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Compression and In-plane Seismic Behaviour of Ashlar Three-leaf Stone Masonry Walls

Meta Kržan^a and Vlatko Bosiljkov^b

^aThe Department for Structures, Slovenian National Building and Civil Engineering Institute, Ljubljana, Slovenia; ^bFaculty of Civil and Geodetic Engineering, University of Ljubljana, Ljubljana, Slovenia

ABSTRACT

There is a lack of knowledge regarding the parameters needed for the effective seismic performance assessment of historical stone masonry buildings. The paper presents the results of an extensive experimental campaign consisting of four compression and fifteen in-plane cyclic shear tests on three-leaf ashlar stone masonry walls and its constituents. The morphology, level of vertical load, and testing boundary conditions were systematically varied, and their influence on the load-bearing and deformation capacities, stiffness, ductility and energy dissipation was analysed. Good consolidation of the inner core enables in-plane failure mechanism of the walls to occur without decisive out-of-plane damage. The boundary conditions applied significantly influence the failure modes and large deformation capacities obtained, whereas transverse connections in the case of a shear failure mechanism do not significantly influence the overall lateral in-plane response of walls.

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1. Introduction

Buildings constructed from multi-leaf masonry present a significant part of architectural heritage building stock, both in Europe and around the world. Many of these buildings are monuments of distinct cultural significance and there is a reasoned tendency to preserve them. In order to ensure these buildings can be safely used, the crucial step is to evaluate whether they meet the structural safety requirements under various types of load and risk. Earthquakes present a significant and specific problem for most historic buildings in earthquake prone regions, as many of them were not designed for the lateral loads that can be induced during earthquakes. In order to assess a (historic) structure with numerical models, adequate material models describing the behaviour of structural elements, as well as the mechanical characteristics of the materials and/or structural elements, need to be available. The structural behaviour of masonry elements is difficult to accurately predict due to the composite heterogeneous nature of masonry (Binda, Penazzi, and Saisi 2003; Borri 2006; Tomaževič, Gams, and Berset 2014). In multi-leaf masonry the problem is even more complex, due to the additional influential parameters that characterize the morphology i.e. the layout, form and size of the elements across the cross section, and the type and quantity of the infill materials, including voids in the various leaves. The

behaviour of masonry elements within the structure also depends on their position, their structural role, and details of the construction. In the case of masonry walls, geometrical characteristics, imposed stresses, and boundary conditions all have a significant influence on their behaviour under seismic loading (Corrêa et al. 2006; Magenes and Calvi 1997; Salmanpour, Mojsilović, and Schwartz 2015; Tomaževič 1999). They influence the damage and failure mechanisms, strength and displacement capacity, and other mechanical properties necessary for characterization of the non-linear masonry response. When considering multi-leaf masonry, either partial or full out-of-plane collapse of the walls due to poor connectivity between the leaves could be critical in seismic events (Calderoni et al. 2009).

The possibility of strengthening multi-leaf masonry by means of grouting was first studied after major earthquakes in the former SFRY and Italy (Friuli '76 and Montenegro '79). Compression and shear tests of grouted and un-grouted two-leaf rubble stone masonry walls were mainly performed within the context of planning remedial or retrofitting activities following these major seismic events (Beolchini 1992; Chiostrini and Vignoli 1992; Tomaževič and Sheppard 1982; Turnšek et al. 1978). Later, the importance of load transfer in multi-leaf masonry was experimentally and analytically investigated. Most studies focused on compression and/

or diagonal compression tests (for the analysis of shear behaviour) in various types of single-leaf stone masonry (Brignola et al. 2008; Drei and Fontana 2000; Galasco, Penna, and Magenes 2009; Milosevic et al. 2013; Modena 1999; Pinho and Lúcio 2017; Pinho, Lúcio, and Baião 2012; Tassios 2004; Toumbakari 2002; Valluzzi 2000; Valluzzi, Da Porto, and Modena 2004; Vintzileou and Miltiadou-Fezans 2008; Vintzileou and Tassios 1995); a few in-plane cyclic shear tests on multi-leaf rubble stone masonry (Borri, Castori, and Corradi 2015; Milosevic et al. 2015; Pinho and Lúcio 2017; Silva et al. 2014a, 2014b) and dry-joint ashlar masonry (Vasconcelos and Lourenço 2009) were also conducted, mostly in laboratory conditions. Studies with compression and shear tests of strengthened and unstrengthened stone masonry walls evaluated in-situ are rare (Andreini et al. 2014; Corradi, Borri, and Vignoli 2003, 2008; Costa et al. 2011; Tomažević, Klemenc, and Lutman 2000; Uranjek et al. 2012). Though the aforementioned studies differ in the type of masonry (texture, morphology, presence of voids, mortar strength, etc.), and most of them do not provide quality statistical data, they have proven that, when the connection between leaves is poor (due to large presence of voids), early separation of the external leaves from the internal filling material may rapidly lead to disintegration of the wall due to enhanced sensitivity resulting from second-order effects (buckling of the outer leaves) under compression loading. The more compact inner core (without any voids) obtained by grouting (even with grout mixtures of a low strength) increases the compressive strength and also postpones or even disables the separation of leaves in such extent so that a response similar to that of single-leaf masonry may be obtained. When walls are connected without transversal units, already keyed collar joints, as opposed to straight collar joints, allow cracks to form in the inner core rather than vertical cracks forming at the interfaces of the leaves (Binda et al. 2006).

