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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

41 P is the diagonal compressive load
 42 P_{max} is the maximum diagonal compressive load
 43 A is the area of the horizontal cross section of the wall panel
 44 f_t is the masonry tensile strength
 45 Δl_{cA} is the shortening of the compressed diagonal on panel's side A
 46 Δl_{tA} is the elongation of the diagonal in tension on panel's side A
 47 l_{cA} is the gage length of the compressed diagonal on panel's side A
 48 l_{tA} is the gage length of the diagonal in tension on panel's side A
 49 ε_c is the diagonal compressive strain
 50 ε_t is the diagonal tension strain
 51 γ is the angular strain
 52 G_1 is the tangential elastic modulus of the masonry, given by the slope value of the secant
 53 line to the shear stress-angular strain curve between 10 and 40 % of the maximum
 54 diagonal load P_{max} and corresponding strain values.
 55 G_2 is the tangential elastic modulus of the masonry, given by the slope value of the secant
 56 line to the shear stress-angular strain curve between 15 and 50 kN and corresponding
 57 strain values.

58

59 **Introduction**

60 There is growing evidence that natural materials and sustainable solutions are beginning to be
 61 taken seriously as viable materials to retrofit and repair old masonry structures (Righetti et al.
 62 (2016), Papayianni and Pachta (2017)). Old masonry structures (public and private buildings,
 63 schools, hospitals, etc.) and infrastructures (bridges, lighthouses, town walls, etc.) generally fulfil
 64 primary functions, and their reinforcement is a priority not only in seismic prone areas (Coburn
 65 and Spence (1992), Binda and Saisi (2005), Cardoso et al. (2005), Rota et al. (2014)).

66

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

72

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

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

81

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

95

96 Another traditional method of reinforcing wall panels is repointing. Repointing is the task of
97 renewing the outer portion of the mortar joint, with new mortar (Alcaino and Santa-Maria (2008),
98 Corradi et al. (2008)). A new development is to reinforce the new mortar used for repointing with
99 steel elements (strips, cords, rods). In past investigations, Borri et al. (2014) and Castori et al.
100 (2016) explored the effect of steel cord reinforcement on the structural response of in-plane
101 loaded wall panels (Reticulatus method). They established that steel cords increased both the
102 deformation capacity and shear strength of the reinforced panels. Repointing of stonework can
103 be used to give structural support and cohesion to individual stone elements, either on its own
104 or in conjunction with grouting. Repointing stone work provides a primary defence against water

105 ingress, as well as having an important structural role for the stability of the building — it is also
106 a critical aspect of the building's maintenance schedule.

107

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

115

116 Another aspect should be also considered: one of the most critical flaws of traditional solid wall
117 construction is its low energy efficiency. Several non-structural methods have been recently
118 developed in response to improving technology and the implementation of new building
119 regulations related to energy efficiency. Until the 1970s there was no maximum value for the
120 thermal transmittance (U) of the layers that make up a building envelope. From the 1980s the U
121 value was gradually reduced, requiring the introduction of insulation. With the release of the new
122 Building Regulations in many parts of Europe (European Directive 2002/91/CE and
123 2010/31/UE; 2014 UK Building Regulations, Italian Act DM 26/6/2015, etc.) we face a situation
124 where all the elements in the fabric envelope – the roof, walls and floor – need to have a very
125 low U-value. In this situation the insulation of old solid walls is a challenging task. Rigid
126 Polyurethane (PUR), polyisocyanurate (PIR) or phenolic foam are typically used for this
127 purpose. Expanded (EPS) or extruded (XPS) are also employed to reduce the thermal
128 transmittance of solid walls. All these materials are often under the form boards, fixed, glued or
129 mechanically attached to the solid walls, without any structural function. Recently, wood fibre
130 boards have been used for this purpose: wood fibre boards are also rigid and have a thermal
131 conductivity similar to EPS.

132

133 Beyond the research efforts described above, the authors of this paper have proposed the use
134 of composite grids embedded into a thermal-insulating mortar coating (Borri et al., 2015). This

135 work experimentally studied the shear response of individual wall panels to help develop high
136 walls' shear capacity and reduced thermal transmittance.

137

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

141

142 One additional study involving the application of a CLT structure connected to the masonry
143 walls looked at issues of seismic resistance (Pozza et al., 2017). This work showed the need to
144 properly prepare the connection of the CLT structure to the horizontal diaphragms (floor) of the
145 masonry building. For interventions on heritage masonry buildings, the reversibility and the
146 compatibility of new materials and techniques with historic ones is often required (Polastri et al.
147 2017). In this case, the reversibility of the proposed retrofitting intervention was guaranteed by
148 the use of steel screws as a method of connection of the CLT structure to the masonry
149 substrate.

150

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

156

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

162

163 **2. The proposed reinforcement method**

164 The two wall leaves of a stone wall panel were reinforced using two different methods. With the
165 aim of preserving the fair-faced aspect, one wall leaf was reinforced using the Reticulatus
166 method. This method was proposed by Corradi et al. in 2016 and consists of embedding steel
167 cords in the mortar joints (both vertical and horizontal ones). By doing this, it is possible to
168 create a nearly square or rectangular mesh, the nodes of which are connected to the
169 reinforcement on the other face of wall panel by means of transverse steel bars, according to
170 the schemes in Figures 1 and 2a. The number of transversal connections depends on many
171 factors (type of masonry, its mechanical properties, wall thickness, etc.): an established number
172 is 5-6 per m². Furthermore, the cords are connected to the transverse steel bars using standard
173 steel eyelets in which the cords can slide: thus, it is possible to apply a moderate tension to the
174 cords, so as to make them immediately functional (Figure 2b).

