

# Analysis of masonry walls strengthened with RC jackets

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## SUMMARY:

The presence of existing brick masonry buildings in the building stock within area with high seismic activity requires careful assessment of their seismic behaviour and retrofitting measures. Strengthening with RC jackets, although traditional, is still highly applicable. This paper presents results of a complete programme for experimental and analytical study of this strengthening method applied symmetrically on both sides of the walls on the whole wall area. Cyclic load tests on masonry walls were performed to evaluate their in-plane shear behaviour and identify shear strength, stiffness and energy dissipation. Two series of unreinforced and strengthened brick walls were tested with the purpose to compare their behaviour under cyclic horizontal loading. The results from the tests showed that the strengthening method leads to significant improvement in the shear resistance of the jacketed walls. Analytical models were used to predict the shear resistance of walls. Good agreement with experimental results was obtained with a model based on tensile strength of masonry.

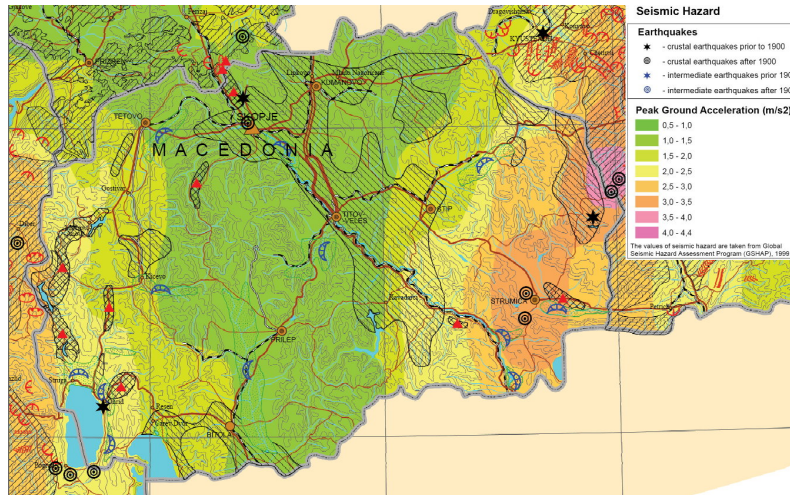
*Keywords: cyclic tests, masonry, RC jacket, strengthening, analytical*

## 1. INTRODUCTION

Masonry together with wood is considered as one of the most important building materials in the history of mankind. It has been used as construction material for several thousand of years. From the first masonry structures in the Neolithic age, to the Tower of Babylon, pyramids of Egypt, Great China Wall, to the Greek Parthenon, masonry has helped build some of the world's most iconic structures.

In the last decades other materials like steel and concrete have been used frequently and thus replacing masonry as a structural material. This situation is present and particularly remarkable in Macedonia, where almost all new buildings are constructed using reinforced concrete. Masonry has been mostly used as a non-structural material, as an infill of reinforced concrete and steel frames. Over the years, the research and development of design rules for reinforced concrete structures increased and caused insufficient knowledge and design procedures for masonry. Although there is national masonry code adopted from the regulations in former Yugoslavia, masonry was not used as load bearing material. This situation got worse after catastrophic 1963 Skopje earthquake.

Macedonia is one of the most active seismic places in Balkan Peninsula, see Fig. 1.1. It lies in the south part of the Eurasian plate where three tectonic plates, African, Asian and European meet. It is mostly the work of African plate sliding beneath Eurasian plate that caused the geographic formation which we see today, and it is also the reason why there are frequent earthquake tremors in the region. The friction between the plates has brought about a number of fault lines which run across Macedonia approximately along the course of river beds. The one under Kochani along the river Bregalnica contributed to the 1904 earthquake, and the Skopje fault line along the river Vardar contributed to the 1963 earthquake which destroyed about 10% of the buildings, while other 65% were damaged beyond economical repair.



