EXPERIMENTAL TESTING OF SOLID BRICK MASONRY WALLS

Mustafa HRASNICA¹, Fadil BIBERKIC², Senad MEDIC³

ABSTRACT

In Bosnia and Herzegovina, many multi-story residential buildings built after the World War II have unreinforced unconfined masonry load-bearing walls which were proven vulnerable during strong earthquakes in the recent past. In order to study the seismic resistance of such walls, in-plane tests were carried out at the laboratory of the Faculty of Civil Engineering in Sarajevo. Four full-scale (233x241x25cm) and nine reduced-size specimens (100x100x25cm) made of solid clay brick and lime-cement mortar were subjected to cyclic shear and monotonic pushover loading program under constant vertical pressure. One-sided or two-sided reinforced concrete or mortar jacketing was applied to improve lateral resistance and displacement capacity. One way of strengthening was with the orthogonal position of reinforcement mesh Q196 and “new” type of connectors made of shaped Ø5 reinforcing bars. The connectors were placed vertically (9 pieces/m²) and horizontally (4 pieces/m²) in joints and grouted with high strength quick-hardening mortar. In the second type of strengthening, the mesh Q196 was inclined to 45° (135°) in order to follow the principal stress trajectories. Plain walls fail in shear with a typical cross-diagonal crack pattern. Jacketed walls exhibit rocking and significantly larger ductility compared to plain walls. Wallets were tested for compressive strength and elastic modulus of masonry and the results show significant variations.

Keywords: masonry walls; jacketing; experimental testing

1. INTRODUCTION

Seismic behavior of unstrengthened masonry is scientifically relevant topic in the western Balkan region. Roughly estimated, so much as 80% of building stock is unreinforced masonry, including historical structures. The majority of multi-story residential buildings erected in Bosnia and Herzegovina in the years following World War II were masonry buildings with 4-6 floors. The country is situated in a seismically active region of South-East Europe and it is divided into seismic zones with peak ground acceleration (PGA) ranging from 0.1–0.2 g for 475 years return period (zone VII-VIII according to MSK-64 or EMS-98 scale) in the most parts of the country to PGA of 0.30–0.35 g (approx. zone IX according to the seismic intensity scales) in some parts (Figure 1a). The traditional art of construction was unreinforced masonry (URM) with wooden floors (later prefabricated R.C. floors) and without vertical R.C. confining elements. Seismic resistance was provided by structural walls laid in two mutually orthogonal directions. However, a smaller number of walls in the longitudinal direction was imposed by functional demands. With respect to seismic vulnerability classification (EMS), masonry structures belong to classes B and C (Hrasnica 2009) which means that heavy and very heavy damages, including partial collapse, could occur in the case of stronger earthquake motions (Figure 1b shows typical damage after an earthquake in Banjaluka 1969). Only after earthquakes in Skopje 1963, Banjaluka 1969 and Montenegro Coast 1979 (Hrasnica and Medic 2012), houses and buildings have been designed according to seismic codes which imposed the use of reinforced concrete confining elements. Research project implemented at the Faculty of Civil Engineering, University of Sarajevo in cooperation with the Institute for Lightweight Structures and Conceptual Design, University of Stuttgart pertains to the in-plane behavior of solid clay brick masonry walls (Medic 2018). First, testing of

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compressive strength and modulus of elasticity on prismatic samples \((l/h/t = 51.4/64.6/12 \text{ cm})\) is presented. Wallets were also jacketed with one- or two-sided mortar strengthening and the obtained load bearing capacity is reported. Then, tests on two plain full scale walls \((l/h/t=233/241/25\text{cm})\) exposed to constant vertical load and cyclic horizontal loads were performed. The walls have no vertical confinement, which was a typical way of construction before introduction of new codes in the mid-60s. Next, one- or two-sided strengthening was applied in order to compare the wall behavior with the unreinforced one. Different concepts and methodologies for strengthening are given in literature (Churilov, 2012). Two full scale walls and nine reduced scale walls \((l/h/t = 100/100/25\text{cm})\) were built in order to test two different jacketing techniques: one with the orthogonal position of the reinforcement mesh with “new” type of connectors and the other having the mesh inclined to 45° (135°) and traditional U-anchors. Depending on the test performed, the results obtained on different wall specimens are presented in the form of hysteretic and pushover curves. Load bearing and deformability parameters related to characteristic limit states (crack limit, maximum resistance and ultimate limit state) are also given.

