IN-SITU TESTED BRICK MASONRY WALLS STRENGTHENED WITH HORIZONTAL CARBON STRIPS AND FRP MESH

M. Jarc Simonič[1], S. Gostič[2] and R. Žarnić[3]
[1] Building and Civil Engineering Institute ZRMK Ljubljana, Dimičeva 12, Slovenia, mojca.jarc-simonic@gi-zrmk.si
[2] PhD, Building and Civil Engineering Institute ZRMK Ljubljana, Dimičeva 12, Slovenia, samo.gostic@gi-zrmk.si
[3] PhD, Professor, University of Ljubljana, Faculty of Civil Engineering and Geodesy, Jamova 2, Slovenia, roko.zarnic@fgg.uni-lj.si

ABSTRACT
One of the most promising methods of strengthening of masonry walls against in-plane load that usually appears in earthquake is application of reinforced polymers (FRP) to the surface of the wall. Within the framework of European FP7 research project PERPETUATE (focused on development of new computation models for masonry and strengthening techniques) new innovative configurations of fibre reinforcement placement on clay brick masonry were developed. Six specimens (width / height / thickness = 100 / 210 / 52 cm) of walls, made of solid bricks in low strength lime-cement mortar, positioned in approximately 100 years old building, were in-situ tested by displacement controlled horizontal cyclic load. Innovativeness of the first presented reinforcement is in efficiency of very narrow horizontally placed CFRP strips, providing the confinement effect to masonry brick rows and of the second the use of less fragile mortar for placing GFRP mesh over entire surface. Two specimen of each were compared to two unreinforced ones. The biggest gain of narrow horizontal strips was in ultimate displacement, ductility and dissipated energy, but also in low costs, easy mounting and low interference with the buildings (appreciated in historic monuments). Both types of reinforcement significantly increase the shear resistance. The horizontal confined strips also successfully overcome the problem of FRP detaching from masonry wall, the main problem of FRP-masonry common behaviour.

KEYWORDS: masonry, FRP, strengthening, in-situ test, shear strength, ductility

INTRODUCTION
Major part of existing buildings which are classified as valuable architectural monuments of cultural heritage consist of brickwork masonry. Unfortunately unreinforced brick masonry (URM) buildings have suffered extensive damage during past earthquakes due to in-plane shear actions and poor connection of load bearing walls. Low seismic resilience of this type of buildings showed the need for improved reinforcing techniques for the structural retrofit since the conventional strengthening methods (changing of weak mortar in joints, jacketing of URM walls with reinforced concrete and similar) are either disruptive to residents, realization takes a lot of time or they are not acceptable from protection of cultural heritage authenticity point of view. Strengthening methods using new materials (Fibre Reinforced Polymers) promise to overcome those problems. After application FRPs can be removed from the original walls if so later required by the heritage conservation.

One of the first studies of effect of strengthening masonry wall by fibres was done by Croci et al. (1987), followed by Triantafillou et al. (1993), (1997), Schwegler (1994) etc. and since this topic has still a lot of open questions many of the researches have been performed in the beginning of this century too (Borri et al. (2001), Valluzzi et al. (2002), Santa-Maria et al. (2004), Gostič et al. (2004),(2006), Alcaino et al. (2007), Marcari et al. (2007), Tomaževič et al. (2010) etc). Research up to date includes wide spectra of strengthening techniques of masonry with FRPs. In general conclusions showed the improvement of shear strength and increase of ultimate displacement. Depending on the boundary conditions sometimes diagonal reinforcement is more efficient and sometimes horizontal. Authors are uniform in conclusions that efficiency of vertical reinforcement is low due to flexibility and local detaching of FRP near shear cracks. When diagonal configuration is glued on surface without anchoring it did not perform so well because the failure mechanism was governed by detachment of FRP from the masonry surface. When properly anchored the increment of strength might be greater comparing to horizontal stripes because diagonal strips are placed in same direction as acting of shear stresses. Post peak decrease of load capacity is slower with horizontal reinforcement, while diagonally reinforced wall specimens usually exhibit sudden brittle failure. Horizontal stripes have important advantage comparing to diagonal strips – they can be placed as confinement around the wall. In such case the failure mechanism can change from diagonal shear of un-reinforced masonry to compressive (toe) failure of masonry within the FRP confinement combined with diagonal shear cracks. The confinement also increases compressive strength if concentrated in zones with highest compressive stresses. The strengthening always has to be double sided (or confined) to avoid out-of-plain bending when loaded with in-plane load. Authors in many cases stress that the most critical is detachment of FRP from the masonry surface. The increment of strength can also be achieved by surface coating with FRP meshes or fabrics. Tomaževič et al, 2011 report that the failure mechanism in such case is shear failure with detaching of the coating from surface.