**Figure 1.1.** Seismic hazard map of Macedonia

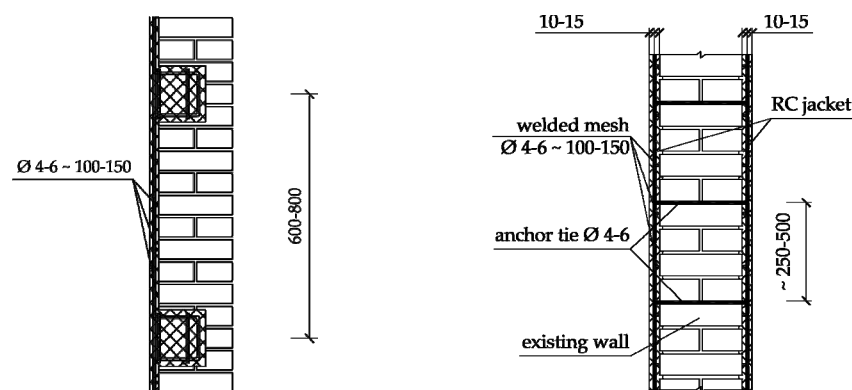
Unreinforced masonry (URM) buildings represent a significant portion of the building stock in many countries in the world. There is a large building stock of low-rise residential and public URM buildings in Macedonia and Balkan region. The primary disadvantage of URM buildings located in active seismic regions is the fact that usually they are old buildings, constructed from inhomogeneous material and mainly designed to support vertical loads only. Moreover, URM is not able to carry tensional forces due to its low tensile strength. Those buildings are particularly vulnerable to seismic actions and therefore susceptible to extreme damage. Their vulnerability is caused by the failure of unreinforced masonry walls due to the in-plane and/or out-of plane seismic loading.

In-plane resistance of unreinforced masonry walls is mainly depended on mortar strength and masonry unit strength and proportions. If the horizontal forces are strong enough to exceed the in-plane strength capacity of the wall, then a shear failure will occur. Shear failure is most common failure mode associated with unreinforced masonry characterized by brittle tensile cracking through mortar; masonry unit and mortar-unit interface, see Fig. 1.2. As can be observed after 1963 Skopje earthquake, most of the damage in masonry buildings occurred in the ground floor, by formation of typical X-cracks and/or collapse of the weak ground floor.

In addition, large number of existing masonry buildings does not satisfy the latest code provisions and to improve their seismic resistance application of strengthening is necessary. This paper presents experimental and analytical results obtained in a research that investigates the effects of strengthening masonry by traditional strengthening method often used in Macedonia. This method is applied in case of seriously damaged brick and stone masonry walls or in needs to strengthen existing structure and consists of application of reinforced concrete (RC) jackets on the masonry walls.



**Figure 1.2.** Damage on residential masonry buildings after 1963 Skopje earthquake (Ambraseys, 1967)



**Figure 1.3.** Application of single- and double-sided RC jackets to brick masonry wall

RC jacketing is strengthening and retrofitting technique that consists of application of single- or double-sided RC walls or coatings. This technique is suitable both for stone and brick masonry. In case of using reinforcing steel, first the plaster is removed from the walls and mortar joints are cleaned from mortar. If any cracks appear, they are grouted and then anchor ties are built in pre-drilled then the surface is cleaned, moistened and spattered with cement milk. The concrete is applied in two layers with reinforcing mesh between them. The mesh is connected on both sides with the steel anchors by welding or by tying with wire. The usual total thickness of the jacket varies from 30-100 mm, depending on the method for application of the concrete layers.

Although similar to shotcrete, it has some distinctive features. Establishing good bond between the new and old material is one of the crucial steps in the application. The shear connection which transfers the loads from the new jacket to the existing wall is provided with reinforcing cages in case of single-sided RC jacketing and with cemented or epoxied reinforcing anchor ties in case of double-sided RC jacketing, see Fig. 1.3.

The effect of strengthening brick and block masonry walls by RC jacketing has been experimentally investigated only in few cases (Sheppard and Tomažević, 1986; Sheppard and Terčelj, 1980; Alcocer et al., 1996; Penazzi et al., 2001; ElGawady et al., 2004). The experiments, which showed that RC jacketing improves the lateral capacity, have also indicated the importance of adequate anchoring of jacket reinforcement to the existing masonry. If the connection was not adequate to prevent splitting, the coating separated from the wall at the occurrence of cracks in the masonry wall and buckled.

In order to test the lateral resistance of masonry walls build from solid clay bricks and lime mortar, an experimental analysis of unreinforced (UMW) and strengthened walls (SMW) has been carried out at Laboratory for testing structures and materials at the Faculty of Civil Engineering in Skopje. The experimental program comprises a set of tests which identify the mechanical properties of masonry components and in-plane cyclic shear tests on UMW and SMW walls for the purpose of comparing their seismic behaviour.

## 2. CHARACTERISTIC PROPERTIES OF MASONRY

To design new masonry buildings or repair and strengthen the existing ones, it is crucial to have knowledge and to understand mechanical behaviour of masonry. Any experimental, numerical or analytical analysis can be accurate only if mechanical properties of masonry and its components are known. It is fundamental to perform experimental tests on masonry bricks, blocks, mortar samples and wallets to be able to determine the properties from the undamaged state until failure.