The presence of various types of transversal connectors in multi-leaf masonry has been proven to limit out-of-plane behaviour and separation of the leaves (Oliveira et al. 2012; Toumbakari and Van Gemert 1997; Valluzzi, Da Porto, and Modena 2004), and to significantly increase a specimen's compressive strength (Corradi et al. 2017). Though most of the studies have been conducted under monotonic or quasi-static cyclic loading, the behaviour of a multi-leaf stone masonry building under dynamic loading has also been studied (Mazzon 2010; Mazzon et al. 2009).

The research presented provides the results of an extensive experimental campaign conducted on unstrengthened plastered multi-leaf stone masonry walls. The configuration of the walls and the mechanical

characteristics of the materials were similar to that of good quality historic ashlar stone masonry walls that may be found in heritage buildings of higher importance. The main idea of the campaign was to systematically study the behaviour of plastered multi-leaf walls under compression and in-plane cyclic shear loading, in order to assess the suitability of existing compression and shear resistance analytical models (the results of which are presented in (Kržan and Bosiljkov 2021)); it further aimed to determine reference drift values for the PBA (Performance Based Assessment) of cultural heritage buildings with regard to the damage of structural walls and artistic assets — namely, decorated plasters attached to the walls (the results of which have been presented elsewhere; (Calderini et al. 2015)).

2. Materials and methods

2.1. Test specimens

Two wallettes of dimensions $100/40/100 \text{ cm}^3$ ($l_w/t_w/h_w$) and sixteen walls of dimensions $100/40/150 \text{ cm}^3$ were built for the campaign by trained masons. External leaves were constructed from regularly coursed, squared and roughly tooled ashlar limestone with an average thickness of 15 cm, while the internal core, with a thickness of approximately 10 cm, was filled with stone rubble and lime mortar. Most of the specimens' external leaves were constructed from 11 rows of stone. As the specimens were designed to study the influence of different morphologies, half of the specimens had header stones in every second row, running throughout the depth of the specimen (referred to as connected walls throughout the rest of the text, Figure 1a). The other half had no header stones (referred to as unconnected walls, Figure 1b). Significant attention was given to the proper choice of mortar, both in terms of its composition and preparation. Traditionally produced lime putty was used to produce the mortar, using a lime putty: tuff: aggregate volume ratio of 1: 0.2: 4 and a water to lime putty ratio of 0.39. The 20% (volume ratio) of tuff was added to the binder in order to accelerate the hardening process. By adding this, a mortar representative of historic mortar (i.e. a compressive strength, f_{mc} , of less than 2 MPa) was produced in time for the masonry tests. The first compression tests were conducted on the specimens at the age of 4 months, while the first shear tests were done on 5-month old masonry specimens.

At the start of the compression tests on the wallettes the average f_{mc} was 1.88 MPa, with a coefficient of variation, CV, (ratio of the standard deviation to the mean value of the samples) equal to 6% (6 samples), and an average flexural strength (f_{mf}) of 0.61 MPa (3 samples, CV 12%). Mortar tests were

Figure 1. Texture and morphology of a) a connected wall with header stones; b) an unconnected wall without header stones; c) a wallette specimen with plaster; d) a wall specimen.

performed according to the standard EN 1015-11 (EN 1015-11 1999). The average compressive strength obtained from the stone-cube specimens ($7 \times 7 \times 7 \text{ cm}^3$) was 171.5 MPa (6 samples, CV 24%), while the average flexural strength obtained from the stone-prism specimens ($4 \times 4 \times 16 \text{ cm}^3$) was 24.2 MPa (3 samples, CV 17%). The bond strength of the stone-mortar junctions was determined by means of a “Bond wrench” test on three samples taken from the wall after the wall had been tested in compression. The failure occurred at the mortar-stone junction, which had an average flexural tensile strength of 0.03 MPa (CV 27%). The compressive strength of the core (15 cm diameter, 30 cm length) was 0.90 MPa (3 samples, CV 24%), with its splitting tensile strength according to EN 12390-6:2001 (SIST EN 12390-6:2001 2001) measuring 0.16 MPa (CV 53%); these values were obtained from tests on the cylindrical core samples (Figure 2). Both the wallettes and the walls were plastered on one side of the specimen (see schematic of a wallette in Figure 1c). The plasters were lime-based and designed to not influence the overall behaviour of the masonry walls.

Figure 2. A cylindrical core samples during the compression (left) and splitting tensile (right) tests.

2.2. Testing program

2.2.1. Compression tests

Two wallettes (Figure 1c) and two walls (Figure 1d), one of each morphology type, were tested in compression through monotonic loading. The test setup is presented in Figure 3. A hydraulic jack of 3000 kN capacity was used for this setup, with vertical and horizontal deformations measured by 11 linear variable differential transformers (LVDTs) (Figure 6).

Figure 3. Compression test.

2.2.2. Cyclic in-plane shear tests

Altogether, 15 cyclic in-plane shear tests were performed on 14 wall specimens under a constant compression load, using the test setup presented in Figure 4a. The vertical load was applied by a weight which acted through a lever on the bottom of the wall. During the test, the displacement was imposed by a servo-hydraulic actuator at the bottom of the specimen, maintaining a constant velocity within blocks of cycles at specific displacement

amplitudes. Each amplitude peak was repeated three times in order to attain the stiffness and strength degradation with deterioration in the nonlinear range (Tomažević, Lutman, and Petković 1996). The displacement time history is presented in Figure 4a. The tests were conducted up to the displacement at which a 20% drop in shear resistance was attained, or up to a displacement at which it was estimated that, as a result of the damage developed, any further increase in displacements would represent a risk of the specimen suddenly collapsing.

On the lower edge of the specimen, two versions of boundary conditions were imposed. In the first case, the rotation and lateral displacement were released, effectively making the wall an upside-down cantilever. In the second case, the rotation at the bottom was restrained, making the wall a laterally deforming, rotation fixed-fixed system (Figure 4a).

