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CYCLIC SHEAR TESTS ON URM AND STRENGTHENED MASONRY WALLS AND ITS MODELLING

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ABSTRACT

In this paper some experimental and numerical results pertaining to unreinforced masonry walls and its components are presented. This is the first phase of the joint project to be implemented by the Faculty of Civil Engineering, University of Sarajevo and the Institute for Lightweight Structures and Conceptual Design, University of Stuttgart. Testing methods for solid clay brick, lime-cement mortar, wallet compressive and shear strength and elastic modulus follow national standards and European norms. Full scale tests of the unreinforced masonry walls were conducted at the Institute for Materials and Structures, Faculty of Civil Engineering in Sarajevo. Numerical modelling concerns prism compression test and full scale masonry wall exposed to vertical and horizontal forces. Snap-back instabilities due to brick-mortar mechanical and geometrical mismatch are tackled as well. In the second phase it is planned to apply several strengthening methods and to compare the wall behaviour with the unreinforced one. The main goal of the research project is to investigate the influence of the different strengthening methods on the structural behaviour of originally unconfined masonry walls under cyclic horizontal loading.

1. INTRODUCTION

Structural assessment of existing buildings is an important task for civil engineers especially in densely populated urban areas and older cities. The existing buildings in Bosnia and Herzegovina are traditionally built as masonry. Depending on the historical period and the art of building in specific regions, brick or stone masonry was applied (Hrasnica, 2009). Despite the fact that reinforced concrete structures prevail in the newly erected buildings, masonry structures are still built, with application of new construction materials. The analysis results of existing masonry structures lead very often to the conclusion that some art of rehabilitation is necessary (Ademović, 2012).

Buildings were traditionally built as unreinforced masonry (URM) with wooden floors. The buildings erected after World War II generally have reinforced concrete floors. Both groups of buildings are relatively stiff and show generally limited ductile behaviour. Regarding seismic vulnerability classification (EMS) they belong to the classes B and C respectively (Hrasnica, 2008, Hrasnica, 2012). It means that already for moderate earthquake intensities some important damages could occur. In the case of stronger earthquake motions heavy and very heavy damages, including

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partial collapse could be expected. These statements were unfortunately proven by strong earthquakes in the Western Balkan Region in the last 50 years; Skopje 1963, Banjaluka 1969, Montenegro Coast 1979 (Hrasnica and Medic, 2012).

Considering these facts, we conclude that it is of great interest to experimentally assess the seismic behaviour of ordinary URM walls. Large stock of these buildings exists in Bosnia and Herzegovina and its retrofit represents a high priority. Adequate seismic evaluation and strengthening can only be done with reliable input data, hence experimental testing needs to be conducted.

The first phase of the project to be implemented at the Faculty of Civil Engineering in Sarajevo envisages material tests of masonry components. Also, it will include construction of URM walls made of solid clay bricks and lime-cement mortar. Two full-scale wall models (see Fig. 10 & Fig. 12) have been prepared. It is important to state that the walls have no vertical confinement, which became typical way of construction in the Western Balkan Region after Skopje Earthquake in 1963. The full-scale wall models without vertical confinement are tested under constant vertical load and cyclic horizontal loads.

In the second phase of the project it is planned to apply several strengthening methods and to compare the wall behaviour with the unreinforced one. The main goal of the research project is to investigate the influence of the different strengthening methods on the structural behaviour of masonry walls under cyclic horizontal loading. Reinforced concrete coating will be applied having different types of reinforcement meshes, and then the strengthening with FRP and plastic meshes will be analyzed. The third phase of the project consists of numerical verification of experimental results which will be used to suggest practical guidelines for analysis of existing URM structures and implementation of strengthening procedures.

In the following sections, the test results of masonry components which were performed according to national and European standards as well as the results of numerical analysis will be presented.

