# EXPERIMENTAL EVALUATION OF THE SEISMIC BEHAVIOUR OF MULTI-LEAF STONE MASONRY WALLS

**M. Kržan[1] and V. Bosiljkov[2]**

[1] PhD Candidate, University of Ljubljana, Faculty of Civil and Geodetic Engineering, Jamova 2, 1000 Ljubljana, Slovenia, meta.krzan@fgg.uni-lj.si

[2] Assoc. Professor, University of Ljubljana, Faculty of Civil and Geodetic Engineering, Jamova 2, 1000 Ljubljana, Slovenia, vlatko.bosiljkov@fgg.uni-lj.si

## ABSTRACT

There is a lack of knowledge in the behaviour of multi-leaf stone masonry walls under compression and shear loads. Therefore within the framework of PERPETUATE project an extensive experimental campaign on three-leaf stone masonry assemblies was performed. The specimens and the tests were designed to study the performance of plastered multi-leaf stone masonry walls, under in-plane seismic loading and with different boundary conditions. Two distinct morphologies of masonry were studied by comparing the behaviour of the walls with and without header stones. Two wallettes and two walls, one of each morphology, were tested in compression. In 14 cyclic in-plane shear tests two different pre-compression levels and boundary conditions were used; walls with single and both fixed ends. Different failure mechanisms were achieved. Results show high compression strength of masonry assemblies despite quite weak mortar. Through stones did not contribute to higher compressive strength. From shear tests different failure mechanisms, and in respect to that, different deformation capacities of walls, were obtained. Leaf separation proved to be more problematic when vertical pre-compression was higher. With no through stones, cracks between the leaves thicker than 15 mm formed, while with header stones present thinner cracks evolved. Connecting stones worked well at lower pre-compression level, where they prevented the splitting, but they had no influence on obtained shear strength or on the overall seismic performance of the walls.

**KEYWORDS**: multi-leaf stone masonry, cyclic shear tests, seismic behaviour, plasters performance

## INTRODUCTION

Masonry buildings present a large part of Slovenian buildings' stock and many of existing stone masonry buildings represent cultural heritage assets. In Slovenia, as elsewhere, most of the stone masonry is constructed from two or more leaves. Three-leaf masonry is the most characteristic for our architectural heritage from Romanesque period onward, where the outer leaves are constructed in different texture and morphology from different types of stones, while the inner core is filled with stone rubble mixed with loose adhesive material. The inner core has more or less voids [1] and is sometimes connected with the outer leaves with transverse elements. The behaviour of stone masonry is hard to predict due to many parameters influencing its resistance; the material characteristics of constituents, masonry texture and morphology, the geometry of the structural element, the boundary conditions and pre-compression level, which depend on the structural elements' integration in the building, etc. The problem becomes even more complex when the masonry consists of more leaves; the characteristics and consequently behaviour of specific leaves under load differs, leaf separation occurs. Many experimental testing on single leaf masonry has been done, but there are just a few test results on multi-leaf masonry. Most of them study compression behaviour and again only some test the behaviour under lateral load [2].

Within the European research project PERPETUATE (PERformance - based aPproach to Earthquake proTection of cUlturAl heriTage in European and Mediterranean countries, www.perpetuate.eu) an extensive laboratory work aimed to study the behaviour of multi-leaf stone masonry walls under seismic loading was conducted in Laboratory of Faculty of Civil and Geodetic Engineering, University of Ljubljana. Altogether 18 walls were built, 4 were tested under compression loading and 14 under combined shear and compression load. As the type of walls tested may be found in representative buildings, such as castles, churches…, our aim was also to study the behaviour of plaster attached to the walls. The evaluation of characteristic limit points is of great importance for analyzing performance limit states of frescoes, stuccos, mosaics in such buildings in the case of seismic events.