![Figure 1. a) Seismic intensity map of B&H according to MSK-64 scale, b) partial collapse after Banjaluka 1969 earthquake](image)

2. TESTING OF COMPRESSIVE STRENGTH AND MODULUS OF ELASTICITY

2.1 Plain wallet

The tests were performed according to the European standard (CEN-EN 1052-1) on 6 prismatic samples (wallets) \(l/h/t = 51.4/64.6/12 \text{ cm}\) labeled with P1-P6 (Figure 2). A solid brick with compressive strength of approx. 30 N/mm\(^2\) and nominal dimensions 250/120/65 mm was immersed in water prior to installation (Hrasnica et al. 2014). The smallest possible specimen size was chosen with respect to EN 1052-1 (thickness of the wallet equal to unit thickness) because of easier manipulation with the samples. Samples P1 and P2 were made with mortar that has an average strength of 4.29 N/mm\(^2\), while others were made with a mortar of 2.3 N/mm\(^2\) compressive strength. It is noted that the hardening conditions of mortar in a mold and in a prism are very different, because the brick absorbs water from the mortar. The mortar is composed of lime: cement: sand=1:0.5:4 (grades by volume). The details of experimental testing of brick, mortar and brick-mortar interface are given in Hrasnica et al. (2017). Horizontal and vertical joints are 1.4 cm thick on average. Compression is applied monotonically using Amsler 5 MN press at a speed of approximately 0.20 N/mm\(^2\)/min, and the displacement is monitored by 8 axial LVDTs (Figure 2). The secant modulus of elasticity was determined from the stress and deformation ratio at 1/3 of the compressive strength \((E_m = \frac{F_{max}}{3\varepsilon_{A_p}})\). The resulting mechanical properties are given in Table 1. Figure 3 shows experimentally determined stress – deformation diagrams for 6 masonry prisms. The vertical displacement (deformation) of each prism is defined as the average displacement (deformation) of all LVDTs. The compressive stress is defined as the ratio of the vertical load to the initial surface of the prism, regardless of the fact that the joints were not ideally filled. The average compressive strength is 6.48 N/mm\(^2\) and the coefficient of variation amounts to 36 %. The average value of the modulus of...
elasticity is 4024 N/mm², while the coefficient of variation equals to 46 %. Based on the measured values, it can be concluded that the modulus of elasticity is considerably smaller than the recommendation given in EC6 according to which \( E_m = 1000 \, f_k \). However, according to some authors (Tomaževič 2009) the modulus of elasticity may vary within the limits \( 100 \, f_k \leq E_m \leq 1000 \, f_k \).

**Figure 2.** Testing of compressive strength and modulus of elasticity

**Table 1.** Compressive strength and modulus of elasticity of masonry prisms.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Compressive strength [N/mm²]</th>
<th>Modulus of elasticity [N/mm²]</th>
</tr>
</thead>
<tbody>
<tr>
<td>P1</td>
<td>9.56</td>
<td>5 500</td>
</tr>
<tr>
<td>P2</td>
<td>9.73</td>
<td>2 750</td>
</tr>
<tr>
<td>P3</td>
<td>5.50</td>
<td>2 550</td>
</tr>
<tr>
<td>P4</td>
<td>3.90</td>
<td>4 145</td>
</tr>
<tr>
<td>P5</td>
<td>4.20</td>
<td>1 930</td>
</tr>
<tr>
<td>P6</td>
<td>6.00</td>
<td>7 270</td>
</tr>
</tbody>
</table>