The in-plane seismic performance of unreinforced masonry walls (URM) before and after their retrofit using fiber reinforced materials is further investigated in our research. Investigation was focused on horizontal strengthening with narrow CFRP fabric stripes and on surface coating with GFRP meshes.

EXPERIMENTAL CAMPAIGN

In-situ tests were performed on an old masonry building dated from 1874 with low seismic resilience due to poor connection and low shear strength of load bearing walls (Figure 1). As such it is similar to masonry box-type cultural heritage buildings. Building had two stories: ground floor and first floor, each 3,2 m high. Load bearing masonry walls were made with solid clay bricks (in average roughly 30 × 12 × 6 cm) and weak lime mortar. The same materials are common for cultural heritage buildings from that period. The thickness of the masonry wall was 52 cm. Six undamaged specimens were isolated by cutting from the walls: two reference unreinforced specimens (marked as R), two strengthened with narrow horizontal stripes (S) and two with GFRP mesh (M) placed over entire surface (Figure 2). Narrow CFRP stripes were glued to surface with epoxy based resin while GFRP mesh was laid into modified cement mortar. Specimens' height was 2.1 m and width 1.0 m. To get the basic information about existing masonry a compression test was also performed where the compressive strength and modulus of elasticity were determined.

Figure 1 : Tests were performed on an old building

Surface of the wall area designated for gluing was prepared by removing plaster and grinding the loose parts. In the case of narrow stripes the edges were rounded on appropriate places to avoid CFRP fibers bend cracking. Unevenness of the surface was corrected with cement mortar (class CS III according to EN 998-1) in thickness up to 5 mm. A thin layer of epoxy resin mixed with fine sand was placed under CFRP stripes and then the CFRP stripes were placed with wet lay-up technique. Stripes Carbon*iar* 1,5 cm wide with weight of 800 g/m^2 were used. Stripes were placed horizontally forming the confinement around the wall specimens on 10 levels. Two of them in each compression loaded areas and one in the middle of each half of specimen (Figure 2, b with upper and lower half of specimen).

Figure 2 (a, b, c): Three configurations (un-strengthened R, with horizontal stripes S and mesh M) were prepared for shear testing

GFRP meshes (Sika WrapGrid 350G) were installed in surface applied modified cement mortar (Sika MonoTop 722 Mur with declared compressive strength 27,13 MPa) in common thickness of 1 cm (Figure 2, c). The strengthening was carried out by the company GRAS from Ljubljana.

COMPRESSION TESTS

To determine the compressive strength and modulus of elasticity a test on brick wall sample was performed. Dimensions of the specimen were (width/height/thickness) 100 x 100 x 48 cm. In-situ compression tests were carried out with a hydraulic jack of 1300 kN capacity (Figure 3). With the system of steel ties and displacement controlled hydraulic jack the sample was tested in-place. Due to limitations of available equipment (and time) a steel profile on the top of the wall did not cover entire cross section but only 101 cm x 28 cm. This provoked a splitting crack through the middle of the wall thickness. Vertical and horizontal deformations were measured with LVDTs mounted on one side of the wall (verticals E1, E2 and horizontal H1) and H2, H3 were horizontal on the sides. Sample was released at 88% of the maximum load and then loaded again up to failure (Figure 4). Mechanical characteristics gained in the compression tests are presented in Table1. Module of elasticity (E_W) was calculated from stresses at 1/3 of maximum divided by average of vertical strains (E1 and E2) at that point.

