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SEISMIC BEHAVIOUR OF MULTISTOREY STRENGTHENED URM MASONRY SHEAR WALLS WITH OPENINGS: AN EXPERIMENTAL STUDY

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Abstract

To research the efficiency of technologies of strengthening of earthquake-damaged masonry buildings built from hollow clay block, an assemblage of three-storey full scale plain masonry shear walls has been tested after strengthening it by fibre reinforced polymers (FRP). Constant vertical load, typical for residential building was applied by means of a hydraulic system. Seismic loads, acting cyclically at the floor levels, were applied in the form of a linear force pattern along the height. The walls were first tested in unstrengthened state up to the occurrence of considerable damage (beyond maximum resistance limit state) and then strengthened by a system based on FRP coating. After that, the walls were retested until collapse. Because the collapse and damage occurred on the first and second floor, respectively, an additional test of only the top floor was performed. The results of the tests show that adequate strengthening solutions provide significantly improved seismic behaviour. If properly applied and anchored, the coating and the wall act together as a composite material and respond in shear.

Keywords: URM; testing; fibre reinforced polymers; strengthening;

1. Introduction

Masonry represents one of the oldest and most important building materials as it was used for majority of public and residential buildings in the past. Construction of such buildings was usually based on local resources and traditional methods of construction, while quality of materials as well as type of construction were not strictly defined nor was there control of quality. As shown by the effects caused by earthquakes in the past, seismic resistance of these masonry buildings is generally inadequate. Furthermore, recent seismic vulnerability studies [1] indicate that resistance of such brick masonry buildings does not meet the resistance demands of the currently valid code requirements for seismic assessment of existing buildings and rules for their redesign. Moreover, this is also the case for masonry built from hollow clay masonry units as recently as a couple of decades ago. The obvious solution is to strengthen existing buildings to bring them up to the required level of safety.

While a number of research papers can be found on strengthening of brick masonry, strengthening of masonry built from hollow clay blocks is much less researched. In the case of brick masonry, various types of strengthening exist. Some are based on traditional materials and techniques (reinforced cement/concrete/shotcrete coating, repointing, injecting) [2], which are effective but require major structural interventions and a temporary eviction of residents. Newer methods, which are based on FRP materials, are also effective but cleaner and easier to apply [3, 4, 5, 6]). A fair amount of confidence in the use of these systems exists and some of the solutions have already been used in practice.

There are several conceptually different ways in which the FRP material can be applied to masonry. One of them is wrapping of the walls with FRP fabric, which is very effective and least sensitive to loss of bond between the wall and the FRP-mortar composite coating. In one of recent in-situ tests a wall, strengthened in such manner, proved to be stronger than the connection of the wall to the rest of the structure, at which the failure occurred [1]. Alternatively, when wrapping is not feasible due to e.g. protected facades, it is possible to apply the coating only on one side and anchor it into the wall using steel or composite anchors [7]. Although large number of anchors may be required, the bond between the wall and the coating is still usually the weak spot. Recently a new way of strengthening using deformable polymers [8] has been tested and it was demonstrated that the bond between the masonry and the coating was not lost even at very large deformations (for masonry).

Despite the fact that due to the industrial pressure some of the modern solutions have already been used in practice, they have not been entirely confirmed yet by the experts. As regards the URM buildings, built of hollow clay masonry blocks, however, the experimental results are scarce and further experimental investigations of new types of strengthening are therefore welcome and necessary, as the strengthening effects are unknown and failure mechanisms are still not identified. In order to clarify some of these issues, an experimental research project has been launched at Slovenian National Building and Civil Engineering Institute in Ljubljana. Some test results of the experimental research will be presented and discussed in this paper.

2. Materials, multistorey shear walls and type of strengthening

2.1 Materials

Hollow clay units with nominal dimensions (length/width/height = 290/190/190 mm) and classified as group 2 units according to Eurocode 6 (EN 1996-1-4) were used for the construction of the specimen. The characteristic compressive strength of units, tested according to EN 771-1, was 15 MPa. Mortar used for building was general purpose mortar with volumetric proportion of cement:lime:sand = 0.5:1:8. Sand with 4 mm maximum grain size was used. The quantity of added water was such that a flow of approximately 170 mm was obtained by the standard flow test. Bed and head joints were 10 mm thick and fully filled.

Mortar strength was determined on prisms (40/40/160 mm) and on cubes (70/70/70 mm) at 28 days and at the time of testing shear walls. At the age of 28 days, mean compressive strength on prisms $f_{m,prism,28}$ was 1.0 MPa, while on cubes $f_{m,cube,28}$ reached 1.8 MPa. At the same time flexural strength of mortar $f_{x1,28}$, obtained on prism tests, was 0.7 MPa. More detailed results of strength tests are summarized in Table 1. The compressive strength of masonry f_c, determined in accordance with standard EN 1052-1:1998 on three samples with dimensions length/height/thickness = 0.79/1.19/0.19 cm was 3.8 MPa.

Table 1 - Compressive and flexural strength of mortar used for the test specimen.