**Figure 3.** \( \sigma - \varepsilon \) diagrams of masonry prisms

Realistic values of elastic moduli for the more compliant mortar vs. the stiffer brick units lead to lateral tension in the brick under far-field compression. The mortar layer attempts 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 (Figure 4) (Blackard et al. 2007). The results of experiments on masonry prisms can be analyzed from two other aspects. The average strength of the prism is significantly greater than the uniaxial mortar strength. The lateral deformation of mortar is prevented so that the mortar is confined, which increases the vertical load bearing capacity. The second aspect relates to the significant ductility of the masonry prism in relation to the brick itself, which is quite brittle. It can be concluded that mortar has a crucial influence on prism deformation, whereby the strength reduction is compensated by the stable post-peak behavior (Oliveira 2003). All prism samples with lower compressive strength and weaker mortar (samples P3-P6) have more ductile responses than samples with greater compressive strength and stronger mortar (P1 and P2), which is consistent with results found in literature (Binda et al. 1988).

![Figure 4. Testing of compressive strength – splitting of the specimen: a) frontal view, b) side view](image)

2.2 Strengthened wallets

During cyclic testing of reduced-scale jacketed walls for different vertical stress intensities, it was noticed that no crushing occurs at the compressed edge or that certain cracking develops due to compression at the toe, but without reduction of the bearing capacity and loss of structural integrity. It was assumed that the strengthening (jackets) prevents lateral wall deformation causing a multi – axial stress state in the wall, thus increasing the vertical load bearing capacity. Masonry prism illustrated in Figure 2 was strengthened with one- or two-sided jackets (Figure 5) made of 3-4 cm thick mortar (compressive strength 8.47 N/mm² and tensile strength 3.47 N/mm²).

![Figure 5. Masonry prism with two-sided strengthening](image)
The jackets were reinforced with Q196 mesh (S500) and specially developed reinforcement detail (Figure 5) made of Ø5mm bars (5 pieces per prism face) and placed in horizontal and vertical joints. This bar is bonded to the wall using a high strength quick hardening mortar. A levelling layer was grouted at the bottom and the top of the prism. Three samples were built with one-sided strengthening and six samples with two-sided strengthening. The measured load bearing capacities and moduli of elasticity of the strengthened wallets are listed and compared with the plain prism results (units from the same consignment) in Table 2. However, most of the tested samples have a bearing capacity of about 600 kN regardless of whether the specimen was strengthened or not. The probable cause of this behavior lies in the fact that lateral buckling of vertical reinforcement was not prevented so that most samples had premature failure (Figure 6), which was not the case during wall testing.

### Table 2. Compressive strength and modulus of elasticity of plain and strengthened masonry prisms.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Capacity [kN]</th>
<th>Elastic modulus [N/mm²]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Plain prism</td>
<td></td>
<td></td>
</tr>
<tr>
<td>P1</td>
<td>590</td>
<td>5500</td>
</tr>
<tr>
<td>P2</td>
<td>600</td>
<td>2750</td>
</tr>
<tr>
<td>Two-sided strengthening</td>
<td></td>
<td></td>
</tr>
<tr>
<td>P7</td>
<td>1080</td>
<td>1945</td>
</tr>
<tr>
<td>P8</td>
<td>1000</td>
<td>1711</td>
</tr>
<tr>
<td>P9</td>
<td>650</td>
<td>2076</td>
</tr>
<tr>
<td>P10</td>
<td>600</td>
<td>1667</td>
</tr>
<tr>
<td>P11</td>
<td>665</td>
<td>2000</td>
</tr>
<tr>
<td>P12</td>
<td>600</td>
<td>2918</td>
</tr>
<tr>
<td>One-sided strengthening</td>
<td></td>
<td></td>
</tr>
<tr>
<td>P13</td>
<td>490</td>
<td>1500</td>
</tr>
<tr>
<td>P14</td>
<td>600</td>
<td>2052</td>
</tr>
<tr>
<td>P15</td>
<td>650</td>
<td>2270</td>
</tr>
</tbody>
</table>

Figure 6. Failure pattern of jacketed prisms – buckling of steel mesh: a) frontal view, b) side view