To obtain different failure mechanisms, as well as applying different boundary conditions, three different levels of constant vertical loads were applied to the walls during shear testing — i.e. 5.0, 7.5 and 15.0% of the masonry's compressive strength, f_{MC} . The combinations of morphology and testing conditions for the specimens are further summarised in Table 2, which also presents the damage mechanisms of the walls in the shear tests.

Nineteen LVDTs were installed to evaluate the displacements of the wall and monitor the experiment (Figure 4b): six to measure in-plane lateral displacements and to assess the shape of the in-plane deformation; four to measure leaf separation; one to measure out-of-plane displacements on one side of the wall; two vertical and two diagonal on one side of the wall to detect in-plane cracks and deformations of the wall due to compression; two for measuring the

Figure 4. (a) in-plane cyclic shear test setup and (b) position of the LVDTs to measure displacements and deformation in the shear tests.

“opening” of and mortar crushing in the first row of stone as a result of the rocking mechanism; and finally two for monitoring the test setup i.e. the rocking of the concrete base at the top and bottom of the specimen. Due to the possibility of damaging the measuring equipment, the instruments were not attached to the plastered face of the wall. Throughout each experiment, the formation and propagation of cracks both on the wall and on the plaster were visually monitored and recorded.

3. Results

3.1. Compression tests

3.1.1. Failure mechanism and compressive strength

Despite extensive cracking of the masonry assemblage in the wallettes tested, the compressive strength, f_{Mc} , was not attained due to the limited actuator capacity. Two walls were therefore additionally tested in compression. The maximum compressive stress, σ_{max} ,

Figure 5. Damage of the unconnected wall (a, b) and the connected wall (c, d) after a compression test.

Figure 6. Results of the compression tests: (a) Stress-strain results of the attached LVDTs and (b) leaf separation measured on the side with increasing compressive load.

Table 1. Results of the wall compression tests.

Test	f_{Mc} [MPa]	Mean [MPa]	E_M [MPa]	Mean [MPa]	u_M	G_M [MPa]	Mean [MPa]	G_M/E_M
Unconnected wall	6.10	6.05	1138	968	0.226	464	383	0.41
Connected wall	6.00		798		0.319	302		0.38

obtained for the wallettes was 7.34 MPa and 7.28 MPa for the unconnected and connected specimens, respectively.

Failure due to compressive loading was achieved in the case of the walls. There was no apparent difference between the failure mechanisms of the connected and unconnected walls. Contrary to expectation, wider cracks developed between the leaves in the connected walls, which can be seen in the crack patterns presented in Figure 5 and from the measurements of leaf separation (Figure 6b).

Compressive strength f_{Mc} was similar in both wall types; 6.10 MPa for the unconnected and 6.00 MPa for the connected wall. It should, however, be mentioned that the connected wall had only 10 rows of stone compared to 11 in the unconnected wall, meaning the thickness of the horizontal joint was higher in the connected wall. This fact may be the main reason that the results of the compression tests revealed no increase in compressive strength due to presence of connecting stones; if both walls tested had the same thickness of bed joints, the connected wall would provide superior compressive strength results in comparison to unconnected. The magnitude of strains ranged between 0.5% and 1.0% (Figure 6a), and could be attributed to the soft mortar.

Comparing the maximum compressive stresses of the wallettes with the compressive strengths of the walls, it can be concluded that the f_{Mc} of walls is at least 20% lower than that of the wallettes (since the f_{Mc} of wallettes was not achieved). This can be attributed to the scale effect, as well as to stress distribution (De Vekey 1995; Edgell 1990).

3.1.2. Stiffness characteristics

The modulus of elasticity (E_M) was calculated for the mid-height cross section from the average vertical strains and stresses at $1/3 f_{Mc}$. The shear moduli (G_M) were calculated using the linear elastic theory for homogeneous isotropic linear elastic materials, despite this assumption not being accurate for masonry (Equation 1):

$$G_M = \frac{E_M}{2 \cdot (1 + \nu_M)} \quad (1)$$

The ν_M was determined as the ratio between the average horizontal strain (considering both in-plane (LVDTs "U") and transverse horizontal deformations (LVDTs

"W")) and the average vertical strain. The results of the compression tests on the walls are summarised in Table 1.

While the header stones do not seem to have a big influence on the wall's compressive strength, there is a significant difference in the stiffness characteristics obtained. Contrary to expectation, the connected walls exhibited lower stiffness characteristics in comparison to the unconnected walls. One possible reason for the difference in the elastic modulus could be the fact that, in the case of the walls containing header stones, the unconnected wall was constructed from 11 rows of stone, compared to only 10 rows of stone in the connected wall. The thickness of horizontal mortar joints was consequently greater in the connected wall, as was the mortar/stone volume ratio. Another possible reason for the difference in elastic modulus could be that the settlement and shrinkage of the inner core of the walls containing header stones were not continuous across the cross-section; consequently, voids were formed under each header stone, creating additional weak spots in the wall and thus reducing the stiffness. These small differences, in addition to the presence of header stones, did not result in different strength characteristics, but they could have had a more significant influence on the characteristics of stiffness. On the other hand, an alternative explanation for such results could also be linked to the widely dispersed results, since it is known that stiffness characteristics have a higher scatter than strength characteristics (Vanin et al. 2017).