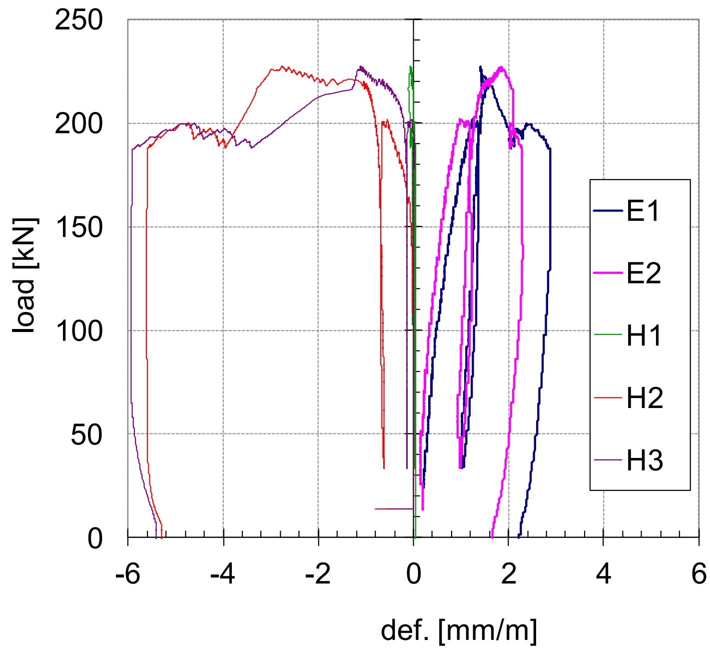


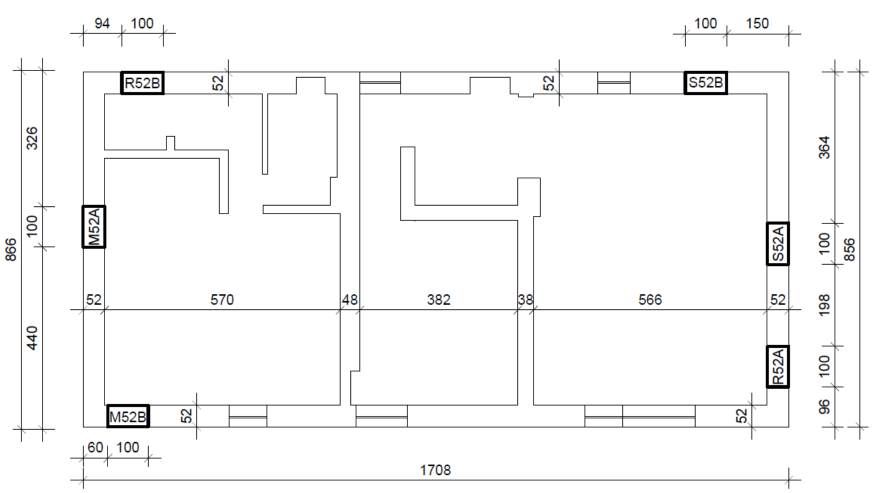
Figure 3: Compression test set up **Figure 4: Diagrams of deformations**

Table1: Results of compression test

max force [kN]	area under steel profile [m2]	compression strength [MPa]	1/3 of max strength [MPa]	vertical strains at 33% F_{max} [mm/m]	elastic modulus [MPa]
150.21	0.283	0.804	0.265	0.688	772.6

SHEAR TESTS

Six 2,1 m high and 1 m wide specimens with thicknesses 52 cm were cut out from load bearing walls for shear tests. Un-strengthened specimens for comparison were labelled R52A and R52B. Horizontally strengthened specimens were S52A and S52B while M52A and M52B had GFRP mesh in cement mortar plaster.