2. COMPONENT PROPERTIES

2.1. Brick unit

Compression tests on specimens 250/120/65 mm (length/width/height) were done according to the national standards and European Norms. This type of brick was typically used in masonry structures erected in the second half of the 20th century during massive reconstruction after World War II. Firstly, testing of brick compressive strength according to national standards was performed. The particularity of this code is the fact that compression tests are carried out on a series of 5 brick sandwich specimens made of 2 bricks having a thin layer of cement mortar in the middle, as shown in Fig. 1a. The obtained mean value of the compressive strength was 29.9 N/mm^2. On the basis of these regulations the brick can be classified as M20, having a characteristic compressive strength of 20 N/mm^2.

Compressive strength of solid bricks was additionally tested according to the EN 772-1:2011 which comprised 4 series of 6 bricks prepared by grinding (the mean value of compressive strength was 53.9 N/mm^2, Fig.1b) and 2 series of 6 bricks capped by mortar (the mean value was 47.1 N/mm^2, Fig. 1c).

(a) (b) (c)

Figure 1. Compression tests: (a) national standard (b) EN – grinding (c) EN - capping mortar

The tensile strength of the considered brick units was determined by two methods. The three point bending test was performed on 9 samples (Fig. 2a) and the tensile strength was at average 5.2 N/mm^2. The coefficient of variation (c.o.v.) is very high and equals 40%. Then, the Brazilian splitting test (Fig. 2b) was executed on 9 cylindrical samples (base/height = 54 mm/50 mm), and following the "Suggested methods for determining tensile strength of rock materials" (IJRMM, 1978), the obtained average tensile strength equals to 3.75 N/mm^2 with the c.o.v. of 14 % (courtesy of GEOlab Sarajevo). Visual inspection confirmed very irregular fracture surfaces pertaining to 3 point bending test as opposed to indirect tensile test which resulted with quite straight splitting area.

(a) (b)

Figure 2. Determination of tensile strength by (a) 3 point bending test (b) splitting test

2.2 Mortar properties

Handmade lime-cement mortar samples were tested with the following ratio of the ingredients: lime: cement: sand=1:0.5:4 (grades by volume), in order to obtain the compressive strength for mortar of roughly 2.5 N/mm^2, typical for most of the existing masonry buildings. The tests were done according to EN 1015-11 as shown in Fig. 3. The specimens with dimensions 40x40x160 mm were cured for 28 days. The compressive strength amounts 2.3 N/mm^2 with the c.o.v. equal to 14%. The average tensile strength is 1.3 N/mm^2 and the c.o.v. reaches 20%.

(a) (b)

Figure 3. (a) tensile strength of mortar by 3PB (b) mortar compressive strength

2.3 Interface properties

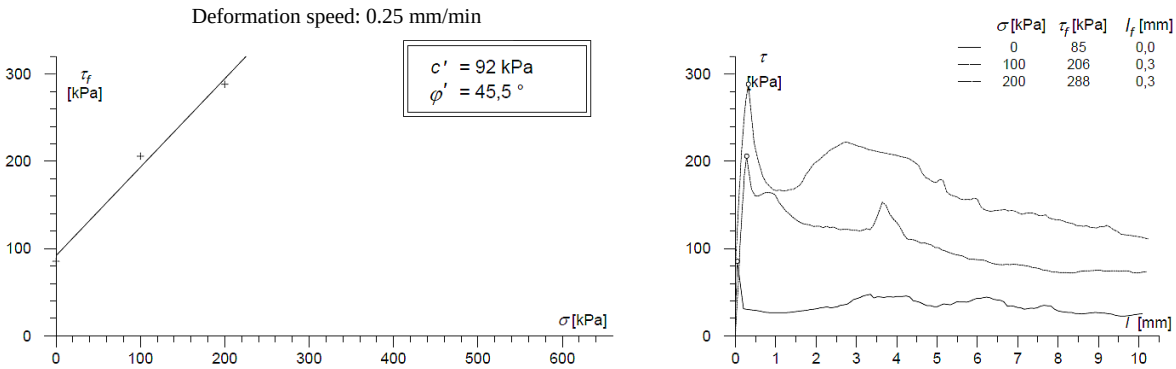
The behaviour of masonry walls exposed to horizontal loads strongly depends on the properties of the brick – mortar interface. The shear failure can be modelled with the classical Coulomb's law for pressure dependent materials which employs the cohesion and the angle of internal friction. The ultimate shear strength is determined for different values of normal stress. Two tests were conducted: the direct shear test in the typical geotechnical laboratory apparatus (Fig 4a & b, courtesy of GEOlab Sarajevo) and the standard triplet test with respect to EN 1052-3:2001 (Fig. 4c). The sample for the former experiment is cylindrical with diameter 100 mm and consists of 30 mm thick brick base and 10 mm thick mortar layer. At the time of writing this article, the direct shear test with the mortar aged 7 days was performed.