### 2.1. Tests on clay bricks

Unreinforced masonry walls made from solid clay bricks and lime mortar are rather common types of structural elements in Macedonia, in particular in rural areas and in many existing low-rise residential

and public buildings. The choice of materials and strengthening method has been designed to recreate conditions that are representative of what can be found in existing buildings. It was decided to test the properties of three series of new solid clay bricks produced from different manufacturers currently offered on the market. To compare and account for time dependent effects, such as ageing, and to determine the properties of bricks which can be found in existing buildings, one series of bricks were pulled out from an +60 years old building. All units have dimensions 250x120x65mm. Physical properties of the bricks and the compressive strength of brick units were obtained according to MKS B.D8.011 (1987), and are summarized in Table 2.1.

**Table 2.1.** Physical and mechanical properties of clay bricks

Series	Density (kg/m <sup>3</sup> )	Compressive strength (N/mm <sup>2</sup> )	Tensile flexural strength (N/mm <sup>2</sup> )
A	1922.60	10.84	2.66
B	1719.90	4.02	1.48
C	1239.70	6.43	0.88
D	1448.30	6.10	2.86
Mean	1582.63	6.85	1.97

## 2.2. Tests on mortars

Preparation and testing of compressive and tensile flexural strength of mortar specimens was according to MKS U.M8.002 (1968) and MKS B.C8.042 (1981). These standards are similar to actual EN 1015-11 (1999). Mortar properties were tested on 6 lime mortar specimens and 9 cement-lime mortar specimens with dimensions lxbxh=160x40x40mm. First, tensile flexural strength was tested, and the compressive strength was tested on the mortar specimen left over from the tensile flexural strength tests. Table 2.2 shows the main test results.

**Table 2.2.** Physical and mechanical properties of lime (LM) and cement-lime (CLM) mortar specimens

Series	Density (kg/m <sup>3</sup> )	Compressive strength (N/mm <sup>2</sup> )	Tensile flexural strength (N/mm <sup>2</sup> )
LM	1383.48	0.55	0.10
CLM	2084.36	26.92	5.97

## 2.3. Tests on concrete and reinforcing steel

The basic properties of concrete used in RC jacketing were determined experimentally. Normal portland cement "Titan" type PC 30p 45S, fine aggregate with three fractions, and regular tap water were mixed to produce the concrete which was applied on the masonry walls. The grades by volume of cement:sand were 1:3. Nine prism specimens with dimensions lxbxh=160x40x40mm were casted and tested. Standardized testing procedures were applied on concrete prism specimens. The density was 2148.95 kg/m<sup>3</sup>, the compressive strength was 30.60 N/mm<sup>2</sup> and the tensile flexural strength was 5.37 N/mm<sup>2</sup>.

Ten specimens with length of 500 mm were cut from reinforcement meshes to determine the mechanical properties. The diameter of the wires in both directions of the mesh was 4.2 mm welded at a distance of 100 mm. The ultimate tensile strength of the reinforcement was 630.85 N/mm<sup>2</sup>.

## 2.4 Compressive strength of masonry

The compressive strength of masonry was determined by tests on a series of wallets according EN 1052-1 (1999). Three wallets with dimensions 520x385x125 mm were assembled in the laboratory. Masonry materials were taken from the same consignment used for erecting the shear walls tested under cyclic loads. Solid clay masonry brick from series A and lime mortar with grading 0:1:3 were used to build wallet samples. All wallets presented similar behaviour under axial compression loads in terms of maximum load and deformation capacity. Wallet UMWC-1 presented a deviation from the other two wallets in relation to stiffness and maximum load. Still, its ultimate deformation capacity

was close to other wallet samples. Main mechanical properties obtained by tests are provided in Table 2.3. The compressive behaviour of masonry appeared to be very brittle. Compared to the approach given in EN 1996-1-1 (2005) a difference of about 31% in the compressive strength of masonry was found out. However, good agreement in the estimated modulus of elasticity suggested in the code and in the tests was obtained.