3.2. Cyclic shear tests

3.2.1. Characteristic damage and failure mechanisms

Due to the numerous pre-compression levels and boundary conditions applied, various damage and failure mechanisms were obtained, as summarised in Table 2. Low pre-compression levels (5.0 and 7.5% f_{Mc}) accompanied by the cantilever boundary condition provoked the rocking mechanism, which caused the horizontal joint between the first and the second rows of stone to open (Figure 7a, mechanism a in Table 2). Restraining the rotations at the bottom enabled the partial development of the shear mechanism (this failure mechanism is referred to as mixed). At a higher pre-compression level (15% f_{Mc}), the shear behaviour was induced regardless of the boundary

Table 2. Specimen characteristics, combinations of shear tests and the damage mechanisms attained in the walls during the cyclic shear tests.

The figure consists of two parts. On the left, a 2D schematic of a wall specimen is shown under loading. A horizontal force F is applied to the top edge, and a vertical load q_0 is applied to the base. On the right, a 3D perspective view of a wall specimen illustrates various damage mechanisms labeled a) through f). A) shows rocking with opening of the 1st or 2nd horizontal joint. B) shows diagonal shear cracks. C) shows crushing of mortar in the middle area of the wall. D) shows vertical column cracks. E) shows toe crushing. F) shows leaf separation.

Loading:

Wall damage:

a) Rocking with opening of 1st or 2nd horizontal joint;
b) Diagonal shear cracks;
c) Crushing of mortar in the middle area of the wall;
d) Vertical column cracks;
e) Toe crushing;
f) Leaf separation.

Test	Vert. load [% f_{Mc}]	Boundary conditions	Header stones	No. of rows of stone	Wall damage mechanism						Failure mechanism
					a)	b)	c)	d)	e)	f)	
1-SPk-5-1	5	Cantilever	Yes	11	✓						Rocking
1.2-SPk-7.5-1	7.5	Cantilever	Yes	11	✓						Rocking
2-SNk-7.5-1	7.5	Cantilever	No	11	✓				Mortar	Minor*	Rocking
3-SNv-7.5-1	7.5	Fixed-Fixed	No	11	✓	✓	✓		Mortar	Minor*	Mixed
4-SPv-7.5-1	7.5	Fixed-Fixed	Yes	11	✓	✓	✓				Mixed
5-SNv-7.5-2	7.5	Fixed-Fixed	No	11	✓	✓	✓			Minor*	Mixed
6-SPv-7.5-2	7.5	Fixed-fixed	Yes	10	✓	✓					Mixed
7-SPv-15-1	15	Fixed-Fixed	Yes	11		✓	✓	✓ (one side)			Shear
8-SNv-15-1	15	Fixed-Fixed	No	12		✓	✓		✓	Major**	Shear
9-SPv-15-2	15	Fixed-Fixed	Yes	11		✓	✓	✓	Mortar	Minor*	Shear
10-SNv-15-2	15	Fixed-Fixed	No	12		✓	✓		✓	Major**	Shear
11-SNk-15-1	15	Cantilever	No	11		✓	✓	✓ (one side)	Mortar	Minor*	Shear
12-SPk-15-1	15	Cantilever	Yes	11		✓	✓	✓ (one side)	Mortar	Major**	Shear
13-SPk-15-2	15	Cantilever	Yes	11		✓	✓	✓ (one side)	Mortar	Major**	Shear
14-SNk-15-2	15	Cantilever	No	11	✓	✓	✓	✓	Mortar	Major**	Shear

Names of the tests are compiled as follows; letter S for specimen, N or P relate to the morphology (N — without and P — with header stones), k or v to the boundary conditions (k — cantilever and v — double fixed), 5, 7.5 or 15 to the level of pre-compression in % f_{Mc} , and 1 or 2 refer to whether it was the 1st test or repetition, respectively, * Minor — at least one of the W1-W4 LVDT measurements was higher than 0.5 mm, ** Major — at least one of the W1-W4 LVDT measurements was higher than 3 mm

Figure 7. (a) rocking (test 2), (b) shear damage and (c) leaf separation (test 13, connected wall) and (d) leaf separation (test 14, unconnected wall).

conditions. With the shear mechanism, diagonal cracks developed in the mortar and some of the stone blocks (mechanism *b* in Table 2), with staircase cracks prevailing in the mortar joints (Figure 7b). Significant crushing and disintegration of mortar occurred in the central area of the wall, with a loss of mortar from the joints occurring (mechanism *c* in Table 2). Mortar crushing was also

observed in the »toe zones« during the formation of sidewall vertical columns (mechanism *e* and *d* in Table 2, respectively).

Against expectation, the out-of-plane damage mechanism did not prove to be critical. Leaf separation (mechanism *f* in Table 2) occurred along with the shear mechanism, although it was not significant until the softening phases of

Figure 8. Typical hysteresis lateral load — lateral displacement diagrams obtained for tests with various combinations of masonry morphology and testing conditions (different pre-compression levels and boundary conditions).

Figure 9. Hysteresis envelopes of lateral load — lateral displacement curves for tests 1–6 and tests 7–14.

the tests. It was more apparent in the tests which had cantilever boundary conditions, regardless of whether or not the walls were connected. In fixed-fixed boundary conditions, the header stones limited the separation (see Figure 7c, d). This proved to be one of two major advantages in favour of header stones in the type of masonry tested.

3.2.2. Hysteresis response of the walls, idealisation of the results, and analysis

3.2.2.1. Hysteresis response. The most significant information regarding the structural response of the walls is obtained from evaluating the lateral load, in the form of lateral displacement hysteresis diagrams. Figure 8, presents typical lateral load — lateral displacement diagrams (with respect to the LVDT at the bottom of the wall), while Figure 9 presents the hysteresis envelopes for all tests conducted. The average lateral displacement and shear resistance values, obtained at the characteristic limit states in both directions of loading, are given in Table 3. The following characteristic limit states were observed and analysed:

- (1) visual observation of the first shear crack — lateral displacement, d_{cr} , and the corresponding shear resistance, F_{cr} ,
- (2) maximum shear resistance, F_{max} , and the corresponding lateral displacement, d_{Fmax} ,
- (3) maximum lateral displacement, d_{max} , and the corresponding shear resistance, F_{dmax} .