The measured values are shown in Fig. 5a, where the cohesion amounts c = 92 kPa and the angle of internal friction is φ = 45°. The sample is sheared along the interface imposing constant displacement rate of 0.25 mm/min, a method which additionally provides the softening branch in the

stress – displacement diagram (Fig. 5b). The triplet tests were implemented for four levels of precompression stress: 0.0, 0.2, 0.6 and 1.0 N/mm^2, and the mortar age at the time of testing was 28 days. The obtained results matched well with the direct shear test (c = 150 kPa, φ = 37°) and the dilatancy angle is negligible. All samples fractured along the joint. The mechanical behaviour of the brick and mortar construction relies on the contrast among the two constituents and the interface bond properties connecting high strength brittle brick to the low strength and quite ductile mortar joint. Thereby, the interface transition zone is usually the weakest element of the composite.

(a) (b) (c)

Figure 4. Determination of interface properties (a) (b) direct shear apparatus (c) triplet test



(a) (b)

Figure 5. (a) Mohr-Coulomb failure envelope for interface (b) shear stress – displacement diagram

2.4 Wallet compressive strength and the modulus of elasticity

In order to have the material properties for the numerical analysis as realistic as possible, masonry compressive strength was tested on six single-wythe thick wallets with dimensions 51.4x37.5x12 cm build from solid clay bricks and lime-cement mortar of the same characteristics as stated above in 2.1 and 2.2 (joint thickness 1.4 cm). The vertical joints were completely filled with mortar. Lead sheets were installed on the top and at the bottom of the wallets to eliminate contact surface irregularities. Wallets were tested in axial compression according EN 1052-1:1998. Load application was done with the rate 0.20 N/mm^2/min and the vertical displacements were measured using four LVDTs with measuring base of approximately 15 cm. The modulus of elasticity was measured at the load level amounting 1/3 of the ultimate compressive strength. The sample of the wallet and its failure is shown on Fig. 6. Mean compressive strength of masonry wallets amounts to 3.92 N/mm^2, while f_k=f/1.2 = 3.27 N/mm^2. The measured values of the modulus of elasticity average 2500 N/mm^2 with the variation coefficient of approx. 20%.

(a) (b) (c)

Figure 6.(a)(b)Experimental test set-up (c) failure mechanisms of wallets

Determination of the characteristic compressive strength of masonry for clay bricks of Group 1, as per EN 1996-1-1:2005, in absence of experimental data, can be calculated according to the equation (1):

$$f_k = K\,(f_b)^{0.65}\,(f_m)^{0.25} \tag{1}$$

With K=0.6 and the rest of the previously defined parameters, we obtain an average value of f_k = 8.7 N/mm^2. The large difference could be caused by the fact that the used bricks were not soaked in water before construction of the wall and the mortar lost necessary water, which resulted in lower prism strengths. For this group of wallets, we purposely cured the specimens in an inappropriate way in order to simulate practical construction site conditions.

Remark: Generally, equation (1) leads to the compressive strength of the masonry lying between brick and mortar strength, which corresponds to experimental results. But, in the case of higher mortar strengths, the compressive strength of the wall determined using the previous formula (1) could be lower than the brick and mortar strengths separately.

