaviour of this retrofitting method and calibrate the design procedures before a real application can start.

2. At high load levels the behaviour of the CLT-steel cord reinforced panels is no longer governed by the elementary elastic theory. Cracking occurs in the mortar joints and detachment of the CLT panel from the masonry substrate. As a consequence the structural response of both unreinforced and reinforced panels was highly inelastic.

3. The application of the combined reinforcements cause moderate increments of the shear moduli. The different normal stiffness's of the two reinforcement materials induced out-of-plane displacements of the reinforced panels under in-plane loading. In order to prevent this phenomenon (asymmetric deformation of the two wall leaves), more effort is required for an adequate design of the proposed retrofitting method.

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Figure 1. The two retrofitting methods used in combination to reinforce a wall panel subjected to diagonal loading (shear test): (a) Steel cords (Reticulatus); (b) CLT panel.

Figure 2. a) Wall vertical cross section: detail of the proposed reinforcement method. b) Detail of the bar-to-cord connection: steel eyelet, in which the steel cords can slide, welded to the threaded round bar.

Figure 3. Reinforcement with the steel cords: a) cord application in the joints after removal of pre-existing mortar; b) detail of the reinforcement system c) a transverse steel bar used to connect the cords to the CLT panel on the other wall face; d) steel eyelet welded to the steel bar, e) wall layout at the end of retrofitting operations.

Figure 4. Reinforcement with CLT panel: a) levelling of the wall face before CLT panel application; b) CLT panel and transverse steel bars; c) application of the steel washers and tightening of the nuts.

Figure 5. The effectiveness of the proposed retrofitting method has been studied using cracked wall panels (these were tested in a previous experimental investigation): a) cracked wall panel, b) repair works by sealing the shear cracks with new mortar (the two arrows indicate the direction of the diagonal load. This was applied along the other panel's diagonal, compared to previous experimental investigation).

Figure 6. Test layout.

Figure 7. The application of two different retrofitting methods (CLT panel and steel cords (Reticulatus method)) to reinforce a wall panel induced out-of-plane displacements during diagonal loading.

Figure 8. Diagonal load, diagonal deformations and out-of-plane deflections vs. time for Test No. MP4-RIN. D3 and D4 are shortenings and elongations, respectively, of wall face reinforced with the Reticulatus method. It can be noted that out-of-plane deflections start occurring near the maximum diagonal load.

Figure 9. Shear stress versus angular strain plot for unreinforced, repaired and reinforced walls.

Figure 10. Failure mode of Panel 1 (Test MP1-UR) and Panel 2 (Test MP2-UR)

Figure 11. Failure mode (both faces, Test No. MP3-REP).

Figure 12. Failure mode of Panel N. 4 (Test No. MP4-RIN): a) wall face reinforced with the Reticulatus method; b) detail of the steel bar embeddment.

Figure 13. Failure modes a) MP5-RIN; b) MP6-RIN