The influence of both the boundary condition and pre-compression level on behaviour is apparent, since, in addition to the failure mechanism obtained, the maximum resistances and ultimate displacement capacities of the walls also differed. A lower resistance was obtained for the

rocking mechanism, while the displacements were significantly higher compared to other prevailing mechanisms. In the mixed mechanism, the resistance increased but the displacement capacity was reduced significantly. The latter is comparable with the displacement capacity obtained via the shear mechanism, although it is slightly higher. The mean values of the characteristic limit states, obtained from tests with the same vertical load and boundary conditions applied to the walls (regardless of morphology), are provided in Table 3.

In terms of displacement capacity, the cantilever boundary conditions were, in most cases, favourable compared to the fixed-fixed boundary conditions for all characteristic limit states (Table 3). This effect was expected, even though the influence of the effective height or shear span length of the specimens (i.e. the moment contra flexure points) on displacement capacity was not as evident as anticipated.

Test no.6 should be commented on separately; it can be seen from Table 3 and Figure 9 that the lateral load-bearing capacity and lateral displacement capacity are significantly lower (almost half) compared to the other two tests with identical boundary conditions and levels of vertical load (i.e. tests no. 3–5), as well as the second specimen with the same morphology (test no.4). This difference can be attributed to the greater thickness of the bed-joints, h_j , in the masonry in test no.6. Instead of the typical 11, or even 12 courses of stone, this wall was constructed from only 10 courses, resulting in a higher h_j (2–3 cm vs. 1–2.5 cm for other walls), which obviously weakened the wall. For this reason, test no.6 is excluded from further comparisons throughout the rest of the paper.

With regard to the influence of morphology, the presence of header stones improved the response of the masonry when the mixed failure mechanism occurred;

Table 3. Lateral displacements and resistance at characteristic limit states for tests with the same boundary conditions, with mean results for tests with the same vertical load and boundary conditions applied.

Limit state		First shear crack				Maximum force				Maximum displacement			
Name		d_{cr} [mm]	mean(CV)	F_{cr} [kN]	mean(CV)	d_{Fmax} [mm]	mean(CV)	F_{max} [kN]	mean(CV)	d_{max} [mm]	mean(CV)	F_{dmax} [mm]	mean(CV)
1-SPk-5-1	Rocking	0.00		0.0		49.6		42.1		49.8		41.2	
1.2-SPk-7.5-1		34.70	26.0(-)	57.7	54.8(-)	69.5	63.8(-)	68.5	65.2(-)	69.7	64.8(-)	66.1	63.2(-)
2-SNk-7.5-1		17.37		51.9		58.1		61.8		59.8		60.3	
3-SNv-7.5-1	Mixed	7.44	8.27*(14)	61.2	65.0*(9)	22.3	25.6*(12)	88.2	105.7*(12)	24.9	28.2*(8)	73.0	77.2*(24)
4-SPv-7.5-1		7.45		60.9		29.7		119.9		29.9		101.7	
5-SNv-7.5-2		9.93		73.0		24.7		109.0		29.9		83.3	
6-SPv-7.5-2		7.45		60.2		9.9		61.9		14.9		50.8	
7-SPv-15-1	Shear	2.98	3.23(34)	81.4	70.1(20)	14.8	14.8(6)	129.2	123.0(3)	19.9	21.1(6)	81.9	91.7(7)
8-SNv-15-1		4.96		85.7		14.9		121.2		22.4		90.3	
9-SPv-15-2		1.98		52.5		16.1		122.1		22.4		93.6	
10-SNv-15-2		2.98		60.9		13.4		119.4		19.9		100.8	
11-SNk-15-1		2.98	3.48(25)	68.3	72.7(10)	15.0	18.1(25)	114.5	118.7(4)	22.4	24.9(24)	98.6	103.0(4)
12-SPk-15-1		2.98		65.8		15.2		114.0		22.4		100.9	
13-SPk-15-2		2.98		72.2		16.2		122.5		19.9		104.7	
14-SNk-15-2		4.97		84.5		26.0		123.7		34.9		108.0	

*Test 6-SPv-7.5-2 is not considered in the calculation of mean values due to the increased horizontal bed joint thickness in this test.

for lower pre-compressions and fixed boundary conditions the resistance was 22% higher in the masonry with header stones, while the ultimate drift capacities were similar for both types of morphology. The higher shear resistance obtained in walls with header stones when mixed mode develops likely results from an increased flexural resistance due to the higher compressive strength/ stiffness of the wall; connecting stones decrease the mortar/ unit ratio and consequently increase the compressive strength of the walls, as well as significantly reduce the leaf separation before the header stones crack, thus enabling a higher resistance.

At higher levels of pre-compression, when the shear mechanism developed, no significant or consistent benefit of the header stones was found with respect to the resistance and drift capacity of the walls. It should be noted that the slightly higher shear resistance and higher vertical load (4.4%) observed in the connected walls under fixed-fixed boundary conditions cannot be solely assigned to the presence of header stones, since this could also be attributed to the smaller testing sample. The results for walls with the same morphology are summarised in Table 4, where the average values for all test repetitions are provided.