Property	**Average age**	**Number of samples**	**Mean value**	**Coefficient of variation**
	[days]	[/]	[MPa]	[%]
$f_{m,cube,28}$	28	16	1.8	18
$f_{m,prism,28}$	28	30	1.0	24
$f_{m,cube,55}$	55	58	1.8	20
$f_{m,prism,55}$	55	110	1.2	24
$f_{x1,28}$	28	15	0.7	21
$f_{x1,55}$	55	55	0.8	21

2.2 Multi-storey shear walls

The test specimen is made of two 0.19 m thick three storey shear walls with openings, which are connected to cross walls at both ends to provide out-of-plane stability. Floor structures were 0.12 m thick r.c. slabs, which were built separately and then placed on the fresh mortar course on top of the walls by a crane. The specimen was built in the laboratory for structures at Slovenian national building and civil engineering institute.

Shear walls were 3.79 m long and had two openings on each floor. Above windows of height 1.20 m and length 0.91 m were r.c. lintels with a height of 0.20 m. The cross walls were 1.09 m long. The storey height was 2.01 m, so that the total height of the test specimen above foundations including r.c. slabs was 6.39 m. The masonry walls were built on a 30 cm thick r.c. slab, which had holes for fixing it to the laboratory floor. The dimensions of the test specimen in plan and elevation are shown in Fig. 1 and the entire test specimen can be seen in Fig. 3a.

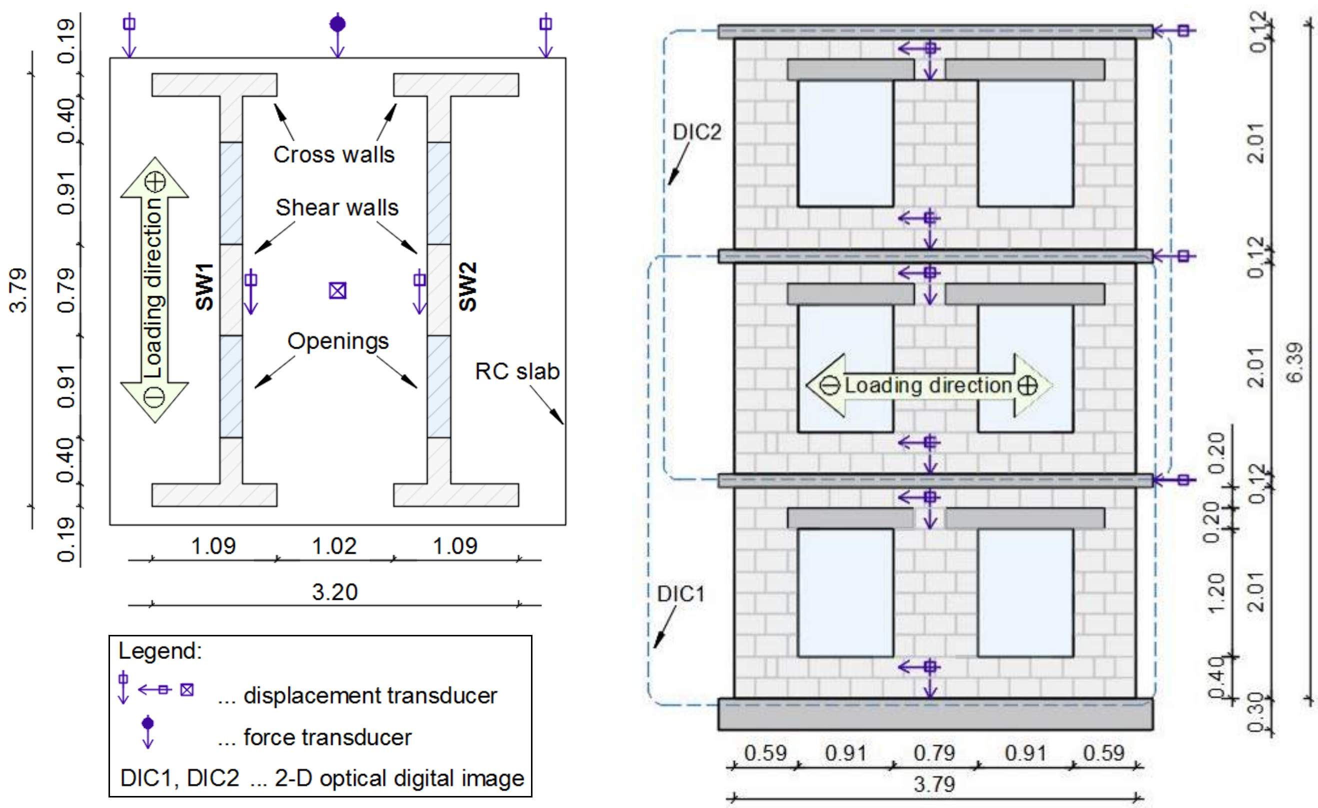

Fig. 1 - Disposition and instrumentation of the test specimen. Dimensions in meters.

2.3 Type of strengthening

To improve the in-plane lateral resistance of masonry walls, both shear and cross walls were strengthened. A system of glass fibre reinforced polymer (GFRP) grid laid in fibre reinforced cementitious mortar matrix and glass fibre anchors for connecting and fixing the layer of coating to the wall and to the r.c. floor slabs have been used. All of the materials are commercially available.

Bi-directional glass fibre grid was used in combination with cementitious mortar, which is designed specifically for using with glass fibres composites and has higher ductility and lower stiffness. Spacing of the strands in the grid was approximately 18/15 mm, and the glass fibre grid was coated by an alkali resistant coating. The tensile strength of dry fibre is 2.6 GPa. The ultimate load in longitudinal direction is 77 kN/m and in transverse direction 76 kN/m.