### 3. EXPERIMENTAL ANALYSIS OF PLAIN AND JACKETED WALLS

Walls were experimentally studied under constant vertical load and the in-plane cyclic horizontal static force. Based on boundary conditions, the walls can be modeled as cantilevers. In total, four walls were
tested, two plain (labeled W1 and W2) and two jacketed walls (W3 and W4). Wall geometry corresponds to a wall in a building typical of the period after World War II and the dimensions are l/h/t = 233/241/25 cm. The strengthened walls additionally have 4 to 5 cm thick jackets across both wall sides. Once the walls are erected, the reinforcement mesh Q196 is placed on both sides. On the first wall (W3) the mesh is placed in the direction of the principal stress trajectories at 45° (135°) inclination and connected to the wall by means of U–shaped anchors Ø5mm (Figure 7a). The anchors were placed in pre-drilled holes which were grouted with cement mortar. The number of anchors is significant and on one side of the wall there is approx. 25 pieces/m². On the second wall (W4) the mesh is placed vertically and connected to the wall using a specially developed reinforcement detail (connector) that is inserted into the horizontal and vertical joints (Figure 7b). The connector is bonded to the wall by using a high–strength quick–hardening cementitious mortar, and on one wall face these is approx. 100 pieces. The concrete class used for jacketing was C35/45.

![Figure 7. Wall geometry and strengthening pattern: a) scheme 1, b) scheme 2](image)

Walls in buildings are usually quite thick and exposed to low compressive stresses. Based on the analysis of a typical building, vertical stress amounts to cca. 0.4 N/mm², which is approximately 1/10 of the measured wallet compressive strength. Therefore, this magnitude of vertical stress is imposed in the tests. The strengthened walls were also tested at a vertical stress of 0.8 N/mm² because the failure did not occur for a lower stress intensity.

The test procedure can be divided in two steps. In the first step, the vertical load is slowly applied to the wall. Then the horizontal force is applied to the upper reinforced concrete beam until the desired displacement is achieved (each cycle consists of three runs).

### 3.1 Plain walls

As expected, the walls failed in shear which is characterized by cross-diagonal cracks passing through bricks or brick-mortar contact (Figure 8a). Crushing and local buckling was noticed in the corners of the wall. The crack pattern at the final stage of the test is shown in Figure 8b. The fissures first began to appear in the middle of the wall, and cracks over 40 mm in width were recorded at the final stage. The final crack width is twice as large as the imposed displacement which was caused by the accumulation of plastic deformation (slip) along the main diagonal cracks. Experimentally obtained hysteretic loop is given in Figure 9. Hysteretic curves of plain walls are generally symmetric and full, meaning that energy is dissipated through non-elastic deformations. It is clear that the slope of the hysteretic curve decreases during unloading, which means that, besides the plastic behavior caused by friction, there are cracks that cause the decrease of the initial stiffness (damage).
Based on the observed failure mechanism, three limit states can be used to describe the behavior of the wall:
1. Crack limit – the state is determined by the displacement $d_{cr}$ and the resistance $H_{cr}$ when first major cracks occur either through bricks or at the brick-mortar contact.
2. Maximum resistance – the state is determined by the maximum resistance $H_{max}$ that is registered during the test and the corresponding displacement $d_{Hmax}$.
3. Ultimate limit state – the state is determined by the maximum displacement $d_{max}$ attained during the test and the corresponding resistance (limit load) of $H_{dmax}$.

Load bearing and deformability parameters for plain walls are listed in Table 3.

![Figure 8. Wall W1 after testing: a) photo, b) crack pattern and crack width](image)

![Figure 9. Force – displacement diagram for W1](image)

Table 3. Load bearing and deformability parameters for plain walls.