3.2.2.2. Idealisation of the hysteresis envelopes. For the purposes of further evaluation and comparison of the test results, the hysteresis envelopes obtained were idealised to bi-linear curves. It should be noted that many different criteria for idealisation can be found in the literature (Bosiljkov et al. 2003; Magenes and Calvi 1997; Tomaževič 1999). In order to idealise the hysteresis envelopes, an equal energy input was assumed between the idealised and experimental hysteresis envelope curve in order to determine the idealised shear

resistance F_{id} . The ultimate displacement, d_u , was limited to the displacement at which the shear resistance decreased to 80% F_{max} , or to the displacement at which it was estimated that continuing the tests would represent a serious hazard due to the sudden collapse of severely damaged specimens, which might jeopardize the components of the test-setup. In the latter situation, d_u corresponds to the maximum displacements obtained. By studying different selection criteria to determine the effective stiffness, K_{ef} , the smallest dispersion (with respect to CV) was obtained when the stiffness was evaluated at a given shear resistance. For the analysis presented, K_{ef} was set to a stiffness value of $2/3 F_{max}$ (see Figure 10).

The idealised results in terms of K_{ef} do not differ with respect to the boundary conditions (Table 5), as they would according to the theory of elasticity; i.e. a higher K_{ef} in the case of a double fixed boundary condition compared to that of a cantilever boundary condition. It can therefore be concluded that the K_{ef} is highly

Figure 10. Bi-linear idealisation of the hysteresis envelope.

Table 4. Mean displacements and resistances at characteristic limit states for tests with the same testing conditions and morphology.

	First shear crack				Maximum force				Maximum displacement			
	d_{cr} [mm]		F_{cr} [kN]		d_{Fmax} [mm]		F_{max} [kN]		d_{max} [mm]		F_{dmax} [kN]	
Morphology Header stones present	Yes	No	Yes	No	Yes	No	Yes	No	Yes	No	Yes	No
7.5% f_{Mc} cantilever	34.7*	17.4*	57.7*	51.9*	69.5*	58.1*	68.5*	61.8*	69.7*	59.8*	66.1*	60.3*
7.5% f_{Mc} fixed-fixed	7.45**	8.69	60.9**	67.1	29.7**	23.5	119.9**	98.6	29.8**	27.4	101.7**	78.1
15% f_{Mc} fixed-fixed	2.48	3.97	66.9	73.3	15.5	14.1	125.7	120.3	21.1	21.1	87.7	95.6
15% f_{Mc} cantilever	2.98	3.97	69.0	76.4	15.7	20.5	118.2	119.1	21.1	28.6	102.8	103.3

*Only one test specimen, maximum resistance and displacement were not achieved; ** Only one test specimen (test 6 excluded).

Table 5. Average results of idealisation for tests with the same testing conditions (CV in % in brackets).

Pre-compression level, Boundary conditions	F_{id} [kN](CV)	d_e [mm](CV)	K_{ef} [kN/mm](CV)	d_u [mm](CV)	μ (CV)
7.5% f_{Mc} cantilever	57.8 (3)	8.4 (15)	7.0 (12)	64.8 (8)	7.80 (7)
7.5% f_{Mc} fixed-fixed	100.2 (12)	13.2 (16)	7.8 (8)	28.2 (8)	2.23 (12)
15% f_{Mc} fixed-fixed	113.7 (2)	5.9 (21)	21.2 (22)	21.0 (5)	3.92 (18)
15% f_{Mc} cantilever	112.6 (4)	5.6 (10)	20.4 (7)	24.9 (24)	4.46 (15)

A 20% decrease in resistance was not achieved in all tests; in such cases, maximum displacements were considered as ultimate displacements.

dependent on the damage mechanism developed, both in terms of the boundary conditions determined by the outer constraints as well as by the type of masonry i.e. the presence of soft mortar. Determining the stiffness of masonry elements analytically, while considering masonry as an isotropic solid material and the boundary conditions applied to structural elements, may be a debatable issue in the case of historic masonry. Considering the elastic and shear modulus obtained from compression tests in the walls tested, for example, the analytical K_{ef} was estimated as 21.5 kN/mm and 49.0 kN/mm under the cantilever and fixed-fixed boundary conditions, respectively. A comparison of the analytical and experimental results shows that analytical K_{ef} corresponds well for the experimental tests at the higher vertical load under the cantilever boundary condition (with an experimental average value 20.4 kN/mm). A similar value of K_{ef} was, however, also obtained from the experiments with double fixed boundary conditions (21.2 kN/mm).

One of the most commonly used indicators of non-linear behaviour when subjected to lateral loading is the ductility coefficient, μ — a characteristic of the idealised curve, defined as the ratio between ultimate (d_u) and elastic (d_e) displacement. Elastic displacement, d_e , is calculated from F_{id} and K_{ef} (all presented in Table 5).

It can be seen that the cantilever boundary conditions produced a higher μ , which was mainly due to a more pronounced flexural failure mechanism. μ was, on average, 286% higher in walls with the rocking mechanism compared to those with the shear mechanism, and more than three times higher than that of the mixed mechanism walls. Such a low μ for the mixed mechanism is a result of the larger d_e obtained with the adopted idealisation criterion for this type of failure mechanism compared to both the rocking and shear mechanisms (d_e is more than double compared to for the shear mechanism). The mixed mechanism proved to be the least favourable in terms of μ , despite the ultimate displacements being 23% higher than in the tests where the shear mechanism developed.