The matrix for the GFRP grid, which functions also as the adhesive for attaching the coating to the wall, is a fibre reinforced cementitious mortar. Its compressive strength, measured according to EN 998-2, is 27.1 MPa and modulus of elasticity by EN 13412, is 8 GPa. The mean values of the compressive strength of mortar, experimentally obtained on three cubes and three prisms at the age of 109 days, were 25.2 MPa and 12.0 MPa, while mean flexural strength of mortar prisms of the same age reached 7.6 MPa.

The anchors, used to link the structural and strengthening part of the masonry, were 10 mm diameter glass fibre rope. The tensile strength of the fibres is 2.5 GPa, while the modulus of elasticity is 70 GPa.

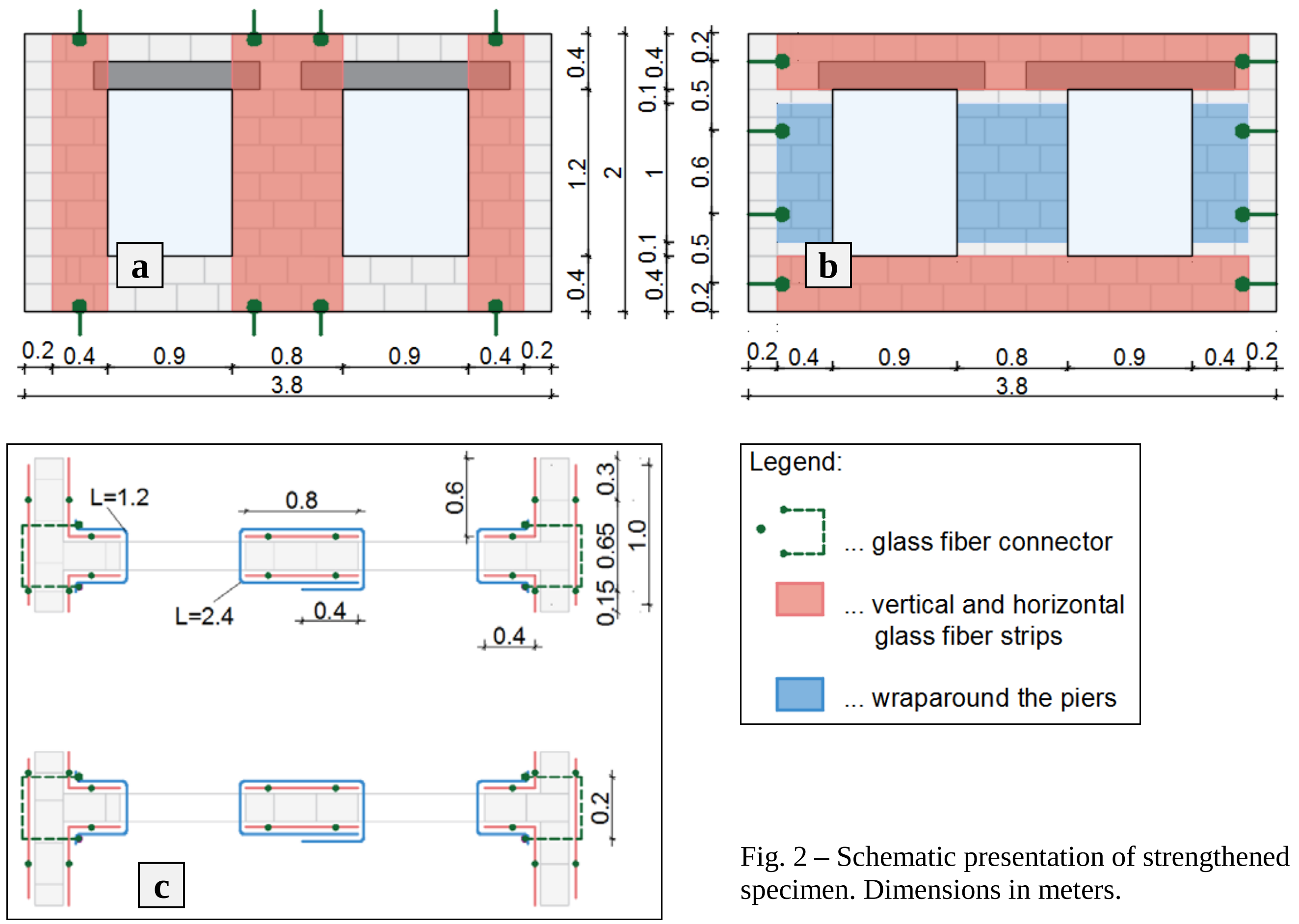

Fig. 2 – Schematic presentation of strengthened specimen. Dimensions in meters.

Strengthening of the specimen consisted of three strips of grid (Fig. 2a), placed over the entire storey height between the windows and at the sides. This was done on front and back of the walls, and the strips were anchored into the r.c. slabs above and below. The cross walls were likewise strengthened with grid over their entire surface. In the next step horizontal strips were placed above and below the windows over the entire length and on both sides of the shear walls (Fig. 2b). The middle pier was wrapped with the GFRP grid, whereas the strips on the walls on the sides were anchored into the cross walls as is shown in Fig. 2c. More detailed information on the positions and dimensions of the glass fibre strips and glass fibre connectors are shown in the Fig. 2c.