WALL	load G (kN)
R52A	61.31
R52B	70.85
S52A	63.60
S52B	71.57
M52A	63.52
M52B	70.33

Figure 5 : Location of test specimens in a building plan and vertical load

A hydraulic jack was applied at the middle of wall height to introduce the horizontal load therefore separating wall into upper and bottom 'specimen' of the wall each with ratio h/l = 1.05. The set up for testing the unreinforced specimen is shown on Figure 6, where three configurations are also schematically shown: M, S and R (from left to right). The specimens were thus tested as elements with symmetrically fixed ends into the surrounding masonry. Vertical load on specimens was due to dead weight of the structure above them (Figure 5). Stress due to vertical load therefore only slightly (±0.01MPa) varied around average value of 0.13 MPa.

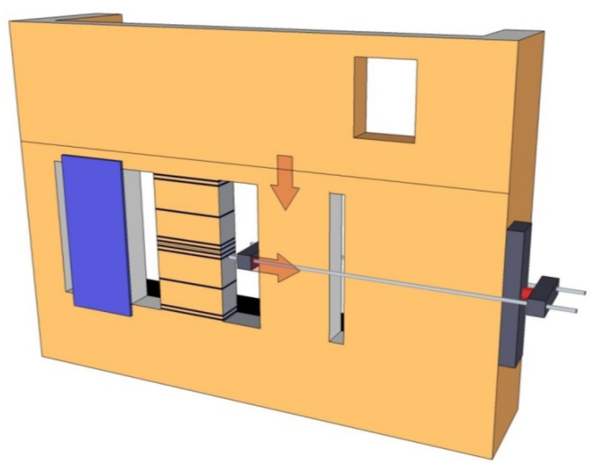


Figure 6: Shear test set-up

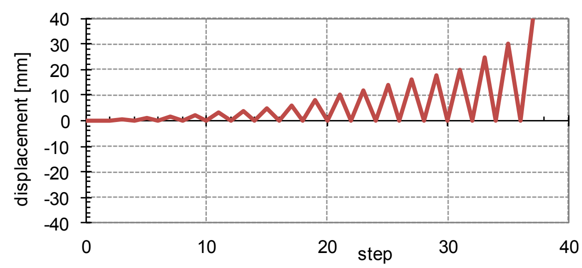
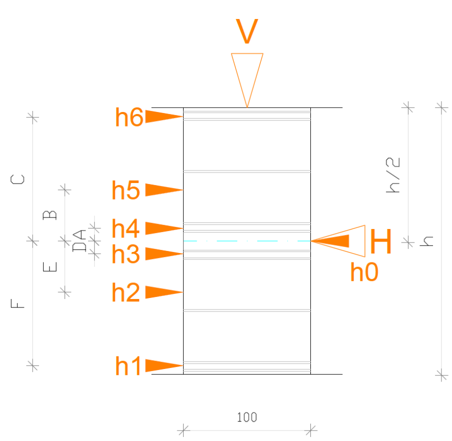


Figure 7: Loading protocol



WALL	h	A	B	C	D	E	F
R52A	220	4	37	88	3.5	36	107
R52B	210	3	29	102	3.5	35.5	102
S52A	213	4	36	101	7	36	101
S52B	202	4	35	100	4	34	86
M52A	216	6	38	106	6	44	100
M52B	225	6.5	36	99	6.5	36.5	102.5

Figure 8 : Position of horizontal LVDTs (h0, h1, h2, h3, h4, h5 and h6)

Displacements and deformations were measured with linear variable differential transducers (LVDTs; exact positions are on Figure 8). Horizontal load was measured with load cell. Loading during tests was displacement controlled and it was progressing in steps to 0.5 mm, 1.0, 1.5, 2.0, 3.0 mm etc with intermediate release near zero (Figure 7). Loading was stopped when lateral force in the current step could not reach 80% of maximum force previously achieved.

EXPERIMENTAL RESULTS

Un-reinforced specimens R52A and R52B started to show first cracks at 2 mm of horizontal displacement that was at max load for R52A (103,5 kN) and 80% of max load for R52B. They failed by propagation of diagonal cracks to width of 17 mm (R52B, Figure 9) at ultimate displacement of 12 mm (R52B). Both un-reinforced specimens fail by diagonal tensile mechanism.