3.2.2.3. Tensile strength. The tensile strength of the masonry (f_{Mt}) was calculated using the average F_{id} of both directions of loading. It was calculated as the critical value of principal stress in the centre of the pier,

Figure 11. Tensile strength, f_{Mt} , in the walls tested.

where diagonal shear failure occurs (Equation 2, (Turnšek and Čačovič 1971)), considering normal stress at the mid-height of the wall (σ_0), average maximum shear stress (τ) and coefficient b indicating the shear stress distribution (depending on the geometry aspect ratio of the wall):

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

The f_{Mt} results, presented in Figure 11, show the influence of the pre-compression level and the damage mechanism developed on f_{Mt} . In general, higher f_{Mt} values were obtained in the tests where the mixed mechanism occurred (Table 6), with the average f_{Mt} 30% higher than that obtained from tests where a (pure) shear mechanism occurred. Comparing the results with the minimum and maximum recommended values for this type of masonry according to the Italian code provisions NTC 2018 (NTC 2018 2018), shows that they are within the expected range for tests where a shear mechanism is obtained.

3.2.2.4. Stiffness characteristics. The shear modulus, G_M , was for each test calculated from K_{ef} and E_M (the latter evaluated from the compression tests) according to Equation (3) (Tomažević 1999) (the average G_M for tests with the same testing conditions are presented in Table 6). The coefficient ψ in Equation (3) considers the boundary conditions, and is equal to 4 for the cantilever and 1 for the fixed-fixed boundary conditions.

Table 6. Average f_{Mt} and G_M , obtained through various methods, for tests with the same testing conditions.

Pre-compression level, boundary conditions	f_{Mt} [MPa](CV)	G_M [MPa](CV)	* G_M [MPa](CV)	** G_M [MPa](CV)
7.5% f_{Mc} , cantilever	/	55.1/	123.5/	123.5/
7.5% f_{Mc} , fixed-fixed	0.214 (17)	38.7 (9)	57.2 (8)	123.1 (8)
15% f_{Mc} , fixed-fixed	0.165 (3)	128.5 (30)	154.8 (22)	333.4 (22)
15% f_{Mc} , cantilever	0.162 (7)	−26,797 (190)	321.7 (7)	321.7 (7)

*Calculated according to Equation (3) considering $E_M = 3 G_M$ and ψ equal to 4 and 1 for cantilever and fixed-fixed boundary conditions, respectively.

**calculated according to Equation (3) considering $E_M = 3 G_M$ and ψ equal to 4 for both cantilever and fixed-fixed boundary conditions.

$$G_M = \frac{K_{ef}}{\frac{l_w \cdot t_w}{1.2 \cdot h_w} - \frac{\psi \cdot K_{ef}}{1.2 \cdot E_M} \left(\frac{h_w}{l_w} \right)^2} \quad (3)$$

The G_M values, calculated according to Equation (3), were in some cases unrealistic (producing negative values), which is a consequence of using the elastic modulus obtained from compression tests on intact masonry, and considering the effective stiffness corresponding to cracked masonry. In order to evaluate G_M from effective stiffness, a secant modulus of elasticity of $E_M = 3 \cdot G_M$ (Bosiljkov, Totoev, and Nichols 2005) is presumed, enabling a more stable calculation dependent on various values of K_{ef} (labelled as $*G_M$ in Table 6). Since the numerous boundary conditions applied proved not to significantly influence K_{ef} , however, it seems that, due to the soft mortar, the cantilever response is dominant even though fixed-fixed boundary conditions are applied. G_M was therefore recalculated for tests with fixed-fixed boundary conditions by setting the value to $\psi = 4$ in Equation (3) (labelled $**G_M$). Through this approach, the results for the shear modulus are more consistent for both types of boundary condition and in accordance with the results obtained from compression tests.

3.2.2.5. Analysis of displacement and shear resistance response in various characteristic limit states. To better understand the shear response with regards to different failure mechanisms, a comparison of various average characteristic limit state values, presented according to the prevailing failure mechanism, is shown in Table 7. The formation of the first shear cracks (d_{cr}) occurred as expected; firstly in the walls where the pure shear mechanism was activated and lastly, if at all, in the walls where the rocking mechanism prevailed. Additionally, the point at which maximum resistance was reached ($d_{F_{max}}$) occurred earlier relative to the failure of the wall (d_{max}) in the case of the shear mechanism (at 71.4% d_{max}) when compared to the mixed (at 90.7% d_{max}) and rocking mechanisms (at 98.8% d_{max}).

Shear resistance at the first shear crack (F_{cr}) ranged from 43.0% to 97.2% of maximum resistance F_{max} , with the latter value obtained in test no.6. Changing the failure mechanism from rocking to shear caused a reduction in the F_{cr} relative to F_{max} , which was, on average, 59.0% F_{max} for each shear mechanism (Table 7).

Table 7. The average ratios between values of lateral displacement or shear resistance at various characteristic limit states, according to the prevailing failure mechanism.

Failure mechanism	$d_{cr} / d_{F_{max}}$	$d_{F_{max}} / d_{max}$	$d_e / d_{F_{max}}$	F_{cr} / F_{max}	$F_{d_{max}} / F_{max}$	$F_{d_{max}} / F_{id}$
Rocking	0.399	0.988	0.131	0.841	0.972	0.915
Mixed	0.328	0.907	0.515	0.624	0.813	0.949
Shear	0.206	0.714	0.357	0.602	0.808	0.937

This explains the common choice of evaluating effective stiffness as $2/3$ of F_{max} when calculating the bi-linear curve in the case of a shear mechanism.

A decrease of up to 36.6% (test no.7) was observed when comparing the post-peak resistance $F_{d_{max}}$ to F_{max} . This drop refers to the 3rd cycles of loading. Again, in general the highest drops occurred with the shear mechanism (19.2% on average), whereas with the rocking mechanism this decrease was negligible (2.8%). The idealised resistance, F_{id} , was at least 90% of F_{max} , with the highest ratios obtained for the mixed mechanism (94.9% on average).

