FAILURE MODES FOR IN-PLANE MONO AND BIAXIALLY LOADED BRICKWORK MASONRY

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ABSTRACT

Brickwork masonry is a composite, heterogeneous, nonlinear structural material. As with other composite materials, also with masonry the mechanical properties are conditioned with the properties of composite components, their volume ratio and the properties of bond between the bricks and the layers of mortar – joints. Furthermore, since its mechanical properties strongly depend from the orientation of the bed joints and the stress state acting on the joints, the failure modes of in-plane loaded masonry can be attributed either to the failure of each of the aforementioned component or due to their most unfavorable combination.

Following the aforesaid, through investigation of the influence of five different mortar compositions on the mechanical properties of brickwork masonry needed for seismic assessment of masonry structures, an extensive program of testing of mono and biaxially loaded masonry has been carried out. Over 50 specimens were tested by means of compressive, diagonal and shear tests. All masonry specimens were made with solid clay bricks. The basic mortars that were investigated were: cement mortar with the volume ratio of cement to sand = 1:4, cement-lime mortar (cement: lime : sand = 1:1:6) and lime mortar (lime : sand = 1:3). The other two types of mortar were derived through further modifications of the cement-lime mortar with polypropylene fibres (micro-reinforced mortar) and with macro reinforcement of the bed joint by using glass-fibre mesh coated with resin and embedded within the bed joint.

Through phenomenological analysis of the actual failure mechanisms which correspond different testing techniques for in-plane mono and biaxially loaded masonry we have experimentally established the influence of the stiff and soft mortar as well as the micro and macro reinforcement of the masonry on the failure modes. Through tests of in-plane biaxially loaded masonry specimens by means of shear tests of masonry elements we have also

Key words: compressive test, diagonal test, shear, test, failure mechanisms, mortars
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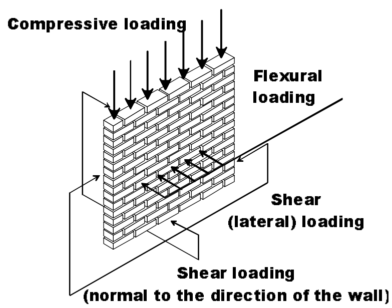
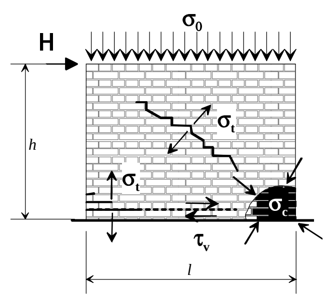
INTRODUCTION

The failure mechanisms of the masonry strongly depends from the most unfavourable loading conditions that could be expected (Figure 1-a). Nevertheless, when subjected to a seismic ground motion, the in-plane walls are those who provide the stability necessary to avoid collapse. In almost all current numerical models for the assessment of the seismic resistance of the masonry structures, the lateral load resistance is provided entirely by the in-plane walls (Figure 1-b). In order to determine the mechanical properties needed for the evaluation of different numerical as well as some analytical models, our tests of the masonry assemblages were conceptually oriented on the compressive, diagonal and shear tests. Compressive and diagonal tests are namely mono-axial tests, since the resultant load which is constantly increased and has the constant inclination to the bed joint. Also the normal stresses in the bed joints are increasing with the applied load. On the other hand, the shear tests are biaxial tests and the normal stresses in the bed joints are almost constant during the testing procedure and the shear stresses are changing.



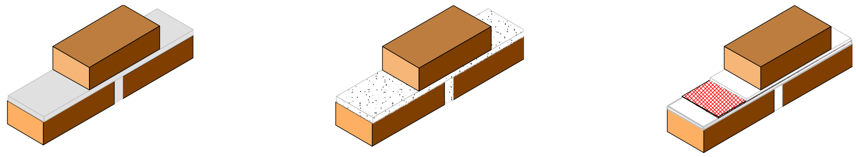
a) In and out of plane loading condition