On the Figure 10 the rotation versus horizontal load for lower part of wall with its envelope is presented. An envelope for upper part of the tested wall is also presented for comparison. Due to test setup the horizontal load from the hydraulic jack is distributed half to bottom part of wall and half to upper part. The forces on two halves of the wall were thus assumed identical but the deformations were different (as measured).

Figure 9: Failure by diagonal cracking

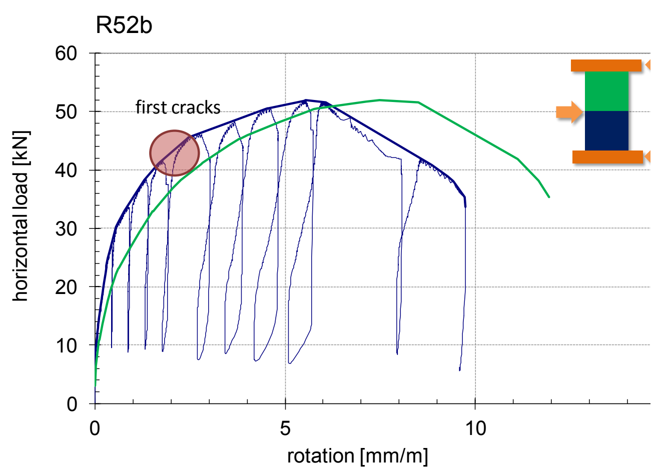


Figure 10: Envelope of rotation vs. load and occurrence of first cracks

Figure 11: Failure by diagonal cracking and crushing of comp. toe

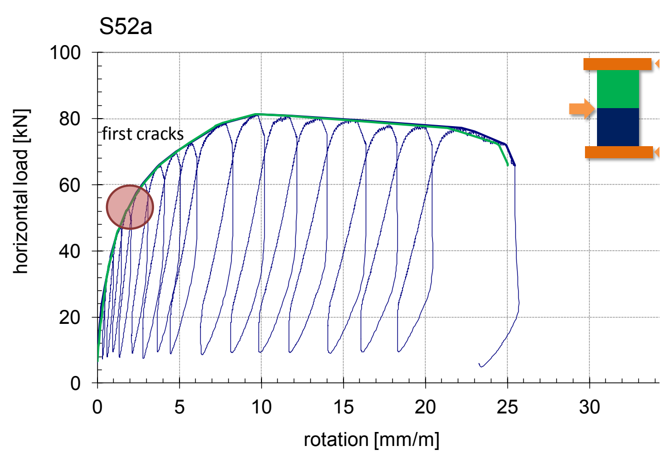


Figure 12: Envelope of rotation vs. load and occurrence of first cracks

Figure 13: Failure by detachment of GFRP mesh in mortar from the surface

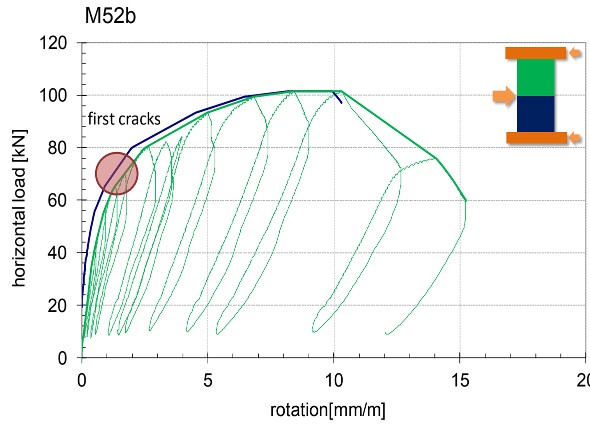


Figure 14: Envelope of rotation vs. load and occurrence of first cracks

Cracks propagation was efficiently obstructed by the CFRP strips, which resulted in formation of many minor shear cracks for specimens S (Figure 15). First diagonal cracks occurred at 65% of max load for S52A (or about 2 mm of horizontal displacement) and 77% of max load for S52B (or about 4 mm of horizontal displacement). Maximum width of crack was 5 mm (S52A) at 18 mm of horizontal displacement and 9 mm (S52B) at 25 mm displacement (Figure 11). FRP stripes haven't detached from the surface because they were well connected around the masonry pier. Load during test S52A reached 162,8 kN at max displacement of 10 mm while S52B reached 181,3 kN at displacement of 14 mm (Figure 12). Failure mechanism exhibited diagonal cracking together with crushing of compression toe. Ultimate displacements were 25 mm and 30 mm (S52A and S52B) respectively.

GFRP meshes in mortar also exhibit increment of maximum load and ultimate displacement comparing to un-reinforced specimens. Maximum load of specimen M52A was 162 kN (at 8 mm displacement) and ultimate displacement 16 mm. Specimen M52B reached maximum load 203 kN at 10 mm and failed at ultimate displacement of 16 mm (Figure 14). Cracks appeared earlier: at 1 mm (53% of max load) for M52A and at 1,5 mm (79 % of max load) for M52B. Noticing cracks earlier might be attributed to easier spotting of cracks on smooth surface. Cracks started with horizontal pattern at zones in tension and diagonal cracks. At maximum load (or at 8 mm of horizontal displacement) the detachment of coating in compression zones (Figure 13) occurred. Both specimens failed at 16 mm displacement due to overall detachment of GFRP mesh coating. Maximum width of cracks observed on the plaster was 0,8 mm (M52A) and 1,9 mm (M52B).