3.2.2.6. Energy dissipation and the equivalent viscous damping coefficient. The dissipated and input energy in each cycle of a given amplitude displacement were calculated for every test. The dissipated (potential) energy at each loading cycle ($E_{DIS,i}$) was calculated as the area within one complete hysteresis cycle (Figure 12a), while $E_{INP,i}$ for a given i th cycle was calculated as the sum of the areas under the positive and negative parts of the hysteresis loop (Figure 12b). The dissipated vs. input energy ratios E_{DIS}/E_{INP} for the 1st cycle of each amplitude displacement (average of the two “half” cycles) for all tests are presented in Figure 13.

The influence of the testing conditions and, consequently, of the different failure mechanisms on the results, is apparent. At d_{cr} and d_e , the E_{DIS}/E_{INP} increases from a lower pre-compression level to a higher level and from cantilever to fixed-fixed boundary conditions. At $d_{F_{max}}$ and d_{max} , however, the values vary significantly according to the failure mechanism; the average E_{DIS}/E_{INP} values at $d_{F_{max}}$ were 1.87%, 5.24% and 4.96% for the rocking, mixed and shear mechanisms respectively. With the shear mechanism, the values are similar regardless of the testing conditions. It is also evident from Figure 14 that the amount of dissipated energy depends on the shear mechanism developed — for each test the ratios of cumulative dissipated to cumulative input energy $cumE_{DIS}/cumE_{INP}$ at d_{max} are plotted against the vertical load imposed. The use of header stones in the connected walls proved to be beneficial in terms of the energy dissipated under the same testing conditions, although the difference was only small.

Energy dissipation is primarily dependent on the damage mechanisms developed. This is also apparent from the calculations of equivalent viscous damping coefficients ξ . The average values of ξ at characteristic points of the hysteresis response in tests with the same boundary conditions are presented in Table 8. Contrary to the E_{DIS}/E_{INP} ratio, which was with the shear mechanism higher under fixed-fixed boundary conditions, the ξ values at $d_{F_{max}}$ and d_{max} were smaller when fixed-fixed compared to the cantilever boundary conditions. This is due to a different definition of the

Figure 12. (a) Dissipated energy in one loading cycle; (b) Input energy of one cycle; (c) Dissipated and input energy in each cycle as defined for the calculation of the equivalent viscous damping coefficient.

Figure 13. Ratio of dissipated to input energy in the 1st cycle of each amplitude displacement in tests 1–6 and 7–14.

Figure 14. Ratio of cumulative dissipated to cumulative input energy according to the pre-compression load (for various testing conditions and morphologies).

input energy (E_{INP}^*) for the calculation of ξ (Equation 4), where E_{INP}^* is considered as depicted in Figure 10d (Chopra 1995).

$$\xi = \frac{1}{2\pi} \frac{E_{DIS}}{E_{INP}^*} = \frac{1}{2\pi} \frac{E_{DIS}}{(E_{INP}^+ + E_{INP}^-)} \quad (4)$$

4. Discussion and conclusions

The structural behaviour of three-leaf stone masonry walls was studied through the results of experimental tests on 18 plastered specimens, built to represent historical masonry. Altogether, 4 compression tests (2 wall-ettes and 2 walls) were conducted, followed by 15 cyclic

Table 8. Average equivalent viscous damping coefficient ξ values at characteristic limit states for tests with the same testing conditions.

Pre-compression level boundary conditions	d_{cr} ξ (CV)	d_e ξ (CV)	d_{fmax} ξ (CV)	d_{max} ξ (CV)
7.5% f_{Mc} cantilever	5.6% (3)	7.6% (4)	4.7% (2)	4.7% (2)
7.5% f_{Mc} fixed-fixed	9.8% (8)	9.0% (5)	9.8% (2)	10.4% (8)
15% f_{Mc} fixed-fixed	12.4% (4)	12.6% (9)	11.3% (4)	12.7% (4)
15% f_{Mc} cantilever	12.0% (6)	11.9% (6)	11.9% (19)	14.9% (13)

in-plane shear tests on 14 walls, accompanied by experimental tests on the masonry constituents. The influence of the masonry morphology (with or without header stones) and the boundary conditions of the walls on seismic performance was studied by applying different pre-compression levels (5%, 7.5% and 15% f_{Mc}) and boundary conditions (cantilever vs. fixed-fixed).

The compression tests for the type of masonry tested (thickness of the inner core < 25% of the overall thickness of the masonry) proved that the connected and unconnected walls exhibit similar behaviour. The main reason for the similar compressive strengths obtained for both connected and unconnected wall might be the increased thickness of the horizontal bed joints in the connected wall compared to the unconnected wall, i.e. the positive effect of the connecting stones on compressive strength was reduced by the influence of increased joint thickness. Leaf separation occurred earlier in the unconnected walls, but was equally as intensive in the connected walls once the header stones had cracked. This effect is the consequence of the typical failure mechanism of single leaf unreinforced masonry under compression, where groups of continuous vertical cracks form through units across multiple rows once the first masonry unit cracks (Bosiljkov 2006). Leaf separation did not significantly influence the compressive strength of the wall, as a similar compressive strength was achieved in both types of masonry. This can be explained by the governing in-plane damage mechanism, which occurred on both walls and wallettes. Failure mainly occurred due to crack propagation along the mortar joints (both the bed and head joints). Due to the soft/weak mortar, significant strain (i.e. 5–10‰) was obtained from the compression tests for both masonry types. On the other hand, the influence of the mortar to unit volume ratio (i.e. the influence of the thickness of the mortar joints) might have affected the stiffness characteristics, since higher stiffness characteristics were obtained for the unconnