OPTIMIZATION OF CONTEMPORARY BLOCKWORK MASONRY FOR SEISMIC REGIONS – STEP BY STEP APPROACH

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SUMMARY

Evolution of masonry construction has been in the past a casual process rather than one preceded by scientific or engineering analysis and thus some of the innovative construction techniques were in the long term incomparably more harmful to the masonry construction than (in the short term) they provided benefits by the means of increasing the performance and economy of masonry. This problem is particularly pronounced and can be closely correlated to the development and optimisation of the vertically perforated lightweight clay blocks with very thin shells and webs, with different types of the execution of the perpend (head) joint and different pattern of perforations and their application in the earthquake prone areas. All recent improvements of the blocks, though not significantly influencing the collapse mechanisms of the masonry when subjected to compressive gravity loads, can govern the overall brittle behaviour of masonry under seismic conditions as they reduce robustness of masonry units and thus overall behaviour of masonry walls as structural elements. In order to identify the governing parameters of this problem, experimental and numerical research of the behaviour of blockwork masonry due to seismic loading have been carried out. Following experimental results of testing series of blockwork masonry walls with aspect ratios of h/b=1.5 and b/h=1.5 in seismic conditions with different levels of precompression, numerical investigation was carried out. For this purpose a simplified micro-modelling technique was applied, where blocks and joints were modelled separately. Chosen strategy was effective and it clarified the critical state of stresses in the blocks at the attained maximum resistance as well as at the failure of masonry specimens due to the lateral loading. This combined step-by-step approach is the only way towards planned 3D numerical study for the optimisation of the shape of the units and perforation patterns of hollow clay masonry units for contemporary masonry buildings in earthquake prone areas.

INTRODUCTION

Environmental issues such are energy savings and global warming are two governing factors of nowadays construction market. For masonry industry this may reflect as new demands for better thermal properties of masonry material. In contemporary masonry construction, this could be easily achieved by introducing multi-leaf masonry walls with or without additional thermal layers and appropriate detailing. However, globalizations of the market and market competition with new materials have already imposed demands in faster and cheaper technologies of construction. To achieve these demands masonry industry had to lower building costs and to improve thermal properties of the units by making more porous ceramic material and introducing highly perforated large masonry units with different levels of perforations. At first, these new types of units were built with highly deformable light weight

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mortar in order to reduce thermal conductivity of new masonry to even higher extent. Than, in order to facilitate building process, mortar in masonry first disappeared from perpend joints, and at the very end it has been completely removed and replaced by glues and nets in the bed joints. All these improvements of the blocks, though not significantly influencing the collapse mechanisms of the masonry when subjected to compressive gravity loads (may be also a debating issue), had definitely negative impact on one other environmental demand (back to environmental issue) which is earthquake resistance. Due to reduced overlapping of units (for this type of units, their width is larger than their length) seismic resistance of contemporary masonry due to the new improvements was significantly reduced. Hence, due to slender shells and webs, robustness of masonry units was also reduced and consequently overall behaviour of masonry walls as structural elements became more brittle. Following all aforementioned, two main questions were imposed for engineering scientists: what is the real performance of contemporary blockwork masonry in seismic regions and what should be the main guidelines in effective optimization of the shape of units.

RESEARCH PROGRAM

Thermal and seismic requirements are at the moment key factors that determine innovative process for contemporary blockwork masonry. They both reflect essential requirements for buildings according to EU regulations (Mechanical Resistance and Stability, Safety in Fire, Hygiene, Health and the Environment, Safety in Use, Protection Against Noise and Energy economy and Heat Retention). However, they both are also very antagonistic. In order to encompass this problem and to ease already imposed severe restrictions for blockwork in seismic codes [Eurocode 8 2003] both experimental and numerical work on blockwork masonry has to be done. In this process, an iterative rather than a deterministic approach should be applied. In order to do so, in Slovenia where the seismic design is the governing factor for contemporary masonry structures, a comprehensive research program was carried out. In this paper, the following steps from the complete research programme are presented:

- Testing of narrow wall specimens made from blocks with the same perforation patterns but differing types of perpend joints.
- Testing large walls made from same blocks, but with different types of perpend joints.
- Accompanying tests on blocks and mortars according to EN (European Normative).
- Numerical analysis of narrow and large wall specimens.