The specimen before strengthening is shown in Fig. 3a. The details of how anchors were installed and GFRP grid laid into mortar are shown in Figs. 3b-3f. The anchors were used in three configurations: (i) for fixing the coating into concrete foundation or r.c slab, (ii) for connecting coatings from two different floors, or (iii) for anchoring coating of walls, which end in a cross-wall. Configuration (i) was used to fix the coating of the first floor into the floor slab at the bottom of the specimen and to fix the coating of the top floor into the top r.c. slab. On one end, the anchors were glued into the r.c. floor by means of epoxy adhesive, whereas on the other end, the rope of the anchor was divided into multiple strands and fanned out into the coating. The length of the fanned out fibres was 50 cm (as shown in e.g. Fig. 3c). In the case of slabs between floors 1-2 and between floors 2-3, the anchors connected the coating between the floors in configuration (ii). In this case, the anchors run through the r.c. slabs and are again anchored into the coating by fanning out the fibres as explained before. In configuration (iii), the anchors completed the wrapping of a wall with a T cross-section. Here, the web of the cross section was wrapped (as shown in Fig. 2c).

Fig. 3 – **a**: test specimen before strengthening; **b**: placing of glass fibre anchors; **c**: spreading the fibres; **d**: application of vertical strips on cross and shear walls; **e**: placing the horizontal grid; **f**: wraparound of the middle pier.

The GFRP was laid into mortar using the following procedure: first the wall was moistened with water, and then covered by the first mortar layer with thickness of about 10 mm. The GFRP grid was then gently pressed into the mortar (Figs. 3d and 3e) and covered by another layer of mortar (5-10 mm). Where there were anchors, they were placed below (applied before) the GFRP mesh.

3. Testing procedure and instrumentation

The whole testing program was composed of three parts. In the first one, the cyclic shear test of the basic (unstrengthened) specimen was performed, while in the second part the testing of the strengthened specimen was done. Finally, after the second experiment was finished, only the top floor (which was practically undamaged) was tested under cyclic shear load.

At the beginning of each experimental test the vertical load, which simulated a typical stress state in the walls of a 5-storey URM residential building, was applied. Average compressive stresses of about 0.90 MPa in the walls of the bottom storey have been attained. Vertical load was applied by means of three hydraulic actuators placed on a steel load distribution system on the top floor slab. The actuators were connected to the laboratory strong floor by means of steel tie-rods. Resultant of the imposed vertical load amounted 1050 kN and remained constant throughout the tests.

After applying the axial load, lateral loads were applied by programmable hydraulic actuators acting on the specimen in the middle of the floor slabs. The actuators imposed the programmed seismic forces, distributed linearly along the height of the specimen, with the highest force at the top. The tests were controlled by imposing programmed displacements at the first floor level and measuring the resisting force. At the upper floors, the lateral load was imposed by multiplying the resisting force, measured at the first floor, according to the assumed triangular distribution of seismic forces along the height of the specimen. The displacements in the first storey were imposed in a cyclic manner. In the case of the third part of the testing program, that is when only the top storey was analysed, the seismic forces were (also cyclically) imposed on the top floor r.c. slab, while other two hydraulic actuators were placed in a fixed zero position.

At each amplitude level, the loading was repeated three times to allow for the strength and stiffness degradation. The amplitude was gradually increased and for the tests of strengthened specimen, displacement amplitudes and drift ratios are shown in Table 2.

During the tests the displacements of both shear walls were monitored by an optical digital image correlation (DIC) system with a measuring accuracy of 2/100 px (pixels) or about 0.04 mm. Two cameras with 5 megapixel resolution were used on each side of the specimen, and each of the cameras recorded the entire outer surface area of two storeys (the middle storey was recorded by both cameras). In order for the system to work, a random black speckle pattern was painted on the surface of the walls, as can be seen from Fig. 3a. In addition to the digital image correlation system, force transducers at each of the hydraulic actuators and a set of displacement transducers, placed at different locations, were used to measure forces and displacements, respectively. By means of displacement transducers, horizontal displacements at the sides and centre of each slab, as well as vertical compression of the walls and slip at the connection between the slabs and the walls have been measured (Fig. 1).

In order to visually inspect the damage occurred to the walls, the experiment was paused in each cycle of loading in positive and negative direction until this was considered unsafe for the inspection teams, which surveyed the building and mapped/photographed cracks and other damage.

The first experiment was terminated after the first storey drift attained 0.24 % of the storey height (displacement of 5.0 mm) in order that the damage remained repairable. After that the black/white speckle pattern was removed and the specimen was strengthened as described in section 2.3. After a period of time of 34 days during which the mortar cured, the testing of strengthened specimen was performed up to 1.39 % of the bottom floor storey drift. Shortly afterwards, the cyclic shear testing of just the top storey of the specimen has been performed. This experiment was terminated at drift ratio of 0.84 %, when huge damage occurred.

Table 2 - Displacement amplitudes and drift ratios for the testing of strengthened three storey specimen (left) and of the top storey (right).