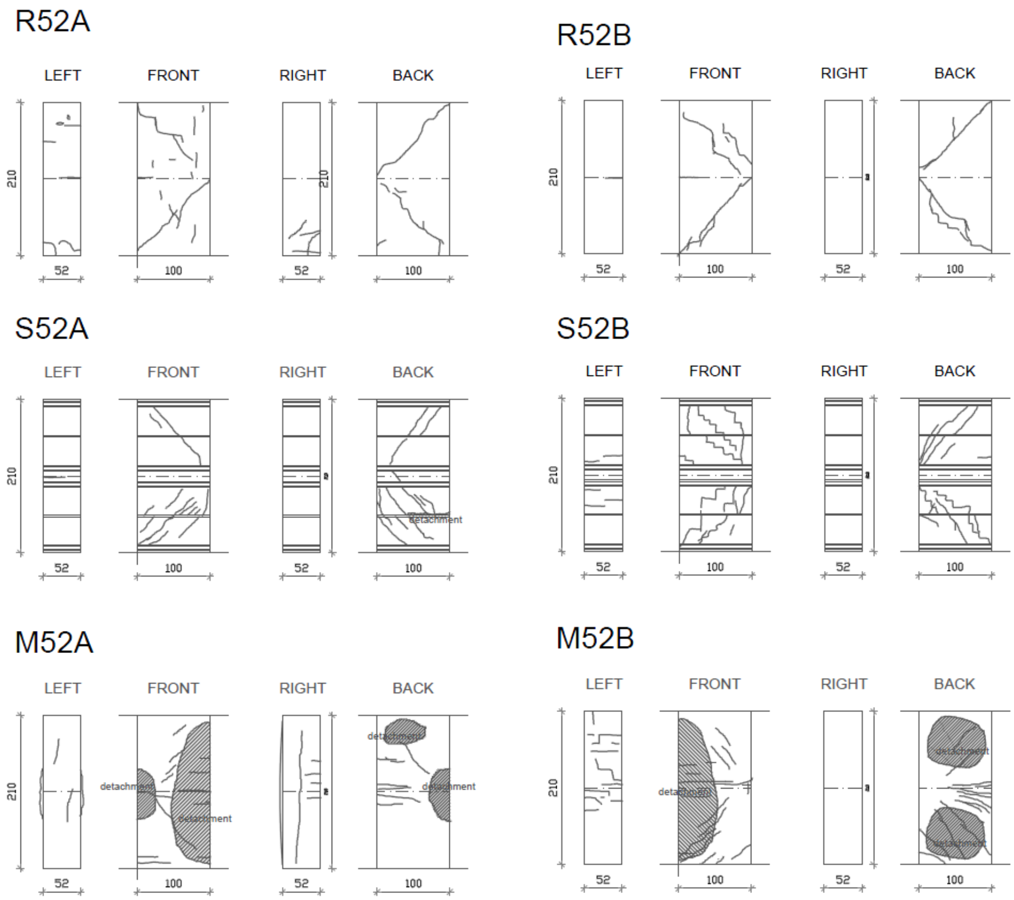


Figure 15 : Crack pattern on specimens

From the cycles of rotation vs. horizontal load it can be observed highly nonlinear behaviour of masonry. There is hardly any elastic range to be observed on the shape of envelope and damage (and most of the rotation) accumulated during horizontal load increase remained even after the release of load back near zero in each cycle. Also the secant stiffness was reduced with each cycle. But the behaviour was pretty stable even after reaching the peak load capacity. As such none of the specimens exhibited a brittle behaviour. Most ductile behaviour with a long after peak load capacity was exhibited by specimens strengthened with narrow horizontal strips (S).

Envelopes of results for all specimens are compared on Figure 16. For those envelopes horizontal load was divided by the horizontal cross section of the wall. There can be seen a certain variation of results for different specimens (a or b) or deformations of upper (marked with ^) and bottom (marked with _) part of each specimen.

The best results both in terms of strength and ductility were gained with S configuration of horizontal stripes (red lines). Strength increased on 170 % and ultimate rotation capacity on 250 % of unreinforced specimens (blue lines).

Load bearing capacity was almost doubled (180 %) with M configuration (green line). In case of M52B specimen peak load was even 196% of the reference one, but on average gained very little in increasing ultimate rotation capacity (110 %).