**Table 2.3.** Mechanical properties of masonry: force at failure ( $F_{\max}$ ), compressive strength ( $f_m$ ), characteristic compressive strength ( $f_k$ ) and modulus of elasticity ( $E$ )

Specimen	$F_{\max}$ (kN)	$f_m$ (N/mm <sup>2</sup> )	$f_k$ (N/mm <sup>2</sup> )	$E$ (N/mm <sup>2</sup> )	$E/f_k$
UMWC-1	206	3.17	2.64	2089.83	791.30
UMWC-2	250	3.85	3.21	3181.27	992.56
UMWC-3	253	3.89	3.24	3353.41	1033.86
Mean	236.66	3.64	3.03	2874.84	939.24
CV (%)	9.09	9.09	9.09	19.46	11.28

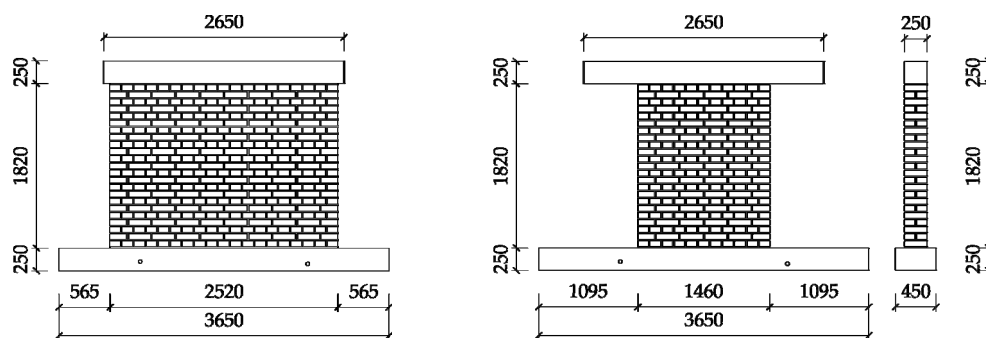
## 2.5 Initial shear strength of masonry

Test according to EN 1052-3 (2002) were conducted on nine specimens with dimensions  $l \times b \times h = 225 \times 118 \times 251$  mm prepared with units from series A and lime mortar. All specimens had three courses of units with one unit in length and a 10 mm connecting joint between them. To determine characteristic initial shear strength of masonry, three different levels of precompression were applied. Normal forces with intensity 6, 18 and 30 kN acting as precompression loads were applied to the specimens to obtain precompressive stresses of approximately 0.2, 0.6 and 1.0 N/mm<sup>2</sup>. The initial shear strength of 0.11, 0.41 and 0.65 N/mm<sup>2</sup> for corresponding precompressive stress levels was obtained. As expected masonry presented low shear strength.

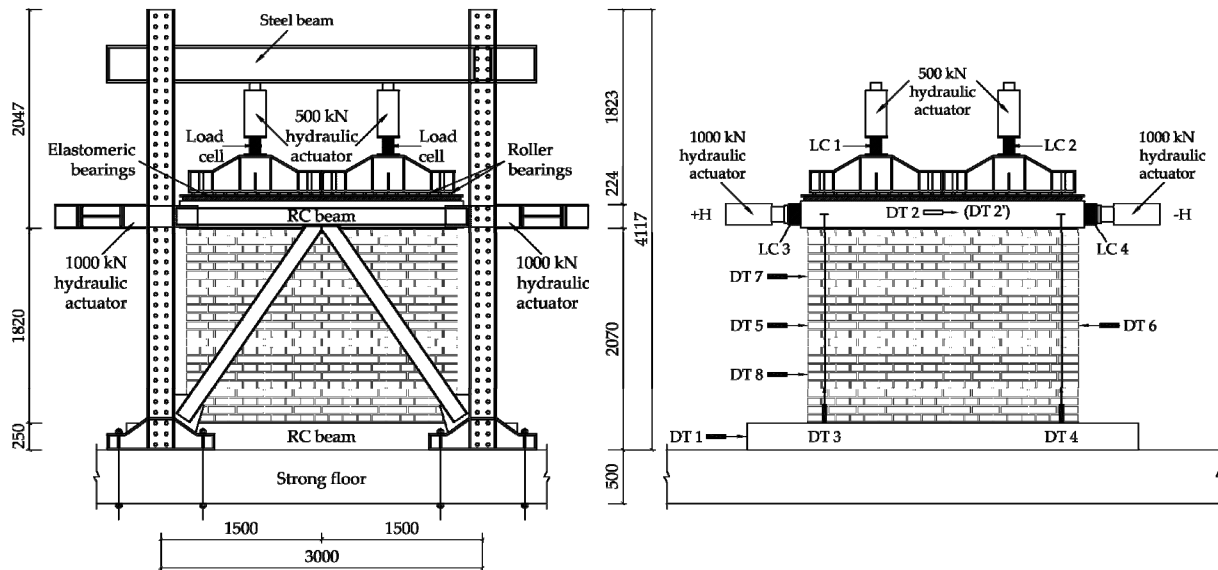
From the linear regression analysis, the initial shear strength at zero normal stress was obtained and was found to have negative value indicating negligible shear strength. For practical applications of masonry of type such as the tested ones, initial shear strength can be assumed zero. The obtained coefficient of friction strongly depends on unit-mortar surface contact and it was obtained as  $\mu = 0.6667$ , which gives characteristic angle of internal friction of  $\alpha = 33.66^\circ$ .