3. NUMERICAL MODEL OF THE WALLET

The prism test or the wallet test according to EN 1052-1:1998, which involves a stack of bricks and mortar layers subjected to axial far-field compression, usually serves for verification and validation of numerical models. It is also the standard experiment for quality assurance of brick and mortar construction. The outcome of the experiment is rather intriguing at first sight, when the compressive strength values of single bricks and mortar layers are compared with the strength of the composite masonry prism (see Fig 8a). The failure mode can be characterized as multiple axial splitting (see Fig. 8b), both in-plane as well as out-of plane. It is not a simple task to realistically model the behaviour of the wall by assembling the brittle brick units with a network of ductile mortar joint. The incompatibility of the constituents can lead to snap-back instabilities.

Even in the 1D case, it can be shown for linear elastic brick and linear-softening mortar that a critical combination of elastic modules can lead to instabilities (see Fig. 7). For this simple case of displacement controlled load scenario the equilibrium equations can be written as in (2):

$$\begin{bmatrix} k_b + k_m & -k_m \\ -k_m & k_b + k_m \end{bmatrix} \begin{bmatrix} d_2 \\ d_3 \end{bmatrix} = \lambda \begin{bmatrix} k_b & 0 \\ 0 & k_b \end{bmatrix} \begin{bmatrix} d_4 \\ 0 \end{bmatrix} = \begin{bmatrix} \lambda F \\ 0 \end{bmatrix} \tag{2}$$

Thus, the singularity condition $k_b+k_m = 0$ (generally k = EA/L) yields the critical incremental softening modulus of mortar for which the prism turns perfectly brittle (3):

$$E_m^{crit} = -L_m/L_b \cdot E_b \tag{3}$$

The geometric size effect of the mortar and brick units enters the expression in the form of thickness ratio L_m/L_b, assuming that the areas of brick and mortar are the same and without furrows. Implementing the modified Riks's arc-length method (Riks, 1979) in MATLAB, one can obtain typical load – displacement diagrams (λ – load factor, d(4) – displacement of the control node located on the top brick) such as the one shown in Fig. 7.