b) Wall subjected to lateral loading

Figure 1. Wall subjected to different loading conditions

Being aware that the failure mechanisms of the masonry assemblages are highly more sensitive to the type of the unit than on the type of the mortar, the main focus of the project was to clarify the variety of different mechanisms of failure that can be expected by using just different types of mortar or their modifications. The basic mortars that were investigated were: very stiff cement mortar with the volume ratio of cement to sand = 1:4 (MIX 1), cement-lime mortar (c:l:s = 1:1:6 – MIX 2) and weak lime mortar (l:s =1:3 – MIX 3). An extension of the project was made by micro-reinforcing ordinary cement-lime mortar with 6 mm polypropylene fibres (Figure 2-b) and macro-reinforcing cement-lime mortar with glass-fibre meshes coated with resigns (5x5 cm^2 openings) and embedded within bed joint (Figure 2-c).



a) ordinary masonry – MIX 1,2 and 3
b) micro-reinforced masonry – MIX 4
c) macro-reinforced masonry – MIX 5

Figure 2. Ordinary, micro and macro-reinforced masonry

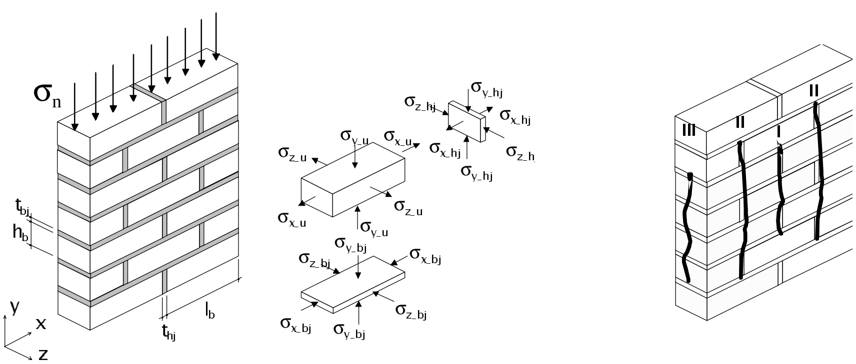
COMPRESSIVE TESTS

<u>Test set-up</u>

For each type of mortar mixes 3 specimens of the size of wallette (52x52x12 cm^3) were prepared and instrumented. The compressive tests were carried out according to the European prenorm prEN 1052-1 [prEN 1052-1, 1993]. The wallettes were tested in the servo-hydraulic testing machine Instron with the actuator capacity of 1000 kN. Each specimen was monotonically loaded by conducting the actuator displacement with a velocity of 0.3 mm/min down to the peak load-bearing capacity of the wallette.

<u>Failure modes for compressivelly loaded masonry</u>

When subjected to the compressive loading , due to the compatibility of the deformation the unit, bed and the head joint are exposed to different state of stresses (Figure 3-a). Those state of stresses are closely related to the stiffness of the unit and mortar, their dimensions and the bond between them. In a Figure 3–a, they are corresponding to a soft mortar and a stiff unit.



a) State of stresses
b) Crack pattern

Figure 3. Compressive loading

Characteristic crack pattern for masonry assemblage under compression is given in Figure 3–b. Usually the first cracks occur in the weakest points of the media, which in the case of the masonry are the head joints. Those cracks are not visible or audible and soon after their formation due to the stress concentration, the first visible crack pattern (I) occur in the middle plane of the wallette – usually passing through the unit by forming two pillars of the specimen. The second crack patterns (II) are not always so distinctive and less pronounced. Together with the first crack pattern characterize the hardening of the masonry. The failure pattern (III), which is situated on the side of the wallette, characterize the achievement of the peak load and the beginning of the softening of the masonry.