EXPERIMENTAL PROGRAM

Traditionally, only the construction of masonry walls with fully grouted (filled) perpend joints has been allowed in seismic zones. By definition given in EC 6 [Eurocode 6 2003], perpend joints can be considered to be filled if mortar is provided to the full height of the joint over a minimum of 40 % of the width of the unit. In the new draft of EC 8 [Eurocode 8 2003], however, a note states that the National Annex will select which of the three classes of perpend joints (fully grouted, ungrouted, and ungrouted with mechanical interlocking between the units) will be allowed to be used in a country. Since not much reliable experimental evidence exists to prove that from the point of seismic design these technologies are suitable and acceptable, a first step was to identify the influence of different systems of filling the perpend joints on the seismic behaviour of the walls. Four different types of filling the perpend joints have been studied (Figure 1):

- Fully filled perpend joint (walls series BN - reference walls),
- Dry, unfilled perpend joint (walls series BG),

- Partly filled perpend joints with mortar in the pockets (walls series BP),
- Dry, grove and tongue perpend joint (walls series BZ).

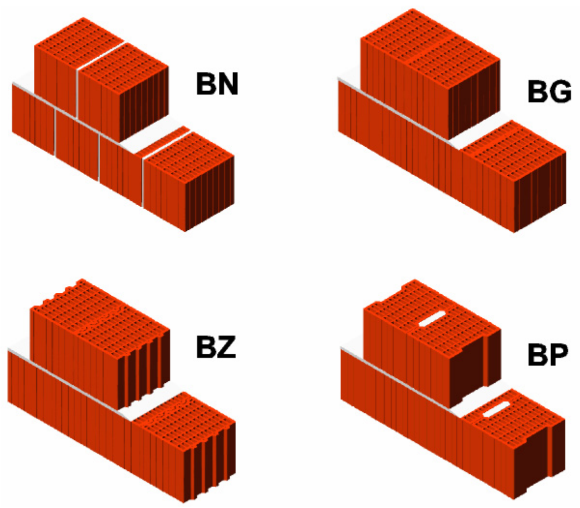

Figure 1. Types of units and perpend joints.

Accompanying Tests on Blocks and Mortars

The following characteristics of masonry units have been determined prior to testing of masonry walls:

- compressive strength of units according to SIST EN 772-1,
- compressive strength of units parallel to bed joints according to SIST EN 772-1,
- water absorption according to JUS B.D8.011,
- dimensions of units according to SIST EN 772-16, and
- net volume and percentage of voids according to SIST EN 772-3.

The dimensions of masonry units (all can be classified into Group 2 units according to Table 3.1 of prEN 1996-1 [Eurocode 6 2003]) for each series of wall specimens as well as the values of compressive strength and porosity are given in Table 1.

Table 1. Parameters of masonry units

Wall series	Length	Width	Height	Percentage of holes	Thickness of shells	Thickness of webs	Combined thickness of webs and shells - transversal	Combined thickness of webs and shells - longitudinal	Compressive strength f_b	Compressive strength parallel to bed joints $f_{b,h}$	Water absorbtion ratio
	(mm)	(mm)	(mm)	(%)	(mm)	(mm)	(%) of width	(%) of length	(MPa)	(MPa)	(%)
BN	245	298	237	50	13.0	7.8	23.3	40.2	10.0	4.4	13.7
BG	245	298	237	50	13.0	7.8	23.3	40.2	10.0	4.4	13.7
BP	245	299	236	52	11.4	7.6	21.7	38.2	11.9	6.2	12.9
BZ	243	298	235	48	13.2	7.8	23.7	40.3	15.1	4.8	12.9

Lime-cement general purpose pre-mixed mortar has been used for the construction of wall specimens of all tested series BN, BG, BP and BZ. Samples of mortar have been taken from each batch of mortar (3 prisms 4 x 4 x 16 cm) during the construction of wall specimens. Compressive and flexural strength of the mortar have been determined according to SIST EN 1015 after the testing of the respective series wall specimens. The number of samples and average values of mortar strength and density are given in Table 2 for each series of wall specimens.

Table 2. Parameters of mortars

Wall series	Age	Compressive strength	Flexural strength	Density
	(days)	(MPa)	(MPa)	(kg/dm^3)
BN	156	5.0	1.6	1.59
BG	153	6.3	1.7	1.65
BP	154	5.3	1.6	1.63
BZ	148	3.3	1.1	1.57

Test on Walls With Aspect Ratio of *h/b=1.5*

The walls to be tested have been designed to fail in shear. The size of masonry walls were 100x150x30 cm (*b* x *h* x *t*). Grade M5 general purpose mortar has been used for the construction of specimens, built on r.c. foundation blocks and tested as shown in Figure 2. Three specimens of each type have been tested at working stress/compressive strength ratio (σ_o/f) ranging from 0.14 to 0.29 (Bosiljkov et al 2004).

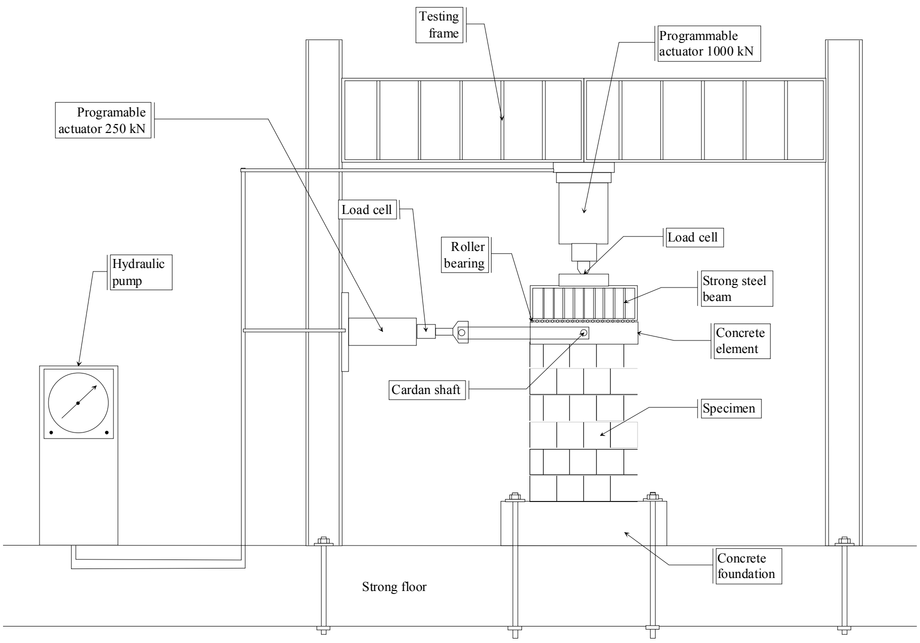

Figure 2. Lateral resistance test set-up for walls with aspect ratios of *h/b=1.5*.

Regardless of the level of applied precompression, all walls failed in shear, as expected. However, brittle local failure of units, i.e. buckling and crushing of thin shells and webs was the predominant phenomenon which determined the failure mode. Test results are summarized in Table 3, where for each wall the average values of lateral force *H* and

displacement *d*, measured at the first displacement cycle of the respective displacement amplitude in positive and negative direction, are given for each of the defined limit states.

Table 3. Test results: lateral load, displacement and rotation angle at shear cracking, maximum resistance and ultimate state limits

Specimen	Shear cracking limit			Maximum resistance			Ultimate state limit		
	H_{cr}	d_{cr}	θ_{cr}	H_{max}	d_{Hmax}	θ_{Hmax}	H_{du}	d_u	θ_u
	(kN)	(mm)	(%)	(kN)	(mm)	(%)	(kN)	(mm)	(%)
BNL1	54.53	7.19	0.50	54.98	9.44	0.66	49.29	28.74	1.99
BNL2	96.17	5.70	0.50	98.83	5.70	0.40	40.00	12.01	0.83
BNL3	54.93	8.56	0.99	56.22	9.54	0.66	24.97	11.12	0.77
BNL4	112.00	9.65	0.83	112.00	9.65	0.33	20.10	16.92	1.16
BNL5	109.36	7.18	0.50	109.36	7.18	0.49	79.11	19.68	1.36
BNL6	63.13	8.66	1.99	65.92	9.74	0.67	50.11	33.36	2.31
BGL1	101.73	9.84	0.66	101.73	9.84	0.30	63.52	12.40	0.83
BGL2	102.92	12.50	0.83	102.92	12.50	0.83	40.18	14.96	0.99
BGL3	93.84	14.96	0.00	93.84	14.96	0.00	93.64	14.96	0.00
BPL1	106.33	6.00	0.40	106.33	6.00	0.40	29.63	12.50	0.83
BPL2	107.73	6.00	0.40	110.34	7.48	0.50	60.00	15.00	0.99
BPL3	104.50	7.48	0.50	110.98	9.65	0.60	57.00	12.50	0.83
BZL1	96.40	5.02	0.33	100.36	5.31	0.35	12.53	7.38	0.49
BZL2	96.70	5.02	0.33	103.76	7.48	0.50	64.97	12.50	0.83
BZL3	91.00	2.46	0.17	102.39	5.71	0.38	27.32	12.50	0.83

Rotation angle θ has been calculated as a ratio between the measured displacement at each of the limit states (d_{cr}, d_{Hmax} and d_u, respectively) and the height of the wall *h*. The lateral resistance degradation and deformation capacity for each series of walls is analyzed in Table 4. Average values are given for each of the tested wall types at each precompression level. The ratios between the measured values of lateral resistance and rotation angle at the attained maximum resistance, shear cracking and ultimate state limits (resistance indicators H_{cr}/H_{max} and H_{du}/H_{max} and displacement capacity indicators $\theta_{cr}/\theta_{Hmax}$, θ_u/θ_{Hmax} and θ_u/θ_{cr}), are given, respectively.

Table 4. Lateral resistance and displacement capacity of the tested types of walls at characteristic limit states

Series	f^* (MPa)	σ_o/f	H_{cr}/H_{max}	H_{du}/H_{max}	$\theta_{cr}/\theta_{Hmax}$	θ_u/θ_{Hmax}	θ_u/θ_{cr}
BNL	4.13	0.15	0.97	0.70	0.85	2.55	3.00
BNL	4.13	0.29	0.98	0.44	1.00	2.15	2.15
BGL	4.31	0.28	1.00	0.66	1.00	1.13	1.13
BPL	6.28	0.19	0.97	0.45	0.84	1.73	2.06
BZL	6.24	0.19	0.93	0.34	0.68	1.75	2.57

f^* - experimental average value of compressive strength of masonry according to EN 1052-1

Following the tests on narrow walls with aspect ratios of $h/b=1.5$ built with different execution of perpend joints, the main conclusions were summarized as follows:

- The ratio between the lateral load acting on the walls at the initiation of diagonal cracking and maximum resistance H_{cr}/H_{max} is close to 1.00. This means that the occurrence of diagonal shear cracks in masonry characterizes the attainment of lateral resistance of the walls. According to previous experimental evidence, the value of H_{cr}/H_{max} ratio of about 0.7 - 0.8 should have been expected for URM brickwork (Bosiljkov et al., 2003) meaning

that at the occurrence of the first diagonal cracks the lateral resistance of the wall is not yet attained. Further increase of the imposed lateral displacements is usually needed and opening of diagonal cracks occurs before the attainment of maximum resistance of the wall. In the particular case studied, however, once the diagonal cracks occurred, the resistance of the walls at increased imposed displacements amplitudes started to degrade.

- Relatively large resistance degradation, i.e. small values of H_{du}/H_{max} ratio, has been observed in all cases, despite the relatively small ultimate displacements.
- The displacement capacity of the tested walls, expressed by displacement capacity indicators with regard to rotation of the walls at the attained maximum resistance and ultimate rotation just before collapse θ_u/θ_{Hmax}, is small, below the expected values for URM walls.

Test on Walls With Aspect Ratio of *b/h=1.5*

There is no standardized way of testing masonry in seismic conditions [Bosiljkov et. al 2003]. Whether the size of the specimen may lead to high compressive stresses in the units and consequently to premature failure of masonry specimen, was investigated during the second phase of the research project. In order to determine the influence of the size of the specimens (geometry aspect ratio) on the mechanism of seismic behaviour and subsequent evaluation of seismic resistance parameters, three specimens (BNW1, BNW2 and BNW3) with dimensions of 250x175x30 cm (*b/h≈1.5*), made of the units of series BN (fully filled vertical joints), were tested. Class M5 mortar was used for the construction of specimens. Each specimen was tested under a different level of precompression (stress/compressive strength ratio of 14, 22 and 30% respectively) in a modified test set-up (Figure 3).

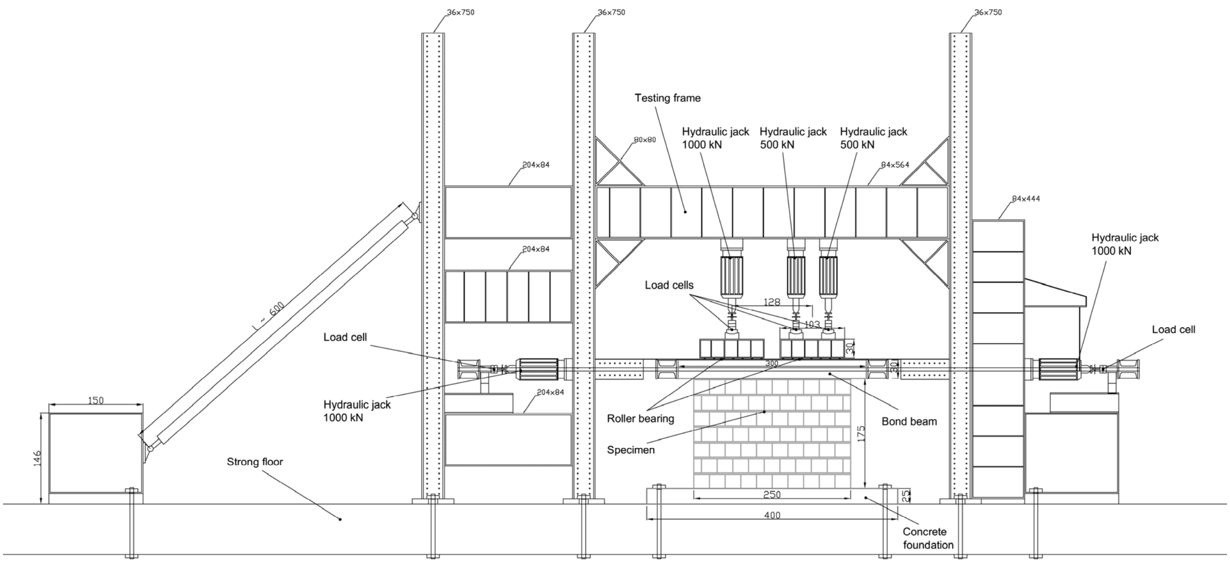

Figure 3. Test set-up for testing the masonry walls with aspect ratios of *b/h=1.5*

All masonry specimens were tested as vertical cantilevers fixed to the foundation block. The walls were subjected to a constant vertical load and a cyclically acting horizontal loading, acting on the RC bond-beams placed on the top of the specimens. Horizontal load was applied in the form of programmed displacements (the same programme of displacement as in the 1st

part of the project), cyclically imposed in both directions, with step-wise increased amplitudes up until the collapse of the specimens. At each displacement amplitude, the loading was repeated three times. Main results of the tests are summarized in Tables 5 and 6.

Table 5. Test results: lateral load, displacement and rotation angle at toe and shear cracking, maximum resistance and ultimate state limits.

	Compressed toe cracking limit			Shear cracking limit			Maximum resistance			Ultimate resistance limit		
	H_r	d_r	θ_r	H_s	d_s	θ_s	H_{max}	d_{Hmax}	θ_{Hmax}	H_u	d_u	θ_u
	(kN)	(mm)	(%)	(kN)	(mm)	(%)	(kN)	(mm)	(%)	(kN)	(mm)	(%)
BNW1	279.9	5.16	0.30	277.0	5.01	0.29	285.1	4.98	0.28	220.4	31.23	1.78
BNW2	434.1	3.54	0.20	447.3	3.31	0.19	467.9	5.69	0.32	388.4	11.38	0.65
BNW3	372.9	6.27	0.36	315.6	2.29	0.13	384.9	5.52	0.32	284.6	15.19	0.87

Table 6. Lateral resistance and displacement capacity of the tested types of walls at characteristic limit states (average values)

Spec.	σ_0 / f_k	H_r/H_{max}	H_s/H_{max}	H_u/H_{max}	θ_r/θ_{Hmax}	θ_s/θ_{Hmax}	θ_u/θ_{Hmax}	θ_u/θ_s
BNW1*	14%	0.98	0.97	0.77	1.04	1.01	6.28	6.23
BNW2	29%	0.93	0.96	0.83	0.62	0.58	2.00	3.44
BNW3	22%	0.97	0.82	0.74	1.14	0.42	2.75	6.63

* failure due to rocking

Characteristic failure patterns for both aspect ratios of masonry walls are presented in Figure 4.


a)


b)

Figure 4. Ultimate limit state for walls with aspect ratio of *h/b=1.5* (a) and *b/h=1.5* (b)

The following general observations regarding the resistance and behaviour of the walls subjected to constant vertical and cyclic lateral load were made as follows:

- Different relative levels of precompression had significant influence on the crack development and behaviour of laterally loaded walls. The ratio between lateral load acting on the walls at the initiation of cracks in compressed toes is for all relative levels of compression H_r/H_{max} close to 1.00. This would imply that all the specimens fail due to the rocking with intact masonry in the middle height of the specimen. For specimens BNW2 and BNW3 this was not the case and both of them, after the formation of shear cracks and

cracks in compressed toes failed with characteristic diagonally oriented shear crack pattern. Thus it has to be concluded that for this method of evaluation of the shear resistance of laterally loaded masonry specimens the proposed test set-up has been appropriate designed.

- The ratio between the lateral load acting on the walls at the occurrence of limit states of toe and shear cracking and maximum resistance (H_r/H_{max}, H_s/H_{max}) for both specimen BNW1 and BNW2 were close to 1.00, although their development of shear cracks differed significantly. For BNW1 specimen, the shear cracks were oriented along the perpend joints due to the combined effect of lateral loading and uplifting of the specimen. For BNW2 specimen, shear cracking was oriented mainly through the units.
- For BNW3 specimen H_s/H_{max} ratio was much lower (0.82), with shear cracks oriented both along the perpend joints and passing through the units. Since the failure of the specimen was due to the characteristic diagonally oriented shear crack pattern, it was decided that the relative level of precompression σ_0/f_k = 22%, may be required for the evaluation of the influence of the resistance of laterally loaded cantilever masonry walls with aspect ratio of *b/h* of 1.5.
- Once again, as it has been already noticed in the 1st part of research project, the displacement capacity of the tested walls, expressed by displacement capacity indicators with regard to rotation of the walls at the attained maximum resistance and ultimate rotation just before collapse θ_u/θ_{Hmax}, is small, below the expected values for URM walls.

NUMERICAL MODELLING

Analysis of the relative levels of compression in critical cross sections of tested walls reveals that analytically calculated compressive stresses for these specimens that failed in shear mode did not exceed 45% of calculated characteristic compressive strength of masonry according to EC6 provisions. Thus, it may be concluded that the falling offs of the shells and crushing of the units were not due to the compressive failure of the units but due to the combined actions of lateral and compressive loading and consequently biaxial state of stresses within the masonry. Since most of the falling offs of the shells occurred in softening range, in order to evaluate real state of stresses in the units it was necessary to perform numerical investigation. For this purpose a simplified micro-modelling technique adopted from Shing et al. 1994 [Bosiljkov, 2004] was applied. Each unit was modelled using two four-noded plane stress smeared crack elements, while bed joints and perpend joints were modelled with four-noded interface elements capable of simulating the initiation and propagation of interface fracture under combined normal and shear stresses in both the tension-shear and compression-shear regions.

First results of the numerical analysis revealed that the chosen strategy was efficient in modelling blockwork masonry (Figure 5). Analysis of compressive stresses in the non-linear range of experimental results (Figure 6) revealed that for both large and narrow specimens compressive stresses in the compressed toe are much higher than presumed by elastic analysis (exceeded compressive strength of masonry). However for both types of specimen these level of stresses are almost the same (around 6 MPa), implying that the size of the specimen was not deciding factor for the brittle behaviour of the specimens due to the shear loading. It should be noted also, that even though the compressive stresses in toes have been exceeded at maximum resistance, considerable displacements in the softening region have been obtained

prior to ultimate failure of the specimen. This was mainly due to the uplifting of the blocks along perpend joints and crushing the units in the corner of the specimen. Ultimate resistance was attained due to the failure in the middle height of the specimen under combined state of shear and compressive stresses.

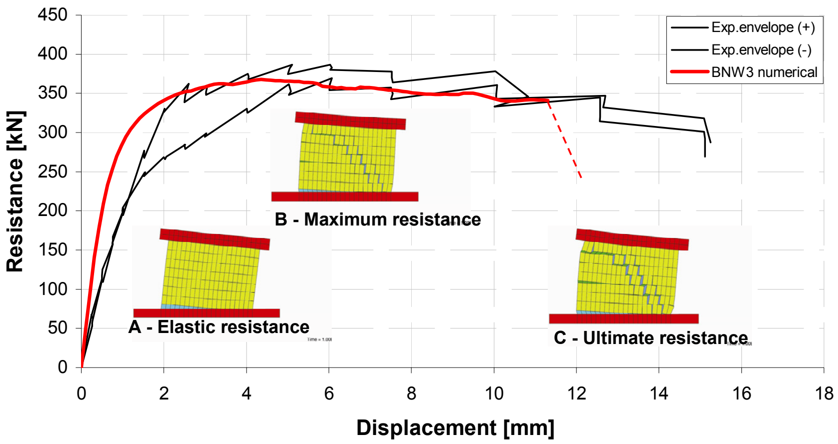

Figure 5. Comparison of numerical results with experimentally obtained envelope of hysteresis loops, measured during the shear resistance tests of wall type BNW3.

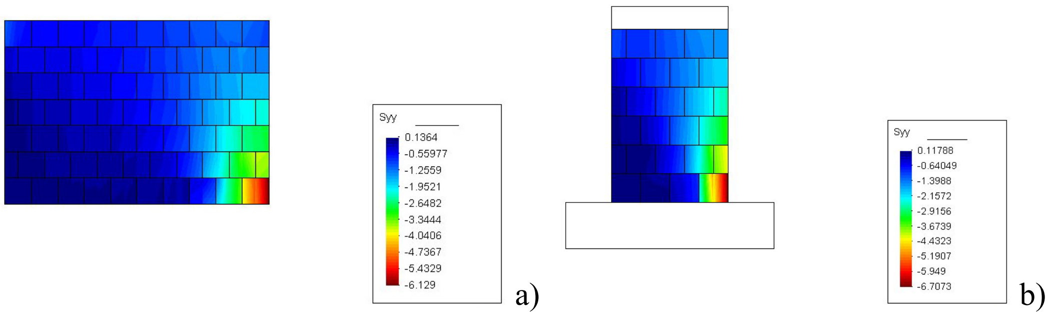

Figure 6. Distribution of normal stresses for large specimen BNW3 (a) and narrow masonry specimens BNL4 (b) at attained maximum resistance.

CONCLUSIONS

At the moment there are not standardized test methods for testing masonry in seismic conditions. From the proposed test methodology of testing narrow walls with different executions of perpend joints it was concluded that in all cases the behaviour of the specimens subjected to cyclic lateral loading has been governed by the premature local brittle failure of units. Consequently, the failure of the walls subjected to cyclic lateral loading occurred at a stage where the type of filling the vertical, perpend joints did not yet influence the behaviour of the walls. The failure mechanisms in all cases depended on the characteristics of masonry units and did not depend on the way of construction of the wall specimens, i.e. the type of the perpend joints.

Significant for all tested specimens made from Group II masonry units according to EC-6 classification (voids between 25-55%) is the lack of shear cracks along bed joints (horizontal sliding of the units on top of each other). This implies a very good bond between unit and bed

joints. However this also implies that large displacements and ductility of the specimens can be obtained solely due to the crushing of the unit, as it has been observed during the tests.

The size of the specimen with geometric aspect ratio of *h/b* = 1.5, may be representative for the evaluation of the shear resistance of laterally loaded URM, since the obtained mechanisms of failure as well as mechanical parameters are comparable with the parameters derived from much wider specimens with geometric aspect ratio of *b/h* = 1.5.

First results of numerical modelling confirm the experimental results related to the influence of the size of the specimens on the behaviour of shear loaded blockwork masonry in seismic conditions.

Following presented stages of investigation a comprehensive test program on different types of blocks with different hole patterns and thickness of the shell and webs of masonry specimen with geometric aspect ratio of *h/b* = 1.5 were already performed. Obtained results will be used for the purpose of future 2D and 3D numerical optimization of the units.

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