<table>
<thead>
<tr>
<th>Wall</th>
<th>$K_{e,i}$ [kN/mm]</th>
<th>$H_{cr}$ [kN]</th>
<th>$d_{cr}$ [mm]</th>
<th>$K_{er}$ [kN/mm]</th>
<th>$H_{max}$ [kN]</th>
<th>$d_{Hmax}$ [mm]</th>
<th>$K_{Hmax}$ [kN/mm]</th>
<th>$H_{dmax}$ [kN]</th>
<th>$d_{max}$ [mm]</th>
<th>$K_{dmax}$ [kN/mm]</th>
</tr>
</thead>
<tbody>
<tr>
<td>W1</td>
<td>116.3</td>
<td>56</td>
<td>0.9</td>
<td>62.2</td>
<td>120</td>
<td>12</td>
<td>10</td>
<td>105</td>
<td>20</td>
<td>5.25</td>
</tr>
<tr>
<td>W2</td>
<td>105.2</td>
<td>52</td>
<td>1.1</td>
<td>47.3</td>
<td>120</td>
<td>8</td>
<td>15</td>
<td>100</td>
<td>20</td>
<td>5</td>
</tr>
</tbody>
</table>
where:
\(K_{e,i}\) – initial elastic stiffness,
\(K_{cr}\) – secant stiffness at the appearance of the first cracks,
\(K_{H\text{max}}\) – secant stiffness at the maximum horizontal force,
\(K_{du}\) – secant stiffness at the maximum displacement.

One can notice significant ductility (ratio of \(d_{\text{max}}\) and \(d_{cr}\)) of the tested plain walls. Although the experimental results indicate higher values, it is recommended that the ultimate ductility factor lies within the range of \(\mu = 2.0 - 3.0\) for unstrengthened unconfined walls (Tomaževič 1999). For practical applications, ductility is maintained at certain limits in order to limit damage to the wall.

### 3.2 Jacketed walls

In order to compare the results, the strengthened walls W3 and W4 were tested under the same vertical load as the plain walls (\(\sigma = 0.4\ N/mm^2\) or \(V = 230\ kN\)). Since failure did not occur, the walls were further tested under doubled precompression (\(\sigma = 0.8\ N/mm^2\) or \(V = 460\ kN\)). Vertical force of higher intensity could not be imposed due to limited capacity of the hydraulic cylinder.

By jacketing the wall, the failure mode fundamentally changes in relation to the previous walls. Specimens W1 and W2 failed in shear with typical diagonal cracking pattern while the strengthened walls behave like a rigid body that rotates around the compressed edge (rocking). There were no cracks caused by compressive stresses in the wall edges, and no loss of bond between the jacket and the wall (delamination) was noticed. The wall was prepared for recording with a digital camera, so that the wall faces were marked with dots of different thickness and stochastic arrangement (Figure 10). After the processing of digital photographs, no cracks were observed on the wall.

Hysteretic curves for wall W3 are given in Figure 11. After the initial increase in force, the stiffness decreases and almost horizontal yield plateau appears. However, the nonlinear response is not a consequence of cracking but of the reduction of the compressed zone on the wall-foundation contact (decompression). The hysteresis has an S-shape, typical for bending regardless of the vertical stress level (Figure 11). When unloading, the curve returns to the starting point following almost the same path as it reached the imposed displacement.

![Figure 10. Rocking of the wall W3](image)

In the real structures the walls cannot rotate (Tomaževič 2016) except possibly in higher stories. Since each wall is a part of a larger structural system, free rotation is mostly prevented (due to the constraining effects of R.C. slabs and other walls), and additional force is diagonally applied, which can cause crushing of the wall toe or the appearance of diagonal cracks. In the experiments conducted it was not possible to model the influence of the rest of the structure on the observed wall. In order to accomplish this, it is necessary to examine the model of the building as a whole. Rotation of the wall can be
prevented by anchoring the reinforcement to the base beam, which is recommended when reinforcing the real structure. In this manner, the strength of steel can be fully utilized. However, the aim of this study was to investigate the seismic behavior of "new" material obtained by coupling the masonry wall with the jackets, rather than the influence of anchored reinforcement on the response of the strengthened masonry. The load bearing and deformability parameters of strengthened walls W3 and W4 for precompression levels of \( \sigma = 0.4 \text{ N/mm}^2 \) and \( \sigma = 0.8 \text{ N/mm}^2 \) are given in Table 4 and Table 5 respectively (negative part of hysteretic curves considered).

Table 4. Load bearing and deformability parameters for strengthened walls (\( \sigma = 0.4 \text{ N/mm}^2 \)).