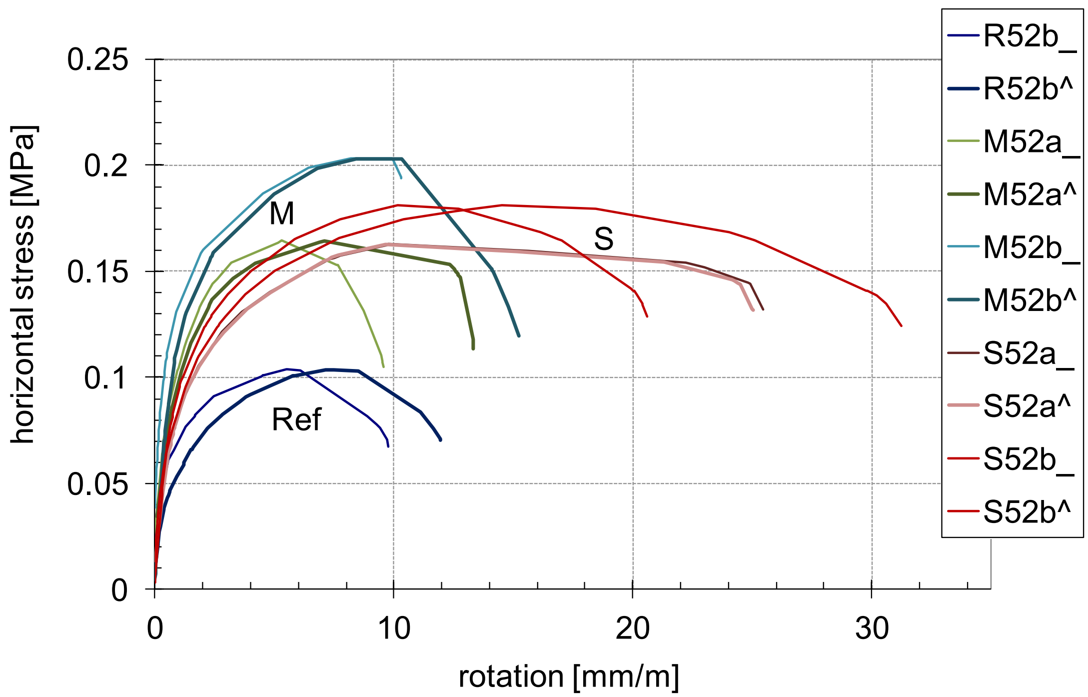


Figure 16: Hysteresis envelopes for all shear tested walls

For study of effectiveness the average values of strengthening configuration was compared to average values of URM walls. The biggest increase of shear strength and ultimate rotation capacity was achieved by configuration S (Figure 17). Narrow stripes S were thus the most effective configuration despite their relatively small size.

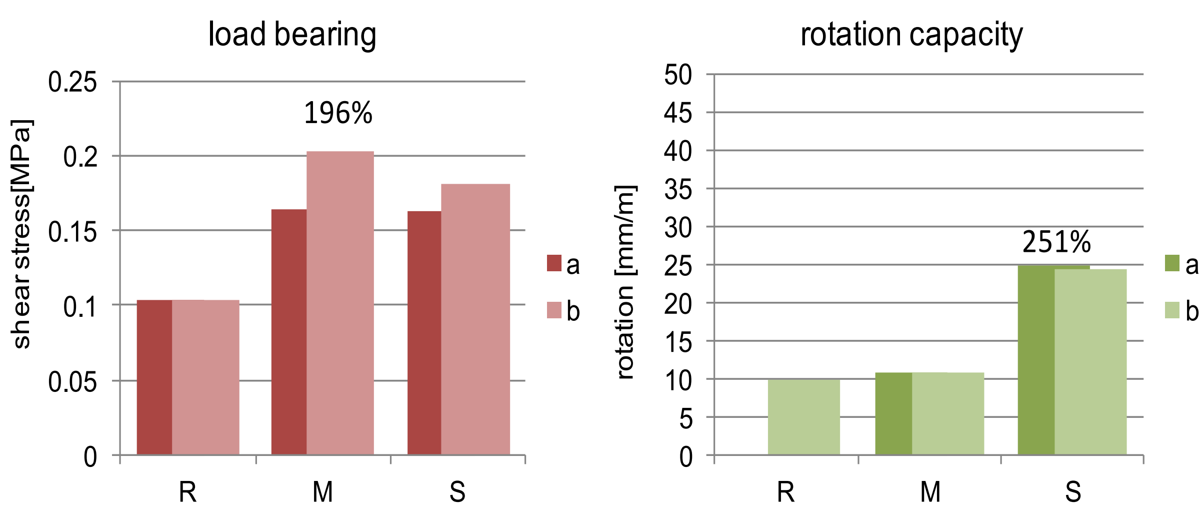


Figure 17: Effectiveness of FRP strengthening configurations M(ash) and S(trips) comparing to R(eference) specimen

CONCLUSION

The application of CFRP strips and GFRP mesh for in-plane strengthening of masonry have been tested in-situ. Tests were performed on load bearing walls of an old building. Six specimens cut from the load bearing walls were tested with half-cyclic displacement controlled horizontal load. Two specimen of each configuration were compared to two unreinforced specimens.

The (reference) un-reinforced masonry typically failed in diagonal shear. The horizontally reinforced specimens failed by masonry compressive failure within the FRP confinement combined with diagonal shear cracks. The failure mechanism of specimens strengthened with GFRP mesh in mortar was governed by detachment of GFRP reinforced plaster from the masonry surface that happened at maximum load.

The best results were gained with S configuration of narrow horizontal stripes attached to masonry surface with epoxy resin. Strength capacity was on average 170 % of reference URM and ultimate rotation capacity was 250 % of reference. Innovative narrow stripes also have advantage because of low costs, easy mounting, reversibility of application and low interference with the buildings (appreciated in historic monuments).

Reinforcement by GFRP meshes inside modified concrete based mortar (M configuration) also increased shear resistance of the walls. Load bearing capacity nearly doubled (180 %), but the ultimate rotation capacity increase was small (110 % of reference).

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