Results of testing

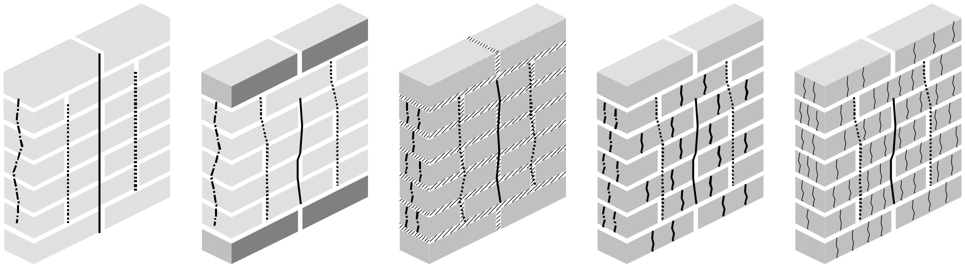
The results of testing, which are related to the compressively loaded specimens and more in details presented elsewhere [Bosiljkov, 1998], are in short presented in the following table (Table 1), where f_{jt} represent bond strength derived from the bond wrench tests, f_m represent the compressive strength of the mortar, f_b – compressive strength of the brick, f_w – compressive strenght of the masonry and E_w – modulus of elasticity of the masonry.

Table 1. Results of the compressive tests

	MIX1	MIX2	MIX3	MIX4	MIX5
f_{jt} (MPa)	0.34	0.23	0.08	0.29	0.44
f_m (MPa)	25.9	15.2	1.3	13.7	12.2
f_b (MPa)	17.4	17.4	17.4	17.4	17.4
f_w (MPa)	8.5	8.3	4.7	9.0	8.3
$E_{w,(30\%)}$(GPa)	2.54	3.20	1.04	2.59	2.77

Observed failure modes

For the specimens made with basic mortars (MIX 1,2,3), the crack pattern was as expected (Figure 3). Once the crack has started (usually in the unit) the mortar allowed the crack to grow through the height of the wallette. Sometimes (especially in the case of the cement mortar – MIX 1) these cracks appeared as a ‘cut off by blade’ (Figure 4-a). This phenomena can be explained by the fact that the MIX 1 was a very stiff mortar with both the modulus of elasticity and the strength higher than the brick. On the other hand, for the MIX 2 and especially for the MIX 3, the spalling of the units were more pronounced. For the latter the crushing of the mortar from the joints almost from the very beginning of the loading was very intensive (Figure 4-c).



a) MIX 1 b) MIX 2 c) MIX 3 d) MIX 4 e) MIX 5

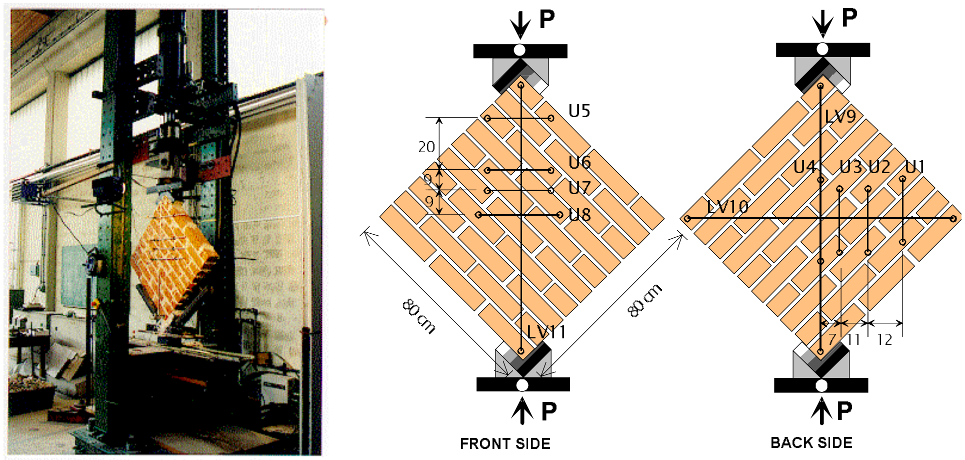
Figure 4. Influence of the type of the mortar on the failure modes

For the masonry made from modified mortars their first cracks (I) usually occurred in the bricks in the plane of the wallette. The first crack pattern consisted of randomly placed cracks within the plane of the wallette. The failure crack, which occurred on the side of the wallette (III), was as a previous one (I), very poorly connected through the height of the wallette. These second mechanisms were provoked mainly due to the presence of either polypropylene fibres or glass mesh which have prevented propagation of the crack through the height of the specimens and thus induced the high tensile strains within the units. The high tensile strains were the main reason for the dense crack pattern of the units after the failure of the specimens, which were more dense for the macro-reinforced masonry (Figure 4-e) in comparison to the micro-reinforced masonry (Figure 4-d).