## 3. IN-PLANE SHEAR BEHAVIOUR OF MASONRY WALLS

The experimental program was based on in-plane static cyclic loading tests performed on UMW and SMW walls. Two geometry configurations appropriate for the laboratory test set-up were selected. The lengths of the walls were 1460 mm and 2520 mm and the height of the walls was fixed to 1820 mm. In total eight walls with solid clay bricks and lime mortar were constructed. These walls were specifically developed for the experimental research purposes of Churilov (2012). Two geometry aspect ratios  $h/l$  of 0.7 (squat walls) and 1.2 (tall walls) were used. The wall geometry in longitudinal and transversal section is shown in Fig. 3.1. Two vertical load levels of 0.5 N/mm<sup>2</sup> and 1.0 N/mm<sup>2</sup> were applied and kept constant during the tests.



**Figure 3.1.** Geometry of the squat and tall walls for in-plane testing



**Figure 3.2.** Test set-up for in-plane horizontal cyclic loading and instrumentation with displacement transducers and load cells

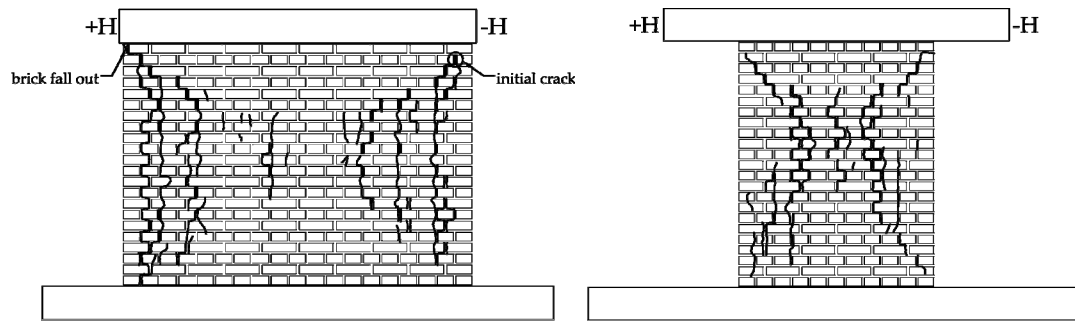
Due to the limited space in the laboratory, first the UMW were constructed and tested. After that the walls designated to be strengthened were constructed on the same RC beams sets, with the same materials. RC jackets were applied 28 days after building the walls, which is estimated time for the lime mortar to reach its designed strength. The concrete was poured into moulds attached to the masonry walls. After the casting, the concrete was cured by wetting the surfaces for the next 28 days. An important notice is that the reinforcing meshes were not anchored to the RC beams due to the fact that the tests were designed to investigate the behaviour of strengthened masonry with RC jackets as structural material tested on the primary structural elements, i.e. walls.

The tests were carried out according to the test set-up shown in Fig. 3.2. Normal compressive loads were applied by two vertical actuators with capacity 500 kN. The vertical load was transferred on a stiff steel load distribution beam positioned over a series of steel roller bearings over the top RC beam. To obtain uniform vertical load distribution over the whole horizontal cross-section of the wall, elastomeric rubber bearings were inserted between the top RC beam and the steel plate. The walls were placed over a layer of mortar on the bottom RC beam which was fixed to the strong floor by steel anchors. Thus, the set-up enables fully fixed boundary conditions on the top and bottom edges of the walls. Lateral loads were simulated with horizontal actuators with capacity of 1000 kN placed on both sides of the wall and were manually applied on the top RC beam with loading rates of 0.2 - 0.5 kN/s until reaching the specified displacement increments. The cyclic load tests were carried out with displacement controlled increments and stepwise increased amplitudes repeated three times at each specified increment.