<table>
<thead>
<tr>
<th>Wall</th>
<th>Crack limit</th>
<th>Max. resistance</th>
<th>Ultimate resistance</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>( K_{e,i} ) [kN/mm]</td>
<td>( H_{cr} ) [kN]</td>
<td>( d_{cr} ) [mm]</td>
</tr>
<tr>
<td>W3</td>
<td>370</td>
<td>64</td>
<td>0.4</td>
</tr>
<tr>
<td>W4</td>
<td>355</td>
<td>63</td>
<td>0.4</td>
</tr>
</tbody>
</table>

Table 5. Load bearing and deformability parameters for strengthened walls (\( \sigma = 0.8 \text{ N/mm}^2 \)).

<table>
<thead>
<tr>
<th>Wall</th>
<th>Crack limit</th>
<th>Max. resistance</th>
<th>Ultimate resistance</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>( K_{e,i} ) [kN/mm]</td>
<td>( H_{cr} ) [kN]</td>
<td>( d_{cr} ) [mm]</td>
</tr>
<tr>
<td>W3</td>
<td>380</td>
<td>115</td>
<td>0.5</td>
</tr>
<tr>
<td>W4</td>
<td>400</td>
<td>110</td>
<td>0.5</td>
</tr>
</tbody>
</table>

Comparison of hysteretic curves for W3

![Comparison of hysteretic curves for W3](image)

Figure 11. Comparison of hysteretic curves for different vertical precompression (specimen W3)

### 3.3 Reduced-scale walls

A total of 9 reduced scale walls denoted with Z1 – Z9 (l/h/t = 100/100/25 cm) was constructed: plain, with one-sided strengthening or two-sided strengthening using 3-4 cm thick mortar. The width of the specimen was not scaled because only standard unit dimensions were available. The motivation for experimenting on scaled walls, where model similitude was not respected (distorted model), was to test possible strengthening techniques, which would later be applied on the full scale walls, in a cost-effective way. The strengthening was applied by machine spraying so that the mortar completely filled out the furrows in vertical and horizontal joints (about 0.5 cm). The dowel effect and good adhesion of mortar increased the shear strength of the wall and the vertical load bearing capacity. Reduced-scale walls were tested for different vertical stress intensities. Figure 12 shows the pushover curves of Z8 and
Z9 plain walls tested under vertical stresses equal to \( \sigma = 0.6 \text{ N/mm}^2 \) and \( \sigma = 1.0 \text{ N/mm}^2 \). Both have failed in shear, but from the diagram it is evident that stiffness and load bearing capacity increase for higher precompression levels. Figure 13 shows the pushover curves of the walls that failed in a rocking mode. Specimen Z6 was jacketed on both sides, while the specimen Z7 was unstrengthened. From the force-displacement relationship it can be deduced that the stiffness and resistance increase as the vertical precompression rises.

![Comparison of pushover curves for Z8 and Z9](image)

Figure 12. Comparison of pushover curves of Z8 (V = 150 kN, \( \sigma = 0.6 \text{ N/mm}^2 \)) and Z9 (V = 250 kN, \( \sigma = 1.0 \text{ N/mm}^2 \)) that failed in shear

![Comparison of pushover curves for Z6 and Z7](image)

Figure 13. Comparison of pushover curves of Z6 (V = 400 kN, \( \sigma = 1.6 \text{ N/mm}^2 \) and V = 250 kN, \( \sigma = 1.0 \text{ N/mm}^2 \)) and Z7 (V = 100 kN, \( \sigma = 1.0 \text{ N/mm}^2 \)) that failed in bending

Regardless of the precompression level, the integrity of the strengthened masonry was preserved. Excluding a few cracks on the compressed edge, the jackets do not separate from the wall, which is the most common cause of failure of such constructions (Figure 14). No increase in brittleness was observed, which is often the result of higher compressive stresses. By applying jacketing, the failure mechanism changes fundamentally. Namely, the shear failure changes to bending failure because by adding jackets, the tensile strength of the strengthened masonry is considerably increased. The failure mode depends on the wall slenderness: rocking and crushing at the wall toe pertain to short walls, and shear failure is typical for longer walls. The tested walls with the width:height ratio of 1:1 do not fall either into long nor short walls so that both failure modes are possible. The failure mode depends also on the vertical compressive stress: for higher vertical stress (1.0 N/mm²), the plain wall fails with diagonal cracks, while for lower vertical stress (0.4 N/mm²) rocking occurs. If rocking and crushing at the wall toe occur, it is possible that the cracks extend diagonally through the wall, so this failure mode is called mixed (hybrid).
4. CONCLUSIONS