Testing of the three storey shear walls		
Phase	**Displacement amplitude**	**Drift ratio**
	[mm]	[%]
1	*0.3*	*0.01*
2	*0.5*	*0.03*
3	*1.0*	*0.05*
4	*1.9*	*0.10*
5	*2.8*	*0.14*
6	*3.6*	*0.18*
7	*4.4*	*0.22*
8	*5.0*	*0.25*
9	*6.1*	*0.30*
10	*7.0*	*0.35*
11	*7.9*	*0.39*
12	*8.8*	*0.44*
13	*10.1*	*0.50*
14	*10.0*	*0.50*
15	*12.1*	*0.60*
16	*14.0*	*0.70*
17	*17.0*	*0.85*
18	*20.0*	*1.00*
19	*24.0*	*1.20*
20	*28.0*	*1.39*

Testing of the top storey shear walls		
Phase	**Displacement amplitude**	**Drift ratio**
	[mm]	[%]
1	*0.5*	*0.03*
2	*0.9*	*0.05*
3	*2.1*	*0.10*
4	*3.2*	*0.16*
5	*4.1*	*0.21*
6	*5.1*	*0.26*
7	*6.3*	*0.31*
8	*7.8*	*0.39*
9	*9.3*	*0.46*
10	*10.8*	*0.54*
11	*12.3*	*0.61*
12	*14.3*	*0.71*
13	*17.4*	*0.87*

4. Test results

4.1 Test results of the strengthened specimen

4.1.1 Damage propagation and failure mechanism

At about 0.10 % drift (1.9 mm displacement) the first visible cracks and damage occurred on the walls of the first floor. Tiny diagonal cracks in the piers were observed on the ground floor, while no damage of the masonry piers was detected on the floors above. Minor cracking developed also in the contacts between masonry and window r.c. down lintels and in the parapets of every floor. From this phase on, cracking of the walls was heard during loading. In the next loading stage, diagonal cracks were developed also in the middle pier of the second floor. At about 0.18 % drift, a crack developed at the contact between the r.c. slab above the first floor and the ground shear walls. The contact between shear walls and cross walls started to crack, and the cracks gradually grew over the entire height in the following phases. When displacement amplitude of 8.8 mm (drift of 0.44 %) was reached, cracking in the coating of the middle piers on floors 1 and 2 developed, which is consistent with placing of FRP grids. Overall cracking still resembled diagonal cracks (see Fig. 4). In the same phase cracks on the cross walls on the first and second storey were detected. The location of the vertical cracks coincided with the connection to the shear walls, while horizontal cracks appeared on many places along the height.

Damage of the walls increased with increasing amplitude of imposed displacement and typical diagonal shear crack pattern formed on all piers of the first and second floor (Fig. 4). Cracks occurred also in areas above and below the window openings. At the displacement amplitude of 10.0 mm, some new cracks occurred at the top of the first storey walls, where the strips were anchored into the r.c. slabs. In the next phase (drift of 0.60 %), the coating delaminated from the wall in the middle piers of the ground floor.

The test was terminated when the first storey drift amounted to 1.39 %. At this stage there were smeared diagonal cracks in the piers of the first floor. The extent of cracking of the second floor was significantly smaller. The middle piers cracked in a similar pattern as on the floor below, while the side piers remained uncracked throughout the entire test. There was no cracking on the top floor, other than minor local cracks at corners of window openings.

The results of digital image correlation (DIC) measurements, which enable information regarding the actual stress distribution in the shear walls as well as information on the boundary conditions of the central wall piers, have been evaluated. An image showing the major strain plot at the bottom storey drift of Φ = 0.60 % is shown in Fig 5. As the strain concentrations indicate the locations of cracks and the magnitude of strain indicates their width, the similarity between the observed crack patterns and DIC measurement plots can be clearly noticed.

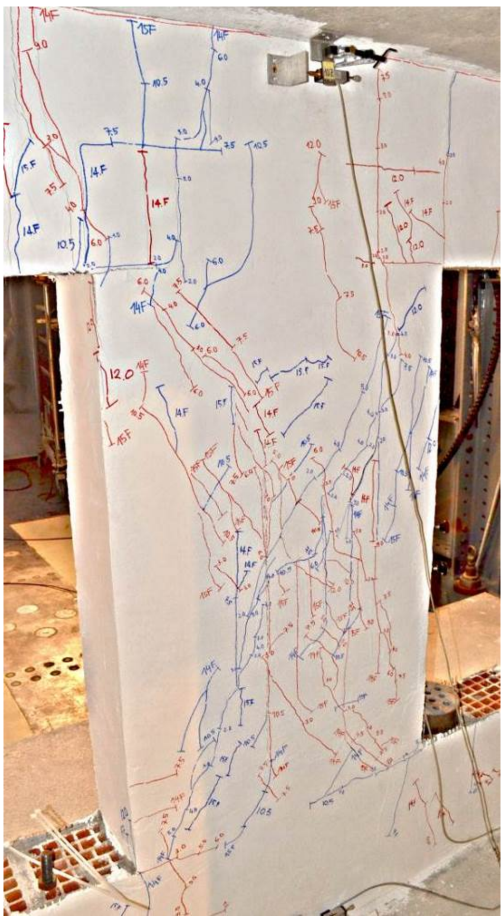

Fig. 4 - Crack propagation pattern of the middle pier on the ground floor at 0.60 % drift.