Horizontal and vertical displacements under cyclic loading have been instrumented with a set of displacement transducers. Applied forces were instrumented with load cells located between the actuators and the load application point. Data acquisition was automatic with 8 channels Spider8, PC-based DAQ unit. Horizontal displacements at points DT 2, DT 2', DT 5 and DT 6, as well as horizontal forces obtained with the load cells LC 3 and LC 4 were automatically stored in digital form with the computer software. Other measured values were manually recorded after each displacement cycle.

### 3.1 Failure modes

The most important results are summarized and presented through obtained failure modes and force-displacement hysteresis curves. All unreinforced walls failed in shear due to the formation of diagonally oriented cracks in the bricks or passing through the bricks or in the brick-mortar interface, see Fig. 3.3. No local buckling or crushing of the bricks was observed except for crushing of the bricks



**Figure 3.3.** Failure modes of UMW3 and UMW4

in the bottom left corner of the wall UMW1 and small brick fall-out in top left corner of the wall UMW3. However, a difference in the crack pattern was noticed for squat and tall walls. In particular, walls UMW1 and UMW3 presented mainly vertically oriented cracks passing through the bricks or along the head joints. At ultimate state, the cracking pattern with 'hour-glass' shape was observed. Initially, vertical cracks started to develop in the vertical planes closest to wall edges and subsequently started to propagate to the centre of the wall. Walls UMW2 and UMW4 demonstrated typical diagonally oriented cracks.

The strengthening method changed the failure modes of the walls if compared to the unreinforced walls. All SMWs failed with predominantly compression failure in the bottom corner areas. The tested walls developed bending failure mode and after reaching the tensile and bond strength between the wall and the bottom beam they started to rotate in each displacement cycle as rigid bodies. Notable failure mode was separation of the RC jacket from the masonry wall and lifting of the specimen from the bottom RC beam in the direction of the applied load. This research confirms that separation of the jackets is associated with their mutual connection and connection to the masonry. The findings of the current research are consistent with the recommendations for the number of anchors given by (Sheppard and Terčelj, 1980 and Tomaževič, 1999).

The described cracking pattern was observed on the outer face of the jackets. The wall SMW1 demonstrated cracking in the bottom corners of the wall. Bending of the vertical reinforcement in a plane normal to the wall plane at a displacement cycle of 4.5 mm was observed. SMW3 showed typical bending failure mode with development of horizontal crack in the connection plane of the wall and the bottom RC beam. Some toe cracking caused by the compression stresses was obtained in the bottom corner. The tall wall loaded with higher vertical load failed with pronounced separation of the jacket from the wall. During this failure, the encased masonry was squeezed from the jackets within a range of 30-50 mm. Similarly to lightly loaded squat wall, SMW3, the wall SMW4 showed characteristic bending failure with horizontal cracking at the bottom RC beam due to exceeding of the tensile strength.

Since the walls did not show many cracks on their outer surface and to inspect the level of damage in the masonry wall, it was decided to remove the RC jackets from the walls prior their complete demolition. The jackets were carefully removed from the masonry by using power hammer. The most interesting finding was that the walls loaded with higher precompression level exhibited cracking in the masonry wall. The most extensive cracking was obtained at tall wall SMW2, see Fig. 3.4. On contrary, the walls subjected to lower precompression stresses did not suffer any particular damage in the masonry. A possible explanation for this might be that the walls with lower precompression level behaved as one compact element and the shear and tensile strength were not exceeded. From the imprint on the inner surface of the jackets it can be concluded that the bond strength between the concrete and masonry was not exceeded. The separation of the jackets was due to splitting of the bricks in the vertical direction with respect to the wall. This result may be explained by the fact that the splitting strength of the bricks was achieved. It can therefore be concluded that the splitting strength of the bricks is important mechanical property which can control the separation failure mode.