DIAGONAL TESTS

<u>Test set-up</u>

For each type of mortar mixes at least 4 specimens of the size of panel (80x80x12 cm^3) were prepared and instrumented (Figure 5-b). The panels were tested with servo-hydraulic jack Instron with the capacity of 250 kN (Figure 5-a). Each specimen was monotonically loaded by conducting the actuator displacement with a velocity of 0.3 mm/min up to the peak load-bearing capacity of the panel.

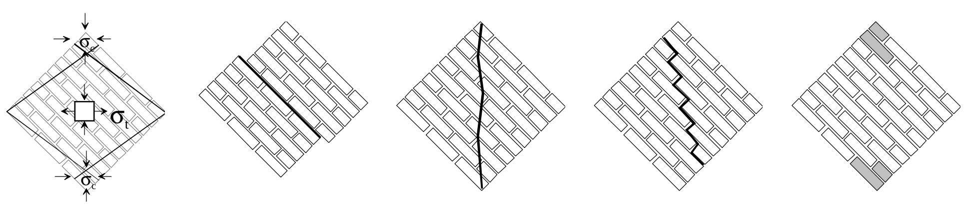


a) Disposition of the testing b) Disposition of the measuring devices

Figure 5. Diagonal tests of the panels

<u>Failure modes for diagonally loaded masonry</u>

When subjected to the diagonal forces the state of stresses are commonly distributed as it is presented in a Figure 6–a. Characteristic for the diagonal testing is that by increasing the shear stresses in the bed joints, the normal component of the stresses of the bed joints are increasing as well. Thus the failure mechanisms of the specimens can be due to the failure of the one component of the masonry assemblage (brick, mortar or their junction) or due to their most unfavorable combinations (Figure 6-b to e).



a) Stresses b) Sliding c) Splitting d) Combined e) Crushing

Figure 6. Diagonal loading

<u>Results of testing</u>

There are different interpretation of the results among researchers, depending from the failure modes [Bosiljkov, 2000]. Especially the calculation of the tensile strength of the masonry can be a matter of discussion. In our calculation of the tensile strength of the masonry, we have chosen Frocht's model [Yokel, 1976]:

$$f_t = 0.52\frac{P}{l \cdot t} \qquad (1)$$

where f_t is diagonal tensile strength, l represent the high of the panel, and t represent the thickness of the panel.

Table 2. Results of the diagonal tests

	MIX1	MIX2	MIX3	MIX4	MIX5
f_{jt} (MPa)	0.28	0.20	0.17	0.31	0.35
f_m (MPa)	21.1	11.7	1.9	10.7	12.6
f_t (MPa)	0.42	0.38	0.10	0.41	0.67

Observed failure modes

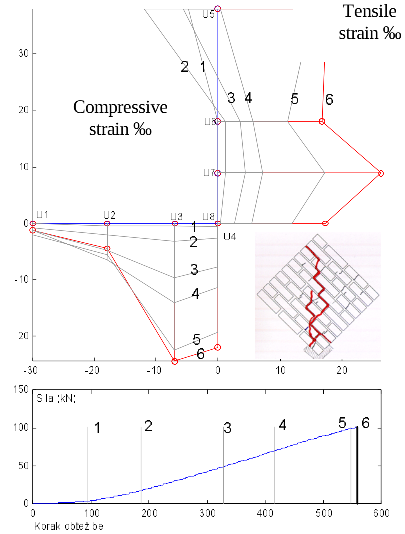
Failure modes of diagonally loaded masonry panels in depending from the type of the mortar mixture are represented in Table 3. Due to the right choice of the dimensions of the loading fixture at the corners, we avoided the premature failure of the specimens caused by crushing of the units within the loading shoes.