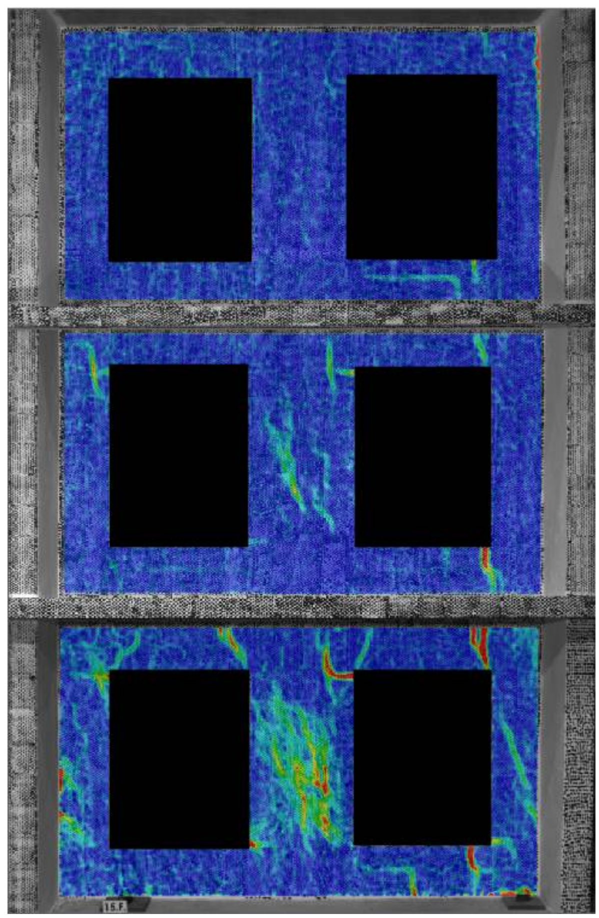

Fig. 5 - Major strain distribution of the multistorey shear wall at 0.60 % drift.

4.1.2 Lateral resistance and displacement capacity

The experimentally obtained lateral load (*H*) – drift (*Φ*) relationships are presented in Fig. 6, separately for each storey, including the hysteretic curves. Red lines are results of test of unstrengthened specimen, while the blue lines are the results of the strengthened one. The improvements due to strengthening can be easily recognized. The values of storey shear (H) and storey drift (*Φ*), obtained at characteristic limit states, are given in Table 3. To assess the efficiency of strengthening, the resistance and displacement capacity of the walls at two limit states (damage limit state and state at maximum resistance) have been compared.

Similarly, as in the experiment of unstrengthened specimen, the hysteretic curves show, that the majority of the damage and energy dissipation was concentrated on the bottom floor. There was some damage and dissipation of energy on the second floor, whereas the response of the top storey was essentially linear. The maximum lateral resistance, achieved in the test of the unstrengthened specimen was, in the case of the strengthened specimen, exceeded at damage limit state. The effect of strengthening is clearly seen in the improvement of the lateral resistance of the first storey by almost 50 %, which is seen in Fig. 6. At the same time stiffness of the specimen remained almost the same. The bond between the masonry walls and the strengthening layer was not a problem until the maximum seismic resistance of the specimen was reached. After this state, signs of delamination were observed and the crack propagation significantly increased. At next loading stages shear force gradually decreased. No brittle behaviour was detected, as confirmed by the hysteretic curves of the first storey.

The behaviour of the strengthened and unstrengthened specimens was similar. Both showed diagonal shear failure of the central piers on the first and (to a much lower extent) second floor.

Table 3 - Limit states. Abbreviation “UN” is used for unstrengthened and “STR” for strengthened specimen.

Storey		**Damage limit state**		**Maximum resistance**		**Ultimate state**	
		Φ [%]	H [kN]	*Φ* [%]	H [kN]	*Φ* [%]	H [kN]
3	**STR**	*0.05*	*151*	*0.12*	*206*	*0.11*	*151*
	UN	*0.06*	*134*	*0.08*	*142*	/	/
2	**STR**	*0.08*	*251*	*0.24*	*344*	*0.23*	*251*
	UN	*0.10*	*223*	*0.15*	*237*	/	/
1	**STR**	*0.10*	*300*	*0.60*	*412*	*1.39*	*301*
	UN	*0.10*	*268*	*0.20*	*285*	/	/