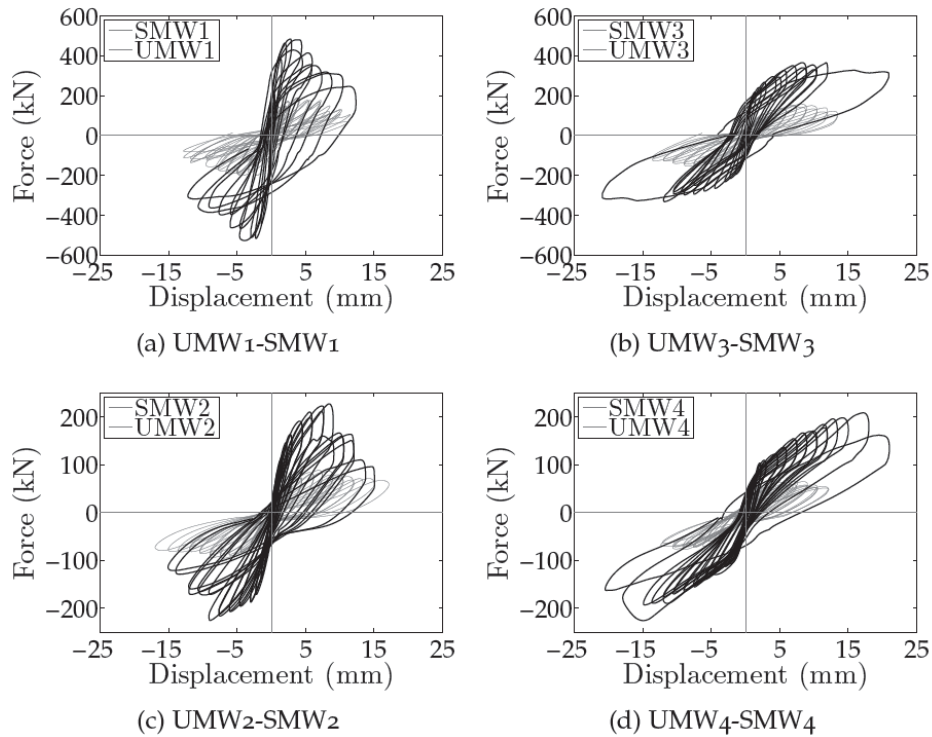


**Figure 3.3.** Experimental cracking pattern for strengthened masonry walls (masonry part)

### 3.2 Force-displacement diagrams

The effects of the strengthening intervention on the resistance of masonry walls have been compared to the in-plane seismic resistance of the unreinforced brick masonry walls. Significant improvement of the shear capacity of SMWs compared to UMWs was achieved, see Figure 3.4. The maximum forces observed at strengthened specimens were higher than the corresponding forces obtained in the unreinforced specimens within the range of 123–204 %.

The highest effect of the strengthening to the lateral capacity was obtained at tall wall loaded with moderate level of vertical stresses  $\sigma_0=0.5 \text{ N/mm}^2$  (SMW4). The deformation capacity in the ultimate limit state of the strengthened walls was increased with significant improvement in the walls loaded with  $\sigma_0=0.5 \text{ N/mm}^2$ . One unanticipated finding was that the deformation capacity of the walls loaded with high vertical stresses was not improved. Moreover, for tall wall SMW2 it was decrease for about 5%, although positive effect was obtained for the other limit states. Table 3.1 presents the average test



**Figure 3.4.** Comparison of force-displacement hysteresis loops of UMWs and SMWs



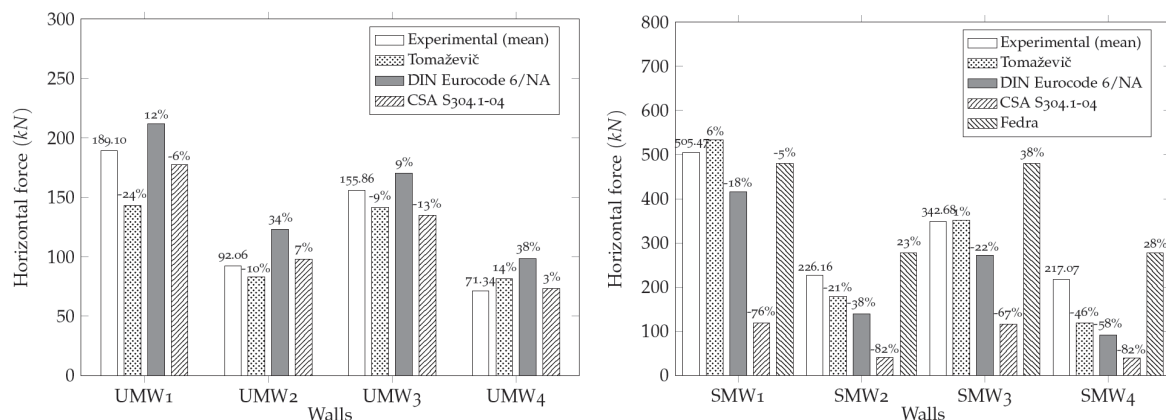
results in terms of capacity of the walls ( $H_{\max, \exp}$ ), drift ratio ( $\theta_{\max, \exp}$ ), tensile strength ( $\tau_{\max, \exp}$ ) and the coefficient of equivalent viscous damping ( $\xi_{eq}$ ) obtained at maximum resistance limit state.