Table 3. Failure modes of diagonally loaded masonry panels (by number of specimens)

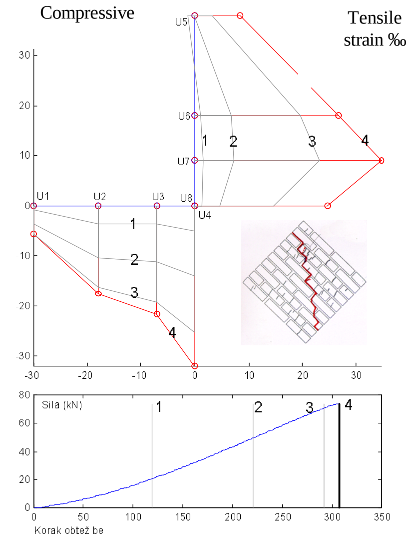
	MIX 1	MIX 2	MIX 3	MIX 4	MIX 5
Shear failure	0	1	1	0	0
Combined failure	2	3	4	4	3
Splitting failure	2	0	0	0	1

For the masonry made with cement mortar (MIX 1) the most predominant modes of failure were splitting and combined one. This can be again explained by the very stiff mortar and a good bond, which enable masonry to act almost as an isotropic media. On the other hand both other ordinary mixtures (MIX 2 and 3) had prevailing combined failure. But their behavior was significantly different. The MIX 2 had (as well as MIX 1) very fragile failure. The behavior of the MIX 3 was very ductile one, which was primarily due to the progress of the crack pattern within the mortar joints (tensile failure of the mortar) and not in the brick-mortar junctions as it was the case with other mortar mixes. The most ductile behavior was observed in the case of MIX 5, which was expected due to the reinforcement of the bed joint. It is worth to mention, that not in a single case of failure, we have get spalling of the brick.

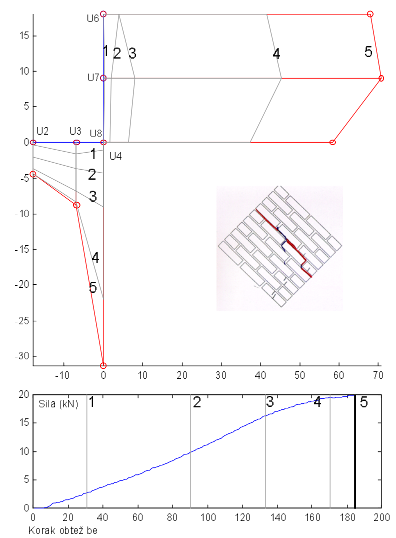
Through extensive measurements of the strain distribution provided by the large number of measurement devices (Figure 5-b) within the plane of the specimens, we have managed also to capture, some of the typical strain distributions depending from the type of the failure mechanisms (Figure 7-a to c). Note, that for the macro-reinforced masonry, after the formation of the crack pattern (Figure 7-d, 4^{th} step), the complete resistance of the specimen is relying solely on the tensile strength of the reinforcement and as a consequence an extensive concentration of the strains occurs in the middle section of the specimen.



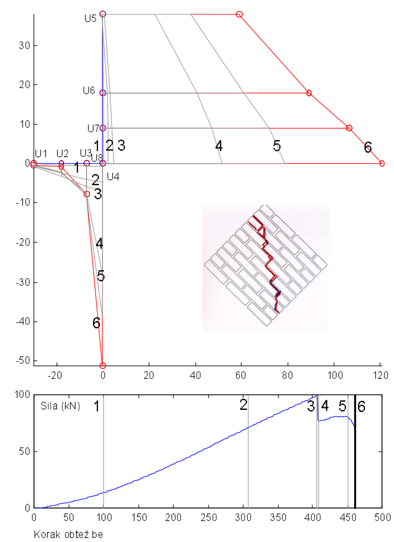
a) Splitting failure mechanisms – MIX 1



b) Combined failure mechanism – MIX 4



c) Shear failure mechanisms – MIX 3



d) Combined failure – macro-reinforced masonry – MIX 5

Figure 7. Strain distribution (‰) through different steps of loading (kN)

SHEAR TESTS

The shear tests are conceptually divided into two parts, where in the first part the level of constant vertical load (pre-compression) was chosen as a 1/6 of the mean compressive strength of the masonry depending from the chosen type of mortar (see Table 1). The second part of testing that is related solely on testing of masonry made with MIX 2 under different levels of pre-compression.