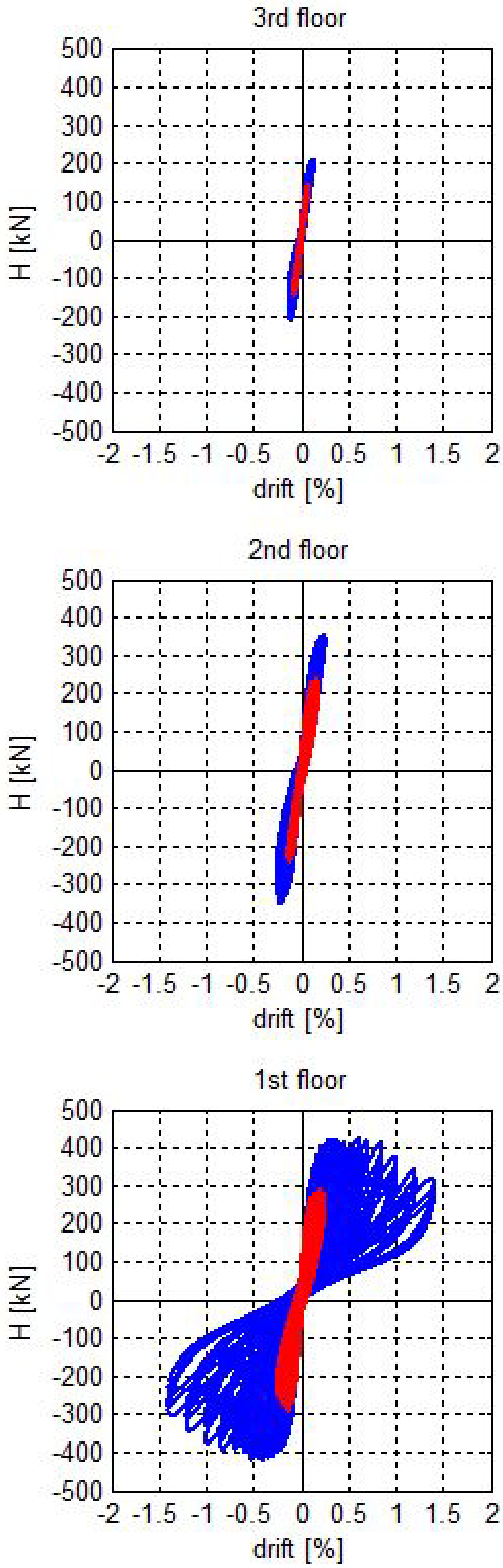

Fig. 6 - Hysteretic storey shear force – interstorey drift curves of individual floors of unstrengthened (red) and strengthened (blue) specimen.

4.2 Test results of the individual testing of the third floor

In the cyclic shear tests of the unstrengthened and strengthened multistorey specimen virtually no damage was observed on the top floor. This made it possible to test just the third floor and compare it to the first floor of the strengthened specimen. Note that the first floor was first damaged and then strengthened, whereas the third floor was strengthened in an undamaged state. Additionally, the middle pier of the third floor was (intentionally) not wrapped to see the effect of wrapping on ductility.

For the individual testing the hydraulic actuators on the first and second floor were locked into position to prevent any movement and serve as a laterally fixed support. The third floor responded in a clear shear mechanism with diagonal cracks.

4.2.1 Damage propagation and failure mechanism

The first new visible cracks, which occurred in the testing, were observed at about 0.10 % (displacement of 2.1 mm). Few tiny diagonal cracks on the side piers of the shear walls appeared. In the next phase (drift of 0.16 %) the middle piers started to crack in a typical diagonal shear pattern. When the storey drift amounted to 0.31 %, shorter horizontal cracks on the sides of the middle piers were detected and cracks in the contacts between masonry and window r.c. lintels developed. Areas above and below the window openings cracked in different directions. Vertical cracks in the contacts of the shear and cross walls and horizontal cracks on the top of the walls where connected to the r. c. slab, were observed. As the imposed displacements were increasing according to the loading protocol (Table 2), existing diagonal cracks continued to develop and open (Fig. 7).

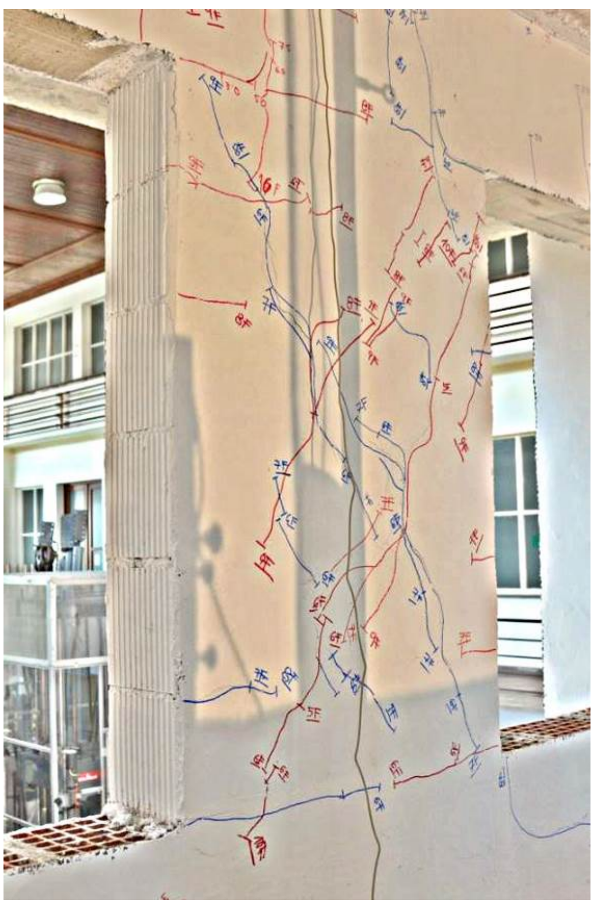

Fig. 7 - Crack propagation pattern of the middle pier on the ground floor at 0.39 % drift.

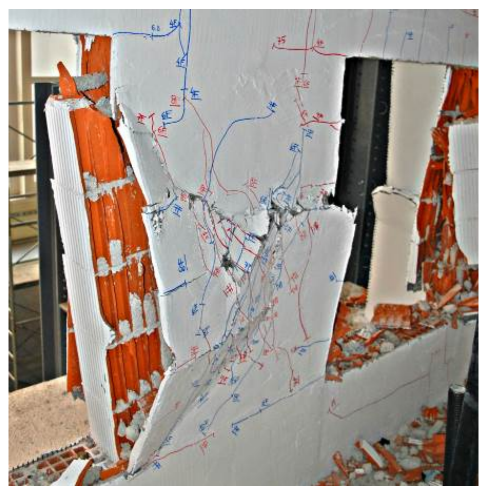

Fig. 8 – The middle pier at the end of the experiment.