175

176 The ends of the steel cords were passed into holes and fixed on the other wall face. To prevent
177 stress concentration and local failures, rounded steel angles were placed near the holes
178 between the cords and the masonry (Figures 3c and 3d) and finally the steel cords were fixed to
179 eyelets, anchored to the stones, using locking devices (Figure 3d).

180 The grid and the eyelets are covered with a final layer of new mortar, making it possible to
181 preserve the fair-faced aspect of the masonry (Figure 3e).

182

183 The other panel's wall leaf was reinforced using a 60 mm-thick CLT panel. This had identical
184 dimensions as the stone masonry panel (1200x1200 mm) and it was made of three 20 mm-thick
185 layers of solid Douglas wood (*Pseudotsuga Menziesii*). According to the producer data sheet,
186 the shear modulus of this type of wood is 810 MPa. This value can be considered compatible
187 with the shear modulus of the masonry. New mortar was used to level the panel surface before
188 the application of the CLT panel (Figure 4a). Furthermore, to protect the CLT (Figures 4b and
189 4c) from water ingress, a membrane made of a polyethylene film was interposed between the
190 stone masonry and CLT panel. The polyethylene films can be considered as vapour barriers
191 and as such detrimental to the CLT material durability due to moisture stagnation at the CLT-
192 mortar interface surface. However, the water ingress has to be prevented. More analysis is

193 necessary in order to assess the best method to prevent wood degradation at interface with
194 masonry.

195

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

201

202 **2. Specimens Construction, Instrumentation and Test layout**

203

204 **2.1 The wall panels**

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

216

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

223 Stones were only barely cut into a parallelepiped shape, with the longest dimension of about
 224 200-220 mm. These were made of solid, high-density (2000-2200 kg/m³), white-coloured,
 225 sedimentary, calcareous rocks. The mortar joints were thick (10-20 mm) and made of a mix of
 226 sand and hydraulic lime (mix design 250 kg lime per m³ of conglomerate).

227

228

Table 1. Test Program.

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

229

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

237

238

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	-

239

*nominal dimensions

240

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)

241

242

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

Wood species	Douglas (<i>Pseudotsuga Menziesii</i>)
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

243

244

245 **2.2 Test arrangement**

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

250 Steel loading shoes were used for testing the wall panels. No modifications were made based
 251 on the dimensions provided in ASTM E519 to produce the loading shoes. Prior to testing each
 252 wall panel, the panel was left to dry to the ambient lab environment for a duration of a minimum
 253 of 30 days. The CLT panel was subsequently applied. The panels were not tilted to the
 254 conventional 45 degree angle for diagonal loading, but these were tested in the same position
 255 they were during their construction (Figure 6a). During the shear tests, wall panels rested over a

256 timber pallet: according to Brignola et al. (2009) this did not have an effect on the structural
257 response of the panels under shear loading.

258

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

265

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

$$272 \quad \tau = 1.05 \frac{P}{A} \quad 1.$$

273 where A is the horizontal cross section of the wall panel ($A=1200 \times 400$ mm).

$$274 \quad \tau_0 = \frac{P_{\max}}{3A} \quad 2.$$

$$275 \quad f_t = 1.5\tau_0 \quad 3.$$

$$276 \quad \varepsilon_c = \frac{1}{2} \times \left(\frac{\Delta l_{cA}}{l_{cA}} + \frac{\Delta l_{cB}}{l_{cB}} \right) \quad 4$$

$$277 \quad \varepsilon_t = \frac{1}{2} \times \left(\frac{\Delta l_{tA}}{l_{tA}} + \frac{\Delta l_{tB}}{l_{tB}} \right) \quad 5.$$

$$278 \quad \gamma = |\varepsilon_t| + |\varepsilon_c| \quad 6.$$

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

282

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

289

290 **3. Test results and analysis**

291 Test results for both unreinforced, repaired and reinforced panels are shown in Table 5. Two
292 unreinforced wall panels were tested in shear. The mean lateral load capacity was 134.3 kN,
293 corresponding to a shear strength τ_0 of 0.99 MPa. Results in terms of shear moduli were similar
294 for both G_1 and G_2 (2693 and 2442 MPa, respectively). Test results of URM panels were highly
295 scattered, but this it is sometimes possible for rubble stone work masonry (Corradi and Borri,
296 2018). The structural response of the wall panels depends not only on the mechanical
297 properties of the constituent materials (stone and mortar), but also on their dimensions and
298 arrangement. This implies that the different masons hired for construction may also have an
299 effect on the structural response of the panels. Stones of Panel No.2 were larger compared to
300 the ones used in Panel No.1 and mortar joints were thicker for panel No.1.

301

302 The shear strength of repaired sample (MP3-REP) was 0.128 MPa. This was about 30% bigger
303 compared to URM panels. We can say that, by sealing the shear cracks and by testing this
304 panel along the other diagonal, it was possible to obtain a shear capacity similar to the one
305 measured for URM panels. On opposite, the shear moduli were very different: for test MP3-
306 REP, G_1 and G_2 were 791 and 1089 MPa, respectively. These values varied between 30 and
307 45%, compared to URM panels.

308

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

323
324

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)

325

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

332

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

340

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

351

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

361

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

365

366 **4. Conclusions**

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

370

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

379

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

384

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

390

391

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397

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500

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

503

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

507

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

512

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

516

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

522

523 Figure 6. Test layout.

524

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

528

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

533

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

535

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

537

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

539

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

542

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

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