Test set-up

For each mortar mixture at least 3 masonry panels with dimensions 95x140 cm^2 and thickness of 12 cm were prepared. The shear tests of masonry panels were performed on the device that was designed for the tests of shear bearing capacity of cantilever walls with different dimensions, where rotation and horizontal displacement are released on the lower edge of the panel (Figure 8). The vertical force was applied first and the horizontal cyclic displacement controlled load was then applied. The design of the testing machine enables the constant vertical load during the testing.

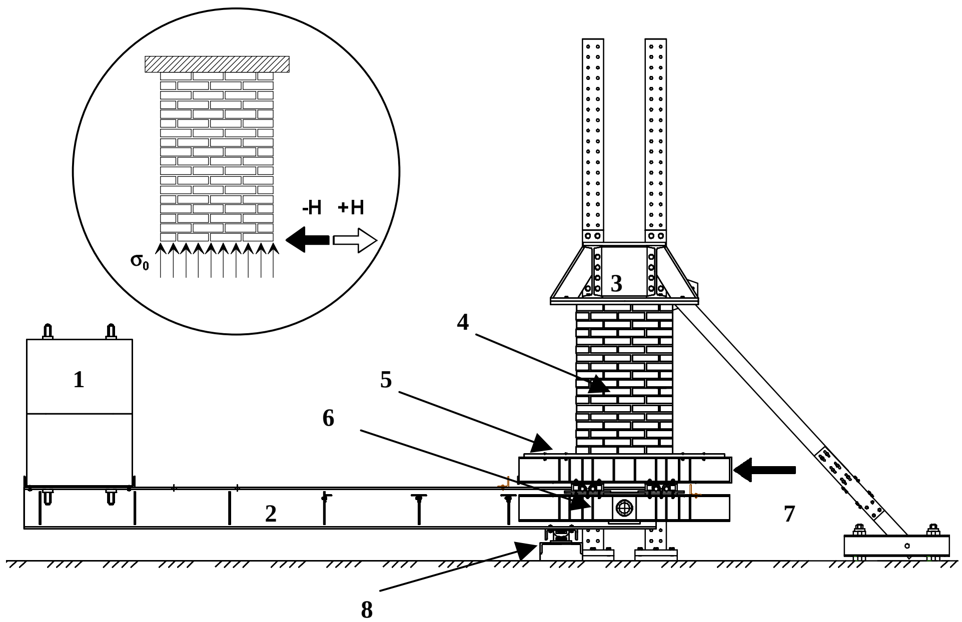


Figure 8. Test set-up for shear testing

The testing sample (4) in the testing machine presents a cantilever turned upside down inserted into a frame structure (3) at the upper edge, and resting on a trolley (5), through which the vertical and horizontal loads (7) are introduced into it, on the lower edge. Constant vertical load is applied, with weight (1), which act through a lever (2) as a vertical load of the testing sample (4). The magnitude of the vertical load can be controlled either through the weight (1) or changing the position of the hinge (8) of the lever arm. The free horizontal displacement and the rotation of the lower edge of the cantilever are provided by a hinge mechanism, which consists of a free rotating beam (6) and a trolley (5), which can freely slide over the rotating beam (6). The horizontal load (7) was applied as a cyclic loading with prescribed time history through servo-hydraulic actuator with the capacity of ±250 kN.

Failure modes for laterally loaded masonry

Failure modes for laterally loaded masony depends from the size of the specimen, boundary conditions and the level of pre-compression. According to the observation and measurements we have defined some basic crack patterns which can occur for the case of cantilever laterally loaded shear panels (Figure 9).

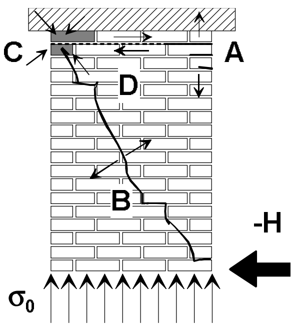


Figure 9. Crack patterns

A - Flexural cracking (H_f) which does not involve collapse but for moderate level of precompression it can provoke a variation of the stiffness and the formation of hysteresis loops. It can be stabilised in one single course or it can be spread through the height of the specimen.

B – Shear diagonal cracking (H_s), which for moderate level of precompression does not always present an ultimate limit state if only one direction of loading is considered. For higher level of precompression it can represent a limit state.