**EXPERIMENTAL CAMPAIGN**

The specimens were built by trained masons. Lime putty with added tuff was used for construction in order to achieve faster reaction and save time. External leaves were constructed from regular coursed squared ashlar rough tooled lime stone, while the internal core was filled with stone rubble and lime mortar. As specimens were designed to study the influence of different morphology, half of the specimens had in every second row header stones going through the whole depth of the specimens (Figure 1a), and the other half had no such connecting stones (Figure 1b).

a) b) c)

**Figure 1: Construction of walls: a) with through stones; b) without through stones and c) the specimens before construction of upper concrete blocks**

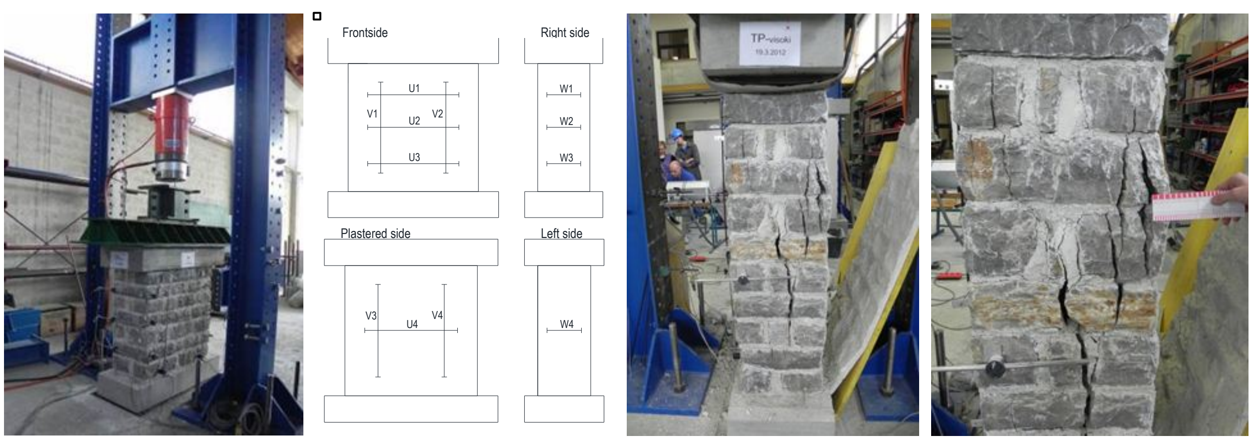
Two wallettes (one of each morphology) of dimensions 100x40x100 $cm^3$ were built for compression tests and 16 walls of dimensions 100x40x150 $cm^3$ were intended for shear tests. Later 2 of these walls were tested in compression instead. Average compression strength of the mortar at day of testing of masonry specimens was 1.88 MPa with standard deviation (st.dev.) of 0.11 MPa and average flexural strength 0.61 MPa (st.dev. 0.07 MPa). Mortar tests were performed according to standard EN 1015-11 [3]. Average compression strength of stone was 171.5 MPa (st.dev. 41.3 MPa) and average flexural strength 24.2 MPa (st.dev. 4.2 MPa).


a) b) c) d) e)

**Figure 2: a) morphology of connected wall and of b) unconnected wall; c) higher wall specimen; d) wallette with plaster and e) two layers of plaster applied**

## COMPRESSION TESTS

At first the two wallettes were tested in compression. Hydraulic jack of 2500 kN capacity was used and constant force increase was applied. Test setup can be seen in Figure 3a. Vertical and horizontal deformations were measured with 11 LVDTs (Figure 3b). Despite numerous cracking within masonry assemblage the maximum capacity of the wallettes was not reached. Therefore also wall specimens (one with and one without through stones) were tested and maximum strength and failure were obtained (Figure 3c and d).




a) b) c) d)

**Figure 3: a) compression test setup; b) measuring positions; c) and d) failure of the connected wall**

Strain-stress relations obtained during tests for various LVDTs for unconnected and connected walls are presented in Figure 4a and Figure 4b respectively. From tests on walls obtained average compression strength $f_{Mc}$ for both walls was 6.05 MPa. There was no apparent difference in the mechanism of failure for the wall with and the wall without through stones. Contrary to expectations, the $f_{Mc}$ of the wall without through stones was even slightly higher.