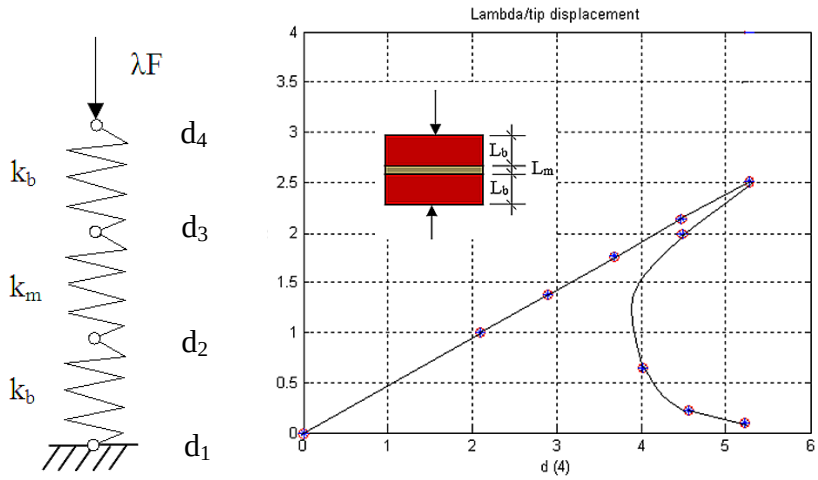


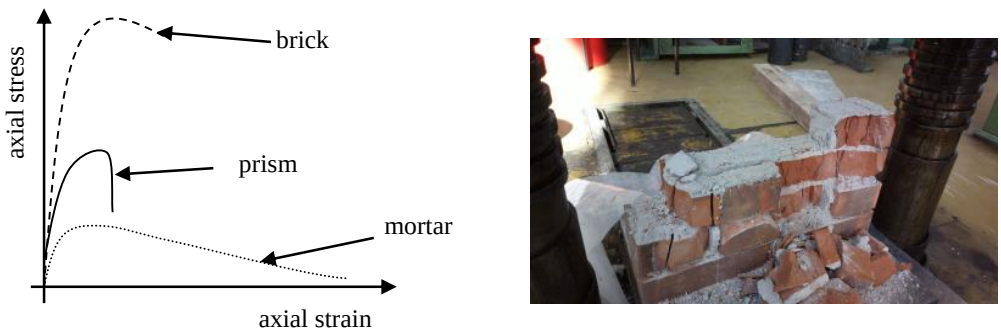
Figure 7.Typical snap – back instability for strong brick and weak mortar joint, 1D case.

At the critical softening modulus of mortar, the masonry prism assembly exhibits perfectly brittle response inferring that the elastic strain energy of the elastic brick is entirely dissipated by the softening mortar joint. Linear elastic analysis in 2D reveals that the lateral tensile brick stress increases in proportion to the level of axial compression depending on the elastic mismatch of the elastic moduli and the cross effects of Poisson in the brick and mortar constituents as in (4) (Willam and Ayoub, 2010):

$$\dot{\sigma}^{b}_{lat} = \frac{\nu^{b}E^{m} - \nu^{m}E^{b}}{E^{m}\left(1-\nu^{b}\right) + L^{b}/L^{m} \times E^{b}\left(1-\nu^{m}\right)}\dot{\sigma}^{b}_{axial} \tag{4}$$

The lateral cross effect diminishes to zero when the elastic mismatch of the bimaterial system vanishes,$\nu^b/E^b \rightarrow \nu^m/E^m$. Realistic values for the more compliant mortar vs. the stiffer brick units lead to lateral tension in the brick under far-field compression. Simply, the mortar layer is attempting to squeeze out laterally introducing lateral tension in the brick unit in exchange for lateral confinement of the mortar layer. On the other hand, the mortar layer is subjected not only to axial but also to lateral compression whereby the triaxial confinement significantly increases the mortar strength due to internal friction. Consequently, it is not the weak mortar that fails in compression, but the brittle brick which fails in bilateral tension.

If we consider the prism test (Fig. 8), we note that the prism strength is generally two – or three- fold the mortar strength contrary to the weakest link concept of elastic brick units in series with degrading mortar layers. The overall ductility of masonry prism is drastically reduced compared to the post-peak performance of the uniform mortar and brick units tested on their own. This reveals the formation of snap-back instabilities due to energy release in the brittle brick units when the weakest mortar joint starts to soften. This elastic rebound results in the brittle post-peak response of the prism experiment as seen in Fig.8(a), (Willam and Ayoub, 2010).

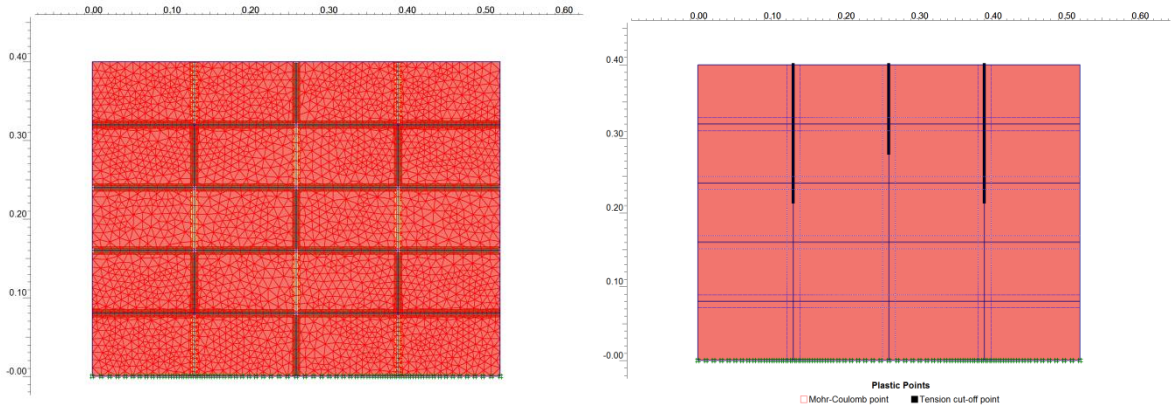


(a) (b)