C – Rocking (toe crushing), can represent a limit state but for moderate level of precompression. Together with the shear crack it can provoke an explosive failure of the specimen.

D – Sliding , which can represent a limit state for very low level of presompression.

Shear tests on the same relative level of precompression

Results of testing for different mortar mixtures under the same level of pre-compression (~1/6 f_w), which are in more details presented elsewhere [Bosiljkov, 2000] are in short presented in Table 4.

Table 4. Horizontal load that correspond to different crack pattern

	MIX 1	MIX 2	MIX 3	MIX 4	MIX 5
f_{jt} (MPa)	0.43	0.34	0.13	0.40	0.37
f_m (MPa)	13.9	9.5	1.1	7.7	9.6
H_f (kN)	59.5	33.5	23.3	47.2	51.5
H_s (kN)	65.6	67.5	27.9	81.1	67.6
H_{max} (kN)	96.6	71.6	44.6	87.8	92.9

On the basis of the test results of shear tests and the observations and monitoring of the developing of the crack pattern we have tried to summarized some basic conclusions of the behavior of shear panels in dependence from the used mortar mixtures.

MIX 1 – Flexural cracking at this level of pre-compression did not provoke the change of stiffness of the specimen. The flexural crack was situated in the first course and after its occurrence it was very quickly localised (did not spread through the height or length of the specimen). The shear crack usually started together with beginning of the rocking, which after a while provoke the failure of the specimen with widely opened shear crack and crushed masonry on both ends of the diagonal shear crack. The direction of the shear crack was in the direction of the main compressive stresses with the cracks, which were passed both through the units and mortar. The mortar in the head joints had a stiff behavior, so that the cracks, which were passed through head joints, were situated on the brick-mortar junction.

MIX 2 –The occurrence of the flexural cracks did not change the stiffness of the panels. But for a difference from the previous case flexural cracks were not localized. With increasing loading they were spreading through the length of the specimen and thus changing the stiffness of the specimen. Shear cracks were predominantly opened before the beginning of rocking. The shape of the shear crack was in the middle third of the specimen linear and it passed both through the units and mortar in the direction of the main compressive stresses. Towards the corner it has turned into zigzag shape and has passed mainly through unit-mortar junction. Behavior of the mortar within head joint was the same as for previous case.

MIX 3 – Very important for lime based masonry was the crushing of the mortar from the joints, which were presented all through the testing. The appearance of the flexural cracks has induced the change of stiffness of the panels. Flexural cracks were not localised and they were spread both through the length and the height of the specimens. The shape of the shear cracks were zigzag all through the specimen, but for a difference from previous two cases the orientation of the cracks through the units were not in the direction of the main compressive stresses but they were mostly concentrated through the units in that direction. Low tensile strength and soft behaviour of the mortar within the joints (both bed and head joint) have provoke that the cracks which were passed through joints were not situated in the brick-mortar junction but within the joints themselves.

MIX 4 – Overall behaviour of those panels were very compact. For a difference from MIX 1 and MIX 2, the first flexural cracks have occurred in the second course of the panels. With the increasing the load the flexural cracks were spreading through the length of the specimens and thus provoke the changes in the stiffness of the specimen. Despite the ductile behaviour of the micro-reinforce mortar, which was observed during the Bond Wrench tests such behaviour, were not noticeable during the opening of the flexural cracks. The failure of the specimens was predominantly because of rocking. The shear cracks, which were occurred shortly before the rocking where oriented in the direction of the main compressive stresses but without, distinguish one. Instead of it there were many closely spaced shear cracks passing both through the unit and the bed joints in the direction of the main compressive stresses. After the failure of the specimens it was observed that the head joints were not properly fulfilled due to the problems of workability of fresh micro-reinforced mortar.