**Table 3.1.** Summary and comparison of cyclic shear test results between UMWs and SMWs

Spec.	l/h/t	$\sigma_0$ (N/mm <sup>2</sup> )	$\sigma_0/f_k$	$H_{\max, \exp}$ (kN)	$\theta_{\max, \exp}$ (%)	$\tau_{\max, \exp}$ (N/mm <sup>2</sup> )	$\xi_{eq}$ (%)
UMW1	2520/1820/250	1.0	0.33	189.10	0.29	0.300	9.86
UMW2	1460/1820/250	1.0	0.33	92.06	0.38	0.252	11.42
UMW3	2520/1820/250	0.5	0.17	155.86	0.46	0.247	11.93
UMW4	1460/1820/250	0.5	0.17	71.35	0.44	0.195	10.73
SMW1	2520/1820/250	1.0	0.33	505.47	0.18	0.802	6.84
SMW2	1460/1820/250	1.0	0.33	226.16	0.48	0.620	7.86
SMW3	2520/1820/250	0.5	0.17	348.62	0.55	0.553	7.39
SMW4	1460/1820/250	0.5	0.17	217.07	0.88	0.595	6.23

## 5. ANALYTICAL MODELS FOR PREDICTING MAXIMUM CAPACITY

In order to provide simplified analytical models for analysis of RC jacketed masonry walls, the current and code defined design procedures for in-plane shear and bending in masonry walls have been investigated. Shear capacity design and bending design procedures in design codes PIOVSP (1981), EN 1996-1-1 (2005) and DIN EN 1996-1-1/NA (2011), CSA S304.1-04 (2004), provisions given by Tomažević (1999) and proposed design equations developed in FEDRA (2003) were considered. Predicted shear and bending resistance of unreinforced and strengthened masonry walls were compared with the lateral capacity of the walls obtained from the tests. Bending resistance was included due to mixed failure modes of the strengthened walls. Due to the effects of symmetrical application of RC jackets on both sides of the walls and the limited experimental programme, analogy with reinforced masonry was established. The correlation between test results and analytical models for unreinforced and strengthened masonry is presented in Figure 4.1. Comparisons between the data were made without considering the contribution of the vertical reinforcement. This was due to the fact that the tested walls were not anchored with the vertical reinforcement to the bottom and top RC beam. The closest results to the experimental ones were obtained with the analytical model proposed by Tomažević. This model was particularly precise for squat walls giving only 1 – 6% difference from the experimental values. The higher disagreement was obtained for tall walls. DIN EN 1996-1-1/NA predicted relatively well as a result of the contribution of the masonry failing in diagonal shear, while as in case with CSA S304.1-04 the contribution of the steel was ignored. Overall, good prediction was shown with the model found in FEDRA. But, it is based on the shear strength of the jacketed masonry section and it is not applicable for the case of failure mechanisms obtained with the tests. The results suggested that the analytical model for reinforced masonry walls proposed by Tomažević could be used in analytical studies of the lateral resistance of RC jacketed masonry walls.



**Figure 3.4.** Comparison between experimental and analytical shear behaviour for UMWs and SMWs

## 5. CONCLUSIONS

Comparison of the seismic resistance of unreinforced and strengthened masonry walls loaded with constant vertical forces and cyclic horizontal forces until failure is presented. The geometry configuration shows influence on the shear behaviour of the strengthened walls. Tall walls exhibited higher maximum capacity if loaded with moderate vertical stresses. The effectiveness of the strengthening method is more significant when applied on the squat walls subjected to high vertical stresses. With respect to deformation capacity, the strengthening method is more effective on walls loaded with low vertical stresses and any geometry configuration. The results from the tests showed that the strengthening, applied symmetrically on both wall surfaces by covering the whole wall area, leads to significant improvement in the shear resistance of the jacketed walls. The findings from this analysis make a contribution in analysis and design of strengthened masonry walls with RC jackets. The best model to correlate the test results for strengthened walls with the analytical model for reinforced walls was the model proposed by Tomažević.

## ACKNOWLEDGEMENT

The authors acknowledge University "Ss. Cyril and Methodius", Faculty of Civil Engineering-Skopje for help in preparation and execution of laboratory tests, Faculty of Mechanical Engineering for contribution in data acquisition during experimental tests, factory for clay bricks and blocks "Elenica", for supplying bricks used in the tests, "ADING" AD for supplying admixtures for test set-up and factory "Karpos" for providing RC beams.

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