Figure 8.Prism test on masonry in axial compression (a) general stress-strain diagrams (b) failure mode

The geometry of the wall prism (wallet) was previously defined in 2.4. The heterogeneous wallet is modeled at the mesoscale, consisting of brick elements and two types of zero-thickness interfaces (Gambarotta and Lagomarsino, 1996, Giambacano et al., 2001, Lotfi and Shing, 1994, Lourenco and Rots, 1997). The existence of mortar is neglected, however the brick-mortar interface is taken into account. The behaviour of brick-mortar interface is characterized by perfectly plastic Mohr-Coulomb material model (the elastic slip is negligible), whereby the cohesion c = 150 kN/m^2 and φ = 37° are taken from our experimental results. The tensile strength is equal to zero and there is no compression cap. The second interface type pertains to brick-brick contact and in the model it divides each brick. An elastic limit condition for the brick interface is based on the von Mises criterion, for which the cohesion c = 1200 kN/m^2 (φ = 0°) is assumed. The brick itself is linear elastic with E = 2500 N/mm^2 and ν = 0.20. Overlapping of the interface is negligible, however the gapping exists. The composite FE model with total number of 5078 elements is shown in Fig. 9.

From the analysis results it can be seen that the wallet fails due to lateral tension which is in accordance with the experimental results. However, the discontinuity planes (weak interfaces in the brick) are imposed by the modeller which dictates the crack disposition. These models shall be further refined in 3D using fracture and damage criteria for brick as a continuum where it is expected to obtain multi-axial splitting as experimentally observed.



(a) (b)

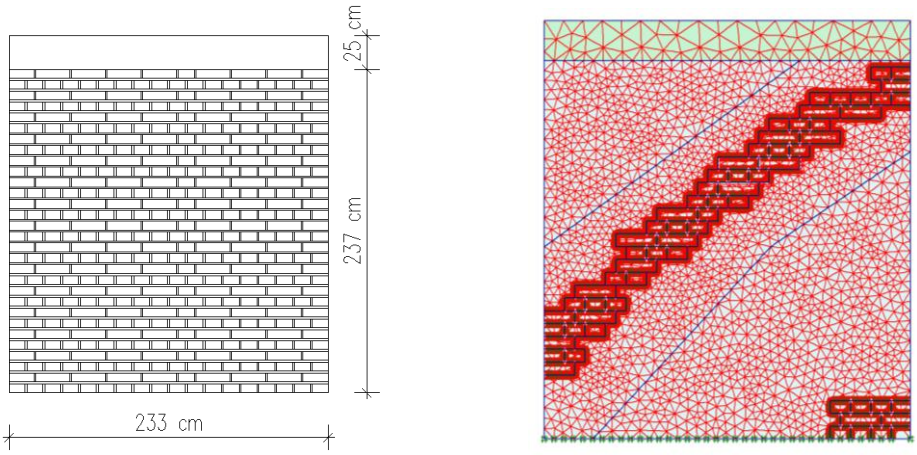
Figure 9. (a) Finite element composite model (b) failure model of the wallet.

4. NUMERICAL MODEL OF THE FULL SCALE WALL

The full – scale wall with dimensions 233/237/25 cm (length/height/thickness) is depicted in the Fig.10. There is a reinforced concrete tie-beam 233/25/25 cm on the top of the structure. The same brick and mortar previously described were used for erecting and modelling the wall. This type of unreinforced masonry wall is typical for B&H masonry buildings, erected after the World War II.