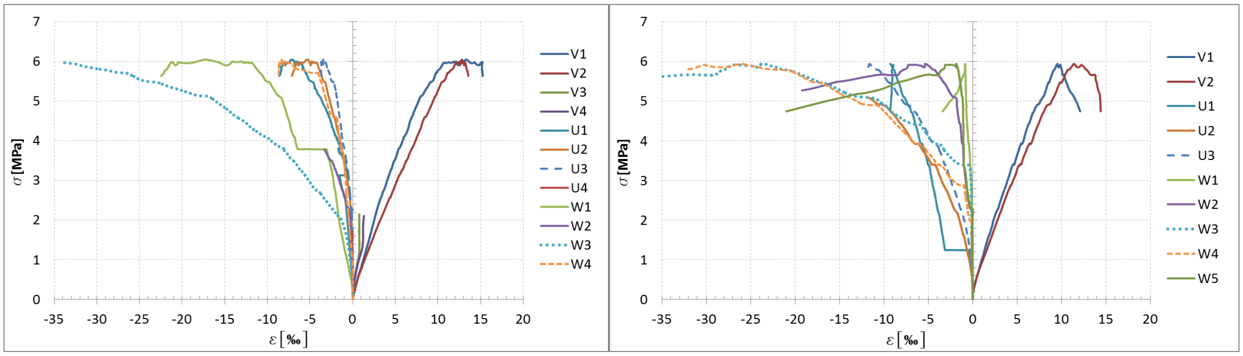



**Figure 4: Stress-strain diagram for various LVDTs for a) wall without through stones and b) wall with through stones**

Modulus of elasticity $E$ was calculated from average vertical strains and stresses considering the whole cross section at mid height at level of stress equal to 1/3 $f_{Mc}$ obtained in tests of walls and for comparison also from tests on wallettes, where reference stress and strain were considered at 1/3 $\varsigma_{max}$. Shear moduli $G$ were also calculated after linear elastic theory for homogenous isotropic linear elastic materials (Equation 1).

$$G = \frac{E}{2 \cdot (1 + \nu)} \tag{1}$$

$\nu$ is the Poisson's ratio and was determined as the ratio between average vertical strain and average horizontal strain, where both in-plane and transversal horizontal deformations were considered. It has to be noted, that results for Poisson ratio differ considerably in dependence of the position of measuring devices for horizontal deformations. The obtained results are summarized in Table 1.

**Table 1: Results of compression tests**

| Test | Results…. | $\sigma_{max}$ [MPa] | avg. $\sigma_{max}$ [MPa] | E [MPa] | avg. E [MPa] | $\nu$ | G [MPa] | avg. G [MPa] | G / E |
|---|---|---|---|---|---|---|---|---|---|
| 1 | Wallette without through stones | 7.34* | 7.31 | 1570 | 1052 | 0.187 | 661 | 438 | 0.42 |
| 2 | Wallette with through stones | 7.28* | | 534 | | 0.265 | 214 | | 0.40 |
| 3 | Wall without through stones | 6.10 | 6.05 | 1138 | 968 | 0.226 | 412 | 357 | 0.36 |
| 4 | Wall with through stones | 6.00 | | 798 | | 0.319 | 302 | | 0.38 |

* values correspond to the peak stress at severely cracked masonry assemblage

If the obtained $f_{Mc}$ are compared with minimal (6 MPa) and maximal (8 MPa) values according to Italian codes NTC 08 [3] for dressed rectangular stone masonry, the results are within expected values. However, moduli $E$ and $G$ obtained in tests are significantly lower; according to NTC, values are between 2400 and 3200 MPa for $E$ and 780 and 940 MPa for $G$. Comparing the results of walls with and without through stones, it may be concluded that elastic and shear modulus are lower in case of wall with through stones; *E* for 30% and *G* for 27%. This difference most probably results due to fact, that connected wall had 10 courses of stone units while unconnected wall had 11 courses. Consecquently the average mortar joint thicknesses for specimens were different. Ratio between shear and elastic modulus calculated at one third maximum stress is for all tests between 0.36 and 0.42.

## CYCLIC IN-PLANE SHEAR TEST RESULTS

Cyclic in-plane shear tests of walls under constant compression load were performed in the testing machine presented in Figure 5. The vertical load was applied with concrete weights which acted through a lever on the bottom of the wall.