From the presented experimental study valuable data were obtained on the mechanical properties and behavior of brick walls typical for buildings erected after World War II in the Western Balkans. The test results of compressive strength and elasticity modulus show considerable variation. Based on the measured values, it can be concluded that the modulus of elasticity is considerably smaller than the recommendation given in EC6. However, the values fit within the limits provided in literature according to which $100 \, f_k \leq E_m \leq 1000 \, f_k$.

Two conclusions can be drawn from experiments on masonry prisms regarding mortar. The average strength of the prism is significantly greater than the uniaxial mortar strength. The lateral deformation of mortar is prevented so that the mortar is confined, which increases the vertical load bearing capacity. The second aspect relates to the significant ductility of the masonry prism in relation to the brick itself, which is quite brittle.

During cyclic testing of reduced scale jacketed walls for different vertical stress intensities, it was noticed that no crushing occurs at the compressed edge or that certain cracking develops, but without reduction of the bearing capacity and loss of structural integrity. It was assumed that the strengthening (jackets) prevents lateral wall deformation causing a multi-axial stress state in the wall, thus increasing the vertical load bearing capacity. However, most of the tested samples have approximately the same load bearing capacity regardless of whether it is strengthened or not. The probable cause of this behavior lies in the fact that lateral buckling of vertical reinforcement was not prevented so that most samples had premature failure.

The experimental study of the behavior of full scale walls refers to the tests under constant vertical pressure of $\sigma = 0.4 \, \text{N/mm}^2$ and in-plane cyclic horizontal static force. Failure is characterized by a cross-diagonal cracks that pass through bricks or brick-mortar contact. Hysteretic curves are full and significant energy is dissipated when shear governs failure. The slope of hysteretic curve decreases when unloading, which means that, aside from plastic deformations typical for joint failure, bricks fail in tension and damage occurs.

Unlike plain walls, the walls strengthened with 4-5 cm reinforced concrete jackets (concrete class C35/45) do not fail in shear. Jacketed specimens rotate in a rigid body mode (rocking) without appearance of diagonal cracks for vertical precompression of $\sigma = 0.4 \, \text{N/mm}^2$ and $\sigma = 0.8 \, \text{N/mm}^2$. Also, no debonding (detachment of jackets from the wall) was observed. Linear increase of force with displacement increment is typical for the initial phase of the obtained pushover curves. The other part of the curve is almost horizontal which implies stiffness degradation and yielding. However, in case of rocking, yielding is not caused by material degradation but the reduction of the compressed zone as imposed displacement increases.

By examining the plain walls, significant ductility was measured, which is also true for jacketed walls where ultimate displacement capacity was not attained due to limitations of the testing apparatus. For practical applications, ductility is maintained at certain limits in order to limit damage to the wall. The
vertical stress intensity affects the behavior of the wall. Increasing the vertical stress increases the load bearing capacity related to horizontal force. Based on testing of the reduced-scale walls, it can be concluded that the reinforced mortar has the same effect as the reinforced concrete. Therefore, the jacket does not have to be 5 cm thick and made of strong concrete, a thinner jacket made of cement-based mortar is acceptable. Unlike concrete that has to be installed in formwork and vibrated in a very tight space, mortar can be applied by spraying so that such strengthening methods would be considerably more efficient. Additionally, it was concluded that one-sided strengthening has almost the same effect as double-sided. Furthermore, two different layouts of reinforcement, which were implemented in the jackets, have the same effect on the wall behavior. However, orthogonal layout of reinforcement mesh is economically more favorable because no drilling and grouting of anchors with expensive material is carried out.

5. ACKNOWLEDGMENTS

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