MIX 5 – Overall behaviour of masonry panels made from macro-reinforced cement-lime mortar were up to the beginning of opening of shear cracks almost the same as it was observed in the case for the reference mortar MIX 2. It is worth to mention that the shear cracks opened in both cases at the same level of lateral force. But in the case of macro-reinforced masonry panels the shape of the shear cracks were much more alike as it was observed for the MIX 4. It is obvious that the presence of either polypropylene fibres or glass mesh which have prevented propagation of the crack through the height of the specimens are inducing the higher tensile strains within the units itself which have for the consequences the dense crack pattern within the units. The macro-reinforced mortar has provoked much more ductile behaviour of the specimens with strong dissipation of the energy. The beginning of the rocking mechanisms did not provoke the failure of the specimens. The failure was a consequence of the softening of the masonry material within the middle third of the specimen, which sometimes provoke an explosive collapse.

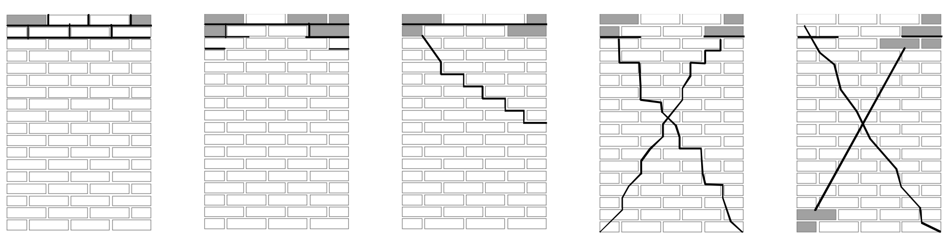
Shear tests on the different levels of precompression σ_0

Results of testing for MIX 2 different levels of pre-compression, which are in more details presented elsewhere [Bosiljkov, 2000] are in short presented in Table 5.

Table 5. Horizontal load that correspond to different level of pre-compression for MIX 2

σ_0 (MPa)	0.686	1.0	1.5	2.0	4.0
number of specimens	1	1	1	3	3
H_f (kN)	13.8	24.7	33.1	33.5	79.8
H_s (kN)	-	-	-	67.5	85.9
H_{max} (kN)	27.0	49.9	70.6	71.6	115.8

As it can be seen from (Figure 10), the observed failure modes were strongly influenced with the different level of σ_0 .



0.686 MPa 1.0 MPa 1.5 MPa 2.0 MPa 4.0 MPa

Figure 10. Failure mechanisms for MIX 2 for different levels of pre-compression σ_0

For the lowest level of σ_0 , despite the beginning of rocking behavior in the upper edge of the specimen the failure of the specimen was due to sliding (Figure 10-a). With further increasing of σ_0 we have got a typical rocking failure, with crushed masonry in the upper corners of the specimen (Figure 10-b). The first shear crack pattern we have got under the σ_0 =1.5 MPa, and it was anticipated due to the crushed masonry in the upper corners of the specimen (Figure 10-c). At the level of σ_0 = 2.0 MPa, we have got a shear failure, where the shear cracks began in the middle section of the specimen (Figure 10-d). Under the highest σ_0 we have got a very fragile shear failure, with the almost simultaneously opening of the shear crack and crushing of the masonry in the upper and lower corners of the specimen. The shape of the shear crack was a straight one (Figure 10-e).

CONCLUSIONS

The following conclusions are based solely on phenomenological analysis of different failure modes of the masonry assemblages depending from the type of testing and type of mortar.

By using very stiff mortar (MIX 1) masonry assemblage act as an almost isotropic media and thus the failure cracks pass both through the unit and the mortar in direction of the main principal stresses. On the other hand by using very soft mortar (MIX 3) the masonry assemblage act almost from the beginning of loading as composite material and is strongly affected by the very low level of tensile strength of the lime mortar. Very important for lime based masonry is the crushing of the mortar from the joints, which is presented all through the testing, no matter which type of loading is applied on.

The most unfavorable type of testing for masonry assemblage is diagonal testing, since the failure modes is attributed to the most unfavorable combination of the weakest properties of its components (brick, mortar and their junction), which is predominantly the brick-mortar junction. This type of testing is not suitable for testing reinforced masonry. The failure modes derived through diagonal and lateral testing for reinforced masonry are significantly different.

The results presented herein which are statistically evaluated represent a valuable set of data which can be used in creation of some new models as well as for verification and calibration of some existing ones for prediction of load-bearing capacity of masonry assemblage.

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