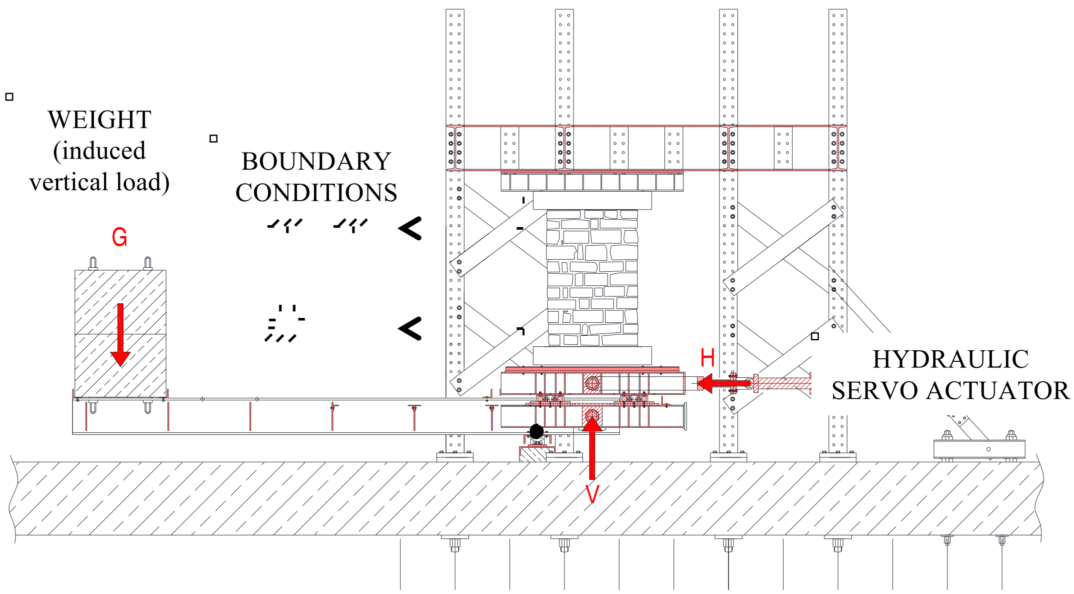



**Figure 5: Shear test setup**

The test setup allows the input of 500 kN of vertical load on the specimens and the servo-hydraulic actuator is capable of inducing two-way horizontal displacements with the capacity of 250 kN. On the lower edge of the specimen two possibilities of the boundary conditions were applied; in one case the rotation and the horizontal displacement were released, which made the wall a cantilever turned upside down, in the other case the rotation at the bottom was restrained, which made the wall a laterally deforming rotation fixed-fixed system. To attain displacements of the wall and to monitor the experiment 19 LVDTs were used (Figure 6a). Due to the possibility of damaging the measuring equipment, the instruments were not attached to the plaster during the experiment; the displacements of the plaster were measured with photogrammetry. All throughout each experiment also a survey of crack formation and their propagation were monitored.

During the test the displacement was imposed with constant velocity within blocks of cycles with specific displacement amplitude. Each amplitude peak was repeated three times to get the stiffness and strength degradation and deterioration in the nonlinear range. The displacement time history can be seen in Figure 6b.