The cyclic tests have not been completed by the time of writing this report, however the push – over analysis of the wall as a vertical cantilever was executed and some preliminary results are discussed here. The boundary conditions are assumed restrained at the bottom (vertically and horizontally fixed). The level of vertical stress (preloading ratio σ/f_k) varies between 0.1 – 0.3 and it is kept constant during the analysis. The horizontal loading applied at the RC beam is increased until the stiffness matrix turns singular.

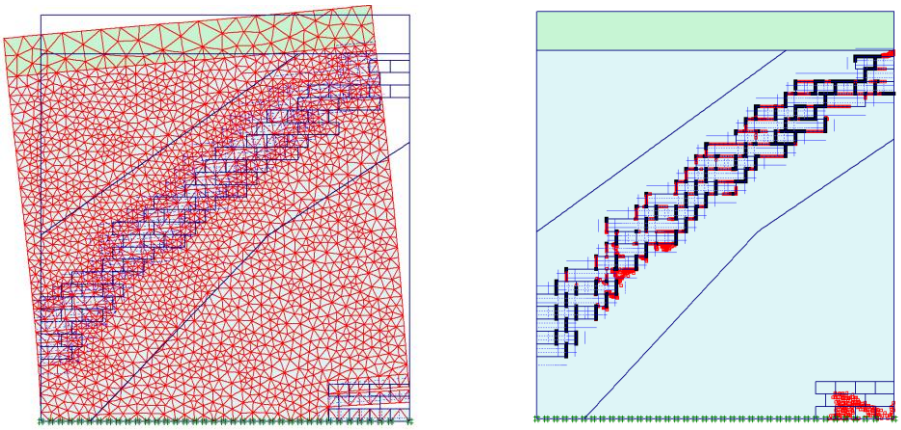
The elements used are triangular with 15 nodes and 12 Gauss integration points. The total number of elements is 4530 (see Fig. 10b). The wall was modelled similarly to the prism test, however, the brick-brick interface was neglected. The same parameters for brick – mortar transition zone were kept, and the wall itself is considered as Mohr-Coulomb material with E=2500 N/mm^2, ν = 0.20, c = 0.8 N/mm^2 and φ = 37° adding the tensile strength equal to 1.3 N/mm^2 (no compression cap). The dispersion of material properties that can be found in literature is very high (Berto et al., 2004, Burnett et al., 2007, Chaimoon and Attard, 2006, Gabor et al., 2006, Magenes and Calvi, 1997).



(a) (b)

Figure 10. (a) Wall geometry (t=25 cm) (b) FE mesh

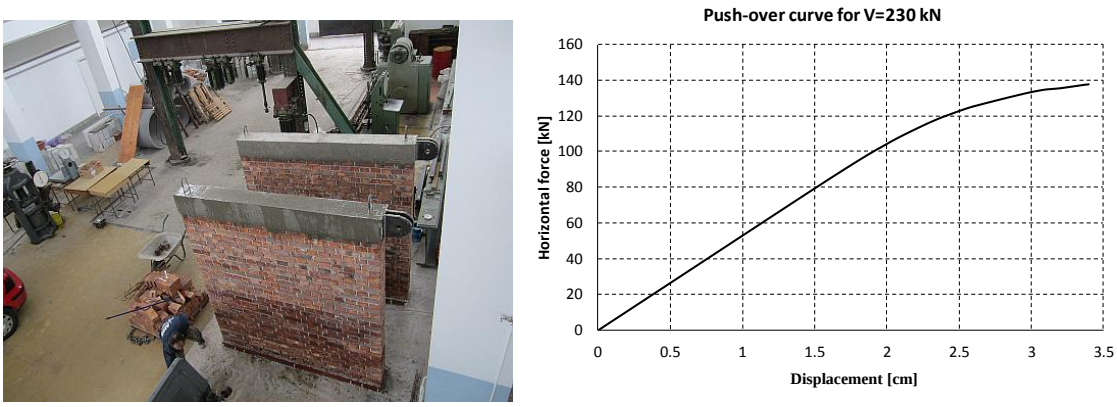
More adequate models will be made after finishing the planned experimental work at the Faculty of Civil Engineering in Sarajevo (see Fig. 12a). Depending on the ratio of horizontal and vertical loading, the wall fails either by rocking or by shear or combined. As it can be seen from Fig. 11, the wall has typical diagonal cracks in the form of steps and the gap is opened at the tensioned side. It can be noted that the crack trajectory is mesh dependent due to fixed interface location. On the other hand, it is known that the wall breaks along the weak mortar – brick transition zone.