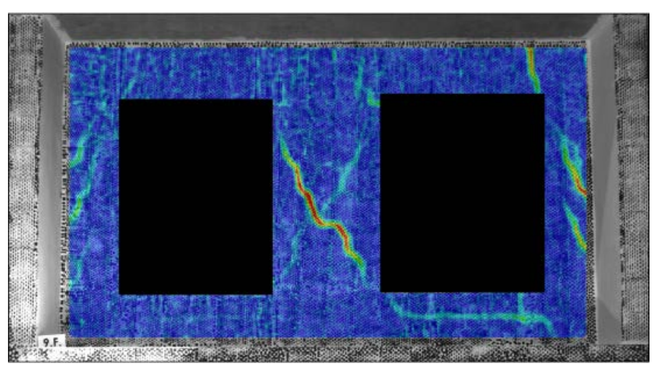

Fig. 9 - Major strain distribution of the third storey shear wall at 0.39 % drift.

At lateral displacement of 14.3 mm some individual blocks started to fall off the specimen piers. The coating delaminated from the wall in the middle and side piers. In the last phase (0.87 % drift), the walls failed in compression (Fig. 8). The vertical strips on the wall piers continued to delaminate from the wall and the glass fibre mesh ruptured.

Observed behaviour coincides with the DIC measurements, as shown in Fig. 9, where major strain plot at the storey drift of 0.39 % is presented.

4.2.2 Lateral resistance and displacement capacity

The experimentally obtained envelopes of the hysteresis curves, showing relationship between the storey shear force (*H*) and storey drift (*Φ*) for third floor are plotted in Fig. 10 and the values of these two parameters at characteristic limit states are given in Table 4.

By imposing the displacements up to the 8th phase, at which maximum resistance was reached, the values of the lateral forces were gradually increasing. The maximum storey shear at this limit state was 501 kN. Severe degradation of lateral resistance of the strengthened wall took place at repeated lateral in-plane load reversals in the last loading phases. This was consistent with the appearance of the delamination of the coating layers. In the last loading stage (at drift of 0.87 %) the lateral force significantly dropped and amounted to only half of the maximum resistance.

Values of the storey shear force and drift at characteristic limit states of the single storey specimen in comparison with values of these two parameters of first storey in the case of whole strengthened specimen were quite similar, as shown in Fig. 10. The main difference in the results is observed for the maximum shear force, which is about 20 % higher while testing a single storey and it appears at a lower storey drift. The reason for higher strength of the third floor compared to the first floor of the strengthened specimen is probably that the first floor was significantly damaged before strengthening was applied. It is also evident from the mode of the collapse and from the increased ductility in case of wrapped walls that wrapping crucially improves behaviour of structures. More concentrated cracking (not smeared as on the first floor) and ultimately a much more brittle failure was observed in the case without wrapping.

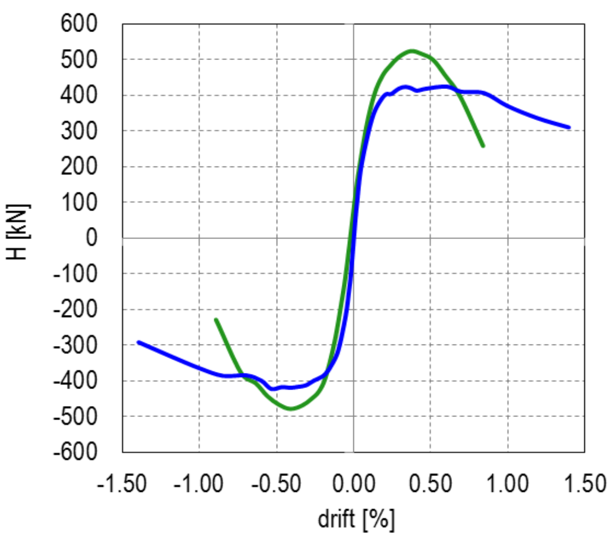

Fig. 10 - Hysteresis envelopes of first floor of the strengthened specimen (blue) and of the individual testing of third floor (green).

Table 4 - Limit states for the first floor of the strengthened specimen and for the individually tested third floor.

Storey	Damage limit state		Maximum resistance		Ultimate state	
	Φ [%]	*H* [kN]	*Φ* [%]	*H* [kN]	*Φ* [%]	*H* [kN]
3$_{\text{individual}}$	*0.10*	*303*	*0.39*	*501*	*0.87*	*243*
1	*0.10*	*300*	*0.60*	*412*	*1.39*	*301*

5. Conclusions

The efficiency of seismic strengthening with FRPs of URM masonry built of hollow clay masonry units has been experimentally investigated on a three-storey full scale shear walls under cyclic shear load. A system of glass fibre reinforced polymer grid in combination with fibre reinforced mortar matrix and glass fibre anchors has been used. The system of strengthening consisted of vertical and horizontal strips, placed over the entire specimen in combination with wrapping of piers. First, the sample was tested in unstrengthened state up to significant damage, then strengthened and re-tested to the collapse.

Observations and results of digital image correlation measurements indicate predominantly shear response in unstrengthened and strengthened state. The tests have confirmed the efficiency of the FRP materials as an alternative to classical materials for strengthening and shown the importance of wrapping for achieving a ductile response. Additionally, the lateral resistance of masonry walls was significantly improved (by almost 50 %).

Subsequent cyclic shear test of the top floor has shown behaviour similar to the multistorey strengthened specimen. As the piers of the top floor were not wrapped, the damage propagation was faster, ductility was lower and collapse brittle. This indicates that much attention needs to be paid to designing strengthening in a way which prevents delamination between the composite coating and masonry.

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7. References

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