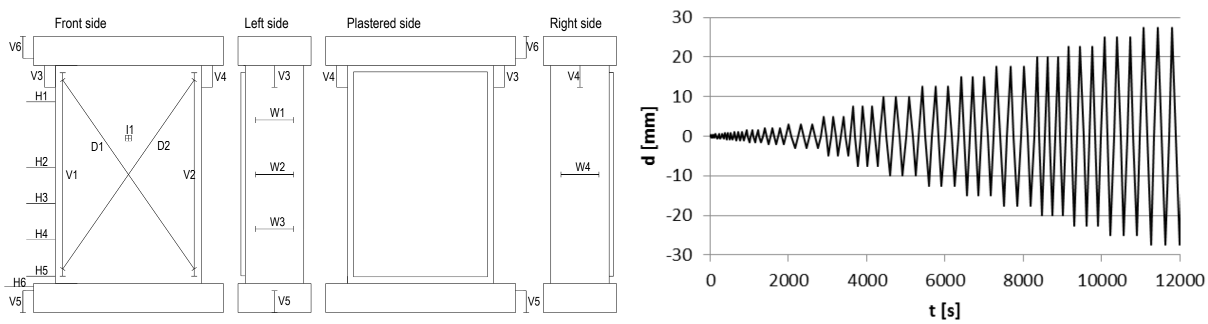



**Figure 6: a) measuring positions; b) loading protocol**

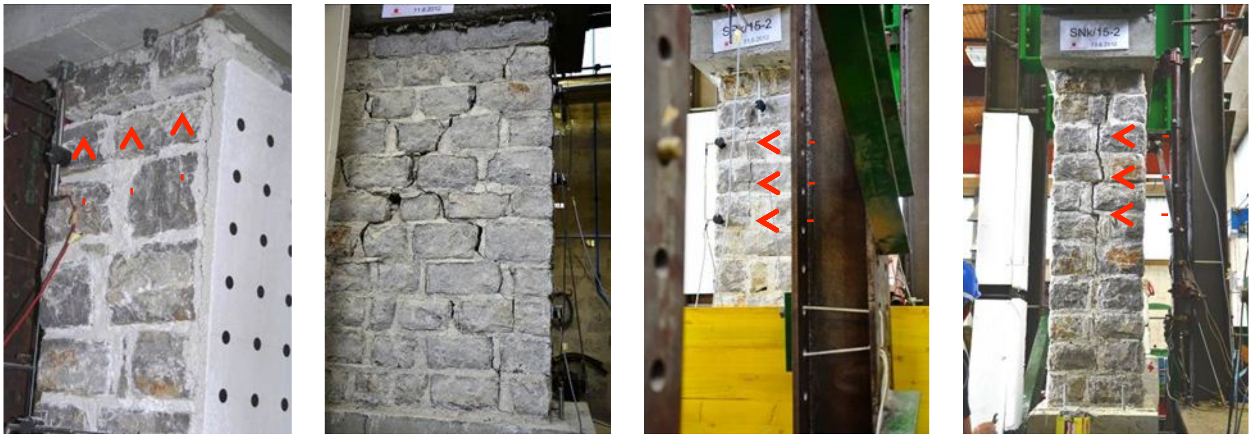
As already mentioned, the possibilities of the testing set-up allowed us to test the walls under various combinations of loading and boundary conditions. To obtain different failure mechanisms, two levels of pre-compression and described single and both fixed boundary conditions were applied. In Table 2, the conditions of experimental tests are presented. Except of the specimens, tested as a cantilever under lower level of pre-compression, each combination of pre-compression, boundary conditions and morphology had one repetition. Test 1.2 refers to second test on the 1st specimen, with higher vertical load level imposed.

**Table 2: Testing combinations**

| No. | Name | Level of pre-compression [% $f_{Mc}$] | Boundary conditions | Connecting through stone |
|---|---|---|---|---|
| 1 | SPk-5-1 | 5 | cantilever | yes |
| 1.2 | SPk-5-1 (7.5) | 7.5 | cantilever | yes |
| 2 | SNk-7.5-1 | 7.5 | cantilever | no |
| 3 | SNv-7.5-1 | 7.5 | fixed-fixed | no |
| 4 | SPv-7.5-1 | 7.5 | fixed-fixed | yes |
| 5 | SNv-7.5-2 | 7.5 | fixed-fixed | no |
| 6 | SPv-7.5-2 | 7.5 | fixed-fixed | yes |
| 7 | SPv-15-1 | 15 | fixed-fixed | yes |
| 8 | SNv-15-1 | 15 | fixed-fixed | no |
| 9 | SPv-15-2 | 15 | fixed-fixed | yes |
| 10 | SNv-15-2 | 15 | fixed-fixed | no |
| 11 | SNk-15-1 | 15 | cantilever | no |
| 12 | SPk-15-1 | 15 | cantilever | yes |
| 13 | SPk-15-2 | 15 | cantilever | yes |
| 14 | SNk-15-2 | 15 | cantilever | no |

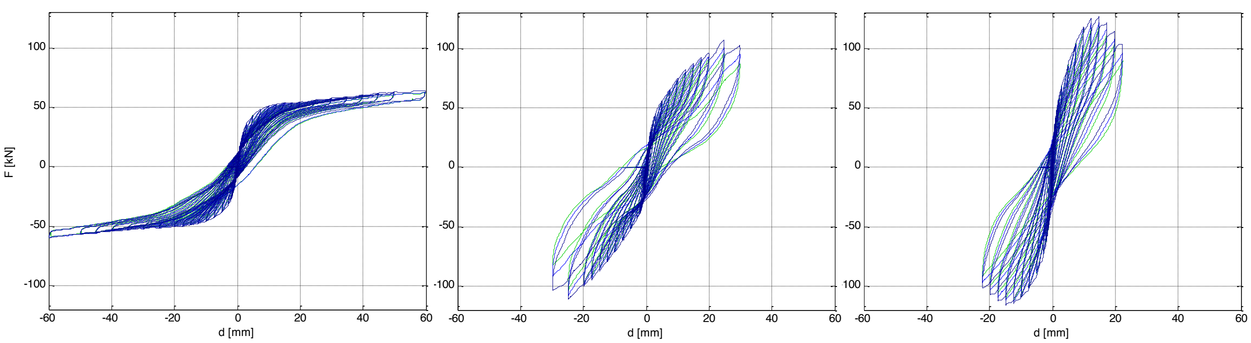
* Name of the test consists of letter S for specimen, N or P relate to morphology (N – without and P - with through stones), k or v to boundary conditions (k – cantilever and v – double fixed),7.5 or 15 to level of pre-compression in % of $f_{Mc}$ and 1or 2 for 1st test or repetition

With these combinations various failure mechanisms of walls were attained. By lower pre-compression level and cantilever boundary conditions rocking occurred, where the joint between the first and the second row of stones opened (Figure 7a). By fixed-fixed boundary conditions the walls rocked, but also shear damage occurred, therefore this failure mechanism was referred as mixed. At higher pre-compression levels shear failure occurred in case of both boundary conditions (Figure 7b). In Figure 8 typical hysteretic responses obtained with different failure modes in tests can be seen; different strength and displacement capacities obtained in dependence of failure mechanism are obvious (rocking in Figure 8a, mixed in Figure 8b and shear failure in Figure 8c). These are all results of walls without through stones.



a) b) c) d)

**Figure 7: a) rocking in test 2, b) shear damage, c) leaf separation at test 13 and d) leaf separation at test 14**




a) b) c)

**Figure 8: Hysteretic response of a) test 2 – rocking; b) test 5 – mixed response and c) test 8 – shear failure**

Force-displacement envelope curves from experimental tests were idealized to bilinear curves considering equivalent input energy. From idealized shear resistance $H_{id}$ a reference tensile strength of masonry $f_{Mt}$ was calculated as an indicator for shear strength of masonry, see Equation 2. This strength was determined according to Turnšek and Čačovič [5] as the critical value of principal stress in the center of the pier, by which the diagonal shear failure occurs.

$$f_{Mt} = -0.5 \cdot \sigma_0 + \sqrt{(0.5 \cdot \sigma_0)^2 + (b\ \tau)^2} \tag{2}$$

Where $\tau$ is the average shear stress on the whole cross section, $b$ a coefficient which takes into account the geometry of the walls; for our case b=1.5 as the walls have aspect ratio equal to 1.5, $\sigma_0$ the mean vertical stress on the pier due to vertical load. The results are compared with minimal and maximal values according to Italian codes NTC 08 [3] (Figure 9).

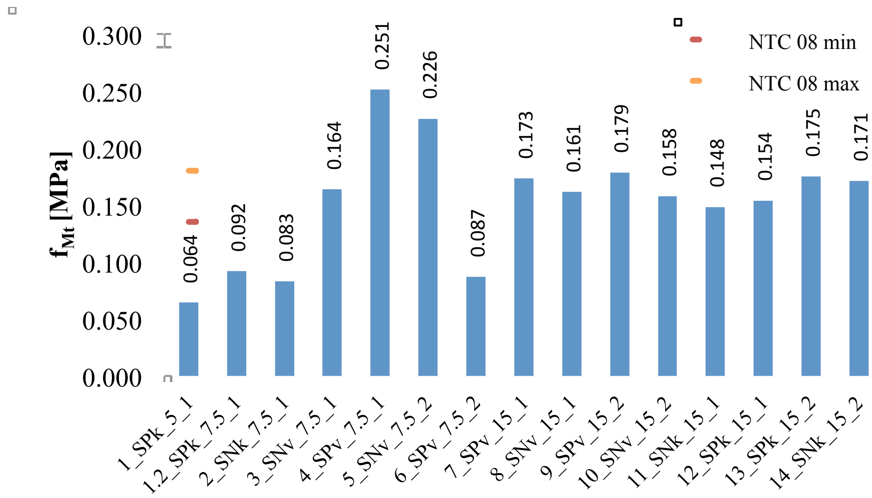



**Figure 9: Tensile strength $f_{Mt}$ of tested walls**

The results again show, that through stones do not contribute to better strength characteristics. Only in tests with high precompression and fixed-fixed boundary conditions both walls with through stone proved higher resistance. In first three tests maximum force was not obtained as rocking mechanism prevailed, which explains the low tensile strengths. Lower $f_{Mt}$ in test no.6 may be attributed to the influence of the joint thickness $h_j$. All masonry specimens had 11 or 12 courses of units with average $h_j$ 1.0-2.5 cm, while specimen no.6 had 10 courses with $h_j$ of 2-3 cm.

From idealized curves effective stiffnesses $K_{eff}$ and from them shear moduli $G$ according to Equation 3 were calculated.

$$G = K_{eff} \Big/ \left( A/(1.2h) - \left(\psi K_{eff}/(1.2E)\right)(h/l)^2 \right) \tag{3}$$

where $A$ is the cross section area, $h$ and $l$ height and length of the wall and $\psi$ coefficient taking into account boundary conditions; $\psi$ = 4 for single fixed and $\psi$ = 1 for double fixed walls. But as the results differed noticeably for different idealization criteria, also stiffnesses $K_{d=1.5mm}$ and shear moduli $G_{d=1.5mm}$ from un-idealized response for state with no or very slight damage that is at 1.5 mm displacement, were evaluated. $G_{d=1.5mm}$ was calculated as the ratio of average shear stress to shear strain at the top. In Table 3 average calculated characteristics for walls with the same morphology tested under the same boundary condition are presented.

As the effective stiffness $K_{eff}$ depends considerably from idealization criteria and as the hysteretic responses differ due to various failure mechanisms obtained, the $G$ modulus calculated from stiffnesses from bilinear idealization produced in some cases unrealistic results. More realistic were the results for state with no very slight damage $G_{d=1.5mm}$. However, even with this approach, obtained values for shear moduli are significantly lower than the values obtained from compressive tests (Table 1).

**Table 3: Average values obtained from idealized curves and from tests for same morphology and boundary conditions**

| | $f_{Mt}$ [MPa] | st. dev | $K_{eff}$ [kN/mm] | st. dev | G [MPa] | st. dev | $K_{d=1.5mm}$ [kN/mm] | st. dev | $G_{d=1.5mm}$ [MPa] | st. dev |
|---|---|---|---|---|---|---|---|---|---|---|
| Through stones, cantilever | 0.147 | 0.06 | 38.7 | 2.6 | 15 | 18 | 25.1 | 13.6 | 94 | 51.2 |
| Through stones, fixed-fixed | | | 18.9 | 9.4 | 116 | 68 | 28.0 | 10.3 | 105 | 38.6 |
| No through stones, cantilever | 0.173 | 0.03 | 16.0 | 7.3 | 196 | 250 | 30.3 | 7.4 | 114 | 27.7 |
| No through stones, fixed-fixed | | | 14.2 | 7.3 | 81 | 48 | 28.2 | 6.5 | 106 | 24.3 |

Regarding displacement performance, analysis of drifts for different limit states is provided in Figure 10; the results for all walls in dependence from failure mode are summarized. Column LD refers to displacement, where first crack was obtained, SD to displacement, where maximum strength was attained and NC presents average of maximal displacements reached in both directions.

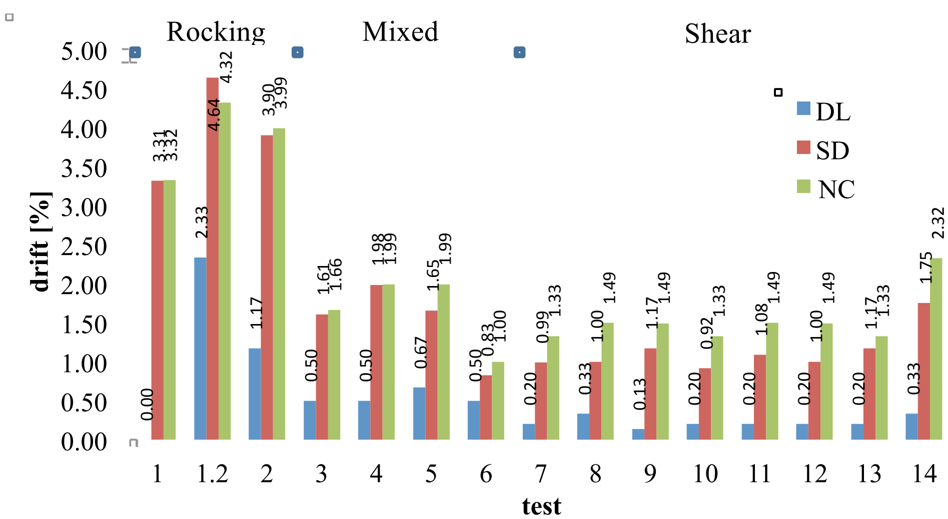



**Figure 10: Performance limit states for tested walls**

As it was expected, specimens that failed due to rocking had the highest drifts. The specimens that failed due to shear exhibited the lowest displacement capacity. By considering the influence of boundary conditions on obtained performance levels it may be concluded as expected that the cantilever walls performed better. The level of pre-compression influenced the behaviour of the specimens under seismic loading more apparent, as the imposed vertical loads differed significantly. The influence of different morphologies on the performance limit states was not apparent, though it should be mentioned that in tests with higher pre-compression level the specimens with through stones exhibited much lower lateral out-of-plane deformations resulting from leaf separation (Figure 7c) compared to unconnected walls (Figure 7d). This was especially apparent in the softening phase.

## PLASTER PERFORMANCE

The behaviour of artistic assets during the tests also differed obviously in dependence of failure mode. A comparison of data collected in visual observations during the tests is presented in chart in Figure 12, where four limit states presented in Figure 11 are defined as:

- first detachment of the plaster, presented in Figure 11a,
- first visible crack on the plaster,
- plaster largely detached but still repairable or significantly damaged, presented in Figure 11b and Figure 11c,
- partial or full collapse of the plaster, presented in Figure 11d.

a) b) c) d)

**Figure 11: a) first visual observation of plaster delamination; b) plaster delaminated over the whole height; c) 5 mm wide shear crack at test 4; d) partially collapsed plaster at test 5**

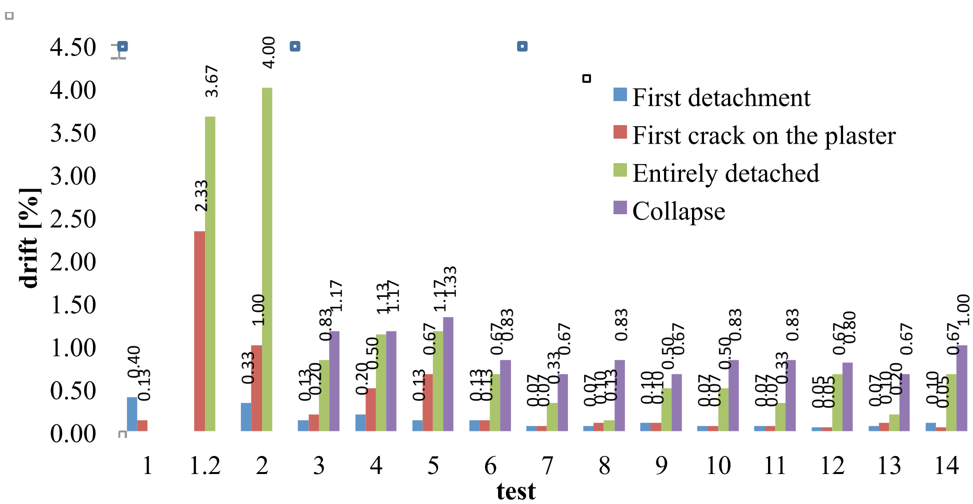



**Figure 12: Drift values of walls for characteristic plaster performance points**

It has to be noted, that displacement/drift values for particular limit state presented here relate to recorded values of the wall's displacements at the bottom and are based on visual inspection following each level of displacement. It can be seen in Figure 12 that for specimens that failed due to rocking, the plaster did not collapse. For mixed failure drift values for all limit states are significantly higher in comparison to the drifts obtained for specimens that failed in shear.

## CONCLUSIONS

Compression and cyclic shear tests under constant vertical load were conducted on three-leaf stone masonry walls constructed in laboratory. The results from compressive tests regarding strength characteristic are within expected values; in average 6.05 MPa was reached. However, as also the influence of morphology was studied, the through stones proved not to contribute to any additional strength. Elastic and shear moduli obtained are significantly lower as expected.

From cyclic shear tests the same conclusion can be drawn. For tested type of stone masonry, where the inner core is in solid state and without voids and the leaves present approximately 75% of the whole cross section, the through stones do not contribute neither to higher shear resistance nor to higher displacement capacity. Their benefit was clear only for specimens tested under higher pre-compression, where shear failure occurred; in the post peak behaviour the leaf separation was considerable smaller. Regarding obtained strength and displacement capacity of walls as well as for the plasters, it was confirmed, that they depend significantly on the failure mechanism of the wall. With rocking, the resistance is lower but displacement capacity of walls and of plaster is very high (drift values over 4%), whereas with shear failure, the resistance is higher, but displacements are considerable lower. Nevertheless, for all walls that failed in shear, obtained maximum drifts for walls were in all cases still higher than 1% while the plaster collapsed in all cases at 0.67% or at even higher drifts.

## ACKNOWLEDGEMENT

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