(a) (b)

Figure 11. (a) deformed mesh (b) tensile splitting and sliding

In the case of vertical load equal to 230 kN, the push-over curve with the maximum horizontal force equal to cca. 130 kN and the peak displacement of cca. 3.5 cm is shown in Fig. 12b.



(a) (b)

Figure 12. (a) Full scale walls in the lab (b) push-over curve for V=230 kN

The obtained horizontal load bearing capacity can be compared with the expression (5) given in EC 6:

$$
\begin{aligned}
&V_{Rk} = f_{vk} \cdot L_c \cdot t \\
&V_{Rk} \text{ - characteristic shear resistance of the wall} \\
&f_{vk} \text{ - characteristic shear strength of the wall} \\
&L_c \text{ - length of the cross - sections's compressive zone after exclusion of tension} \\
&t \text{ - wall thickness} \\
&f_{vk} = f_{vk0} + 0.4\sigma_k \\
&f_{vk0} \text{ - initial shear strength} \\
&\sigma_k \text{ - average compressive stress, } \sigma = N_k/(t \cdot L_c)
\end{aligned}
\tag{5}
$$

The value for $V_{Rk} \approx 120$ kN was obtained which is a good match compared to the numerical model, but it should be emphasized that additional investigations both numerical and experimental are still to come.

5. CONCLUSION

A series of laboratory tests were performed in order to evaluate the mechanical properties of the masonry and its components. The characteristics of solid brick units and mortar as well as the masonry wall are susceptible to high variations. Building procedure and skills of the workforce can have substantial effect on the quality of masonry assembly.

Numerical models comprise simple 1D models where the incompatibility between brittle brick and softening mortar and 2D mesoscale models of the prism test and the shear wall has been investigated. The system composed of two materials exhibits sharp reduction of load bearing capacity when exposed to compression, although the components itself showed considerable ductility in displacement controlled tests. One of the major hindrances to ductile behaviour of the prism is the snap-back instability issue, due to energy release in the brittle brick units when the weakest mortar joint starts to soften. This elastic rebound results in the brittle post-peak response of the prism experiment.

Masonry has been modelled using 2D representation by means of triangular finite elements and zero-thickness interfaces along the possible fracture planes. The inelastic failure surface for the mortar interface was modelled by employing a Mohr-Coulomb failure criterion with tension cut-off and without compression cap. The interface transition zone is usually the weakest element of the composite. For the model of the wallet, the interfaces inside the brick units have been used as well, trying to simulate the multiaxial splitting pattern of the prism compression test.

The cyclic shear test of the full scale wall hasn’t been completed by the time of writing this report. It is intended to calibrate the model after performing the experimental work. A nonlinear pushover analysis was executed for different values of vertical loads, and typical load-displacement diagram is shown. It is noted that the results are mesh dependent due to fixed interface location. On the other hand, it is known that the wall breaks along the weak mortar – brick transition zone.

A lot of existing masonry buildings have load-bearing walls mostly in one direction, hence, they are rather vulnerable to stronger seismic actions. The resistance is usually lower with respect to shear force which imposes strengthening of the structure. It is planned to investigate different strengthening techniques for masonry wall upgrading, which includes FRP, reinforcing steel and plastic meshes. Specific problems arise when confronted with modelling of existing buildings, mostly due to lack of technical documentation and the unknown mechanical properties of the used materials. Therefore, seismic analysis and assessment of safety (performance – based earthquake design) of masonry buildings patrimony represents very complicated, but also interesting and challenging task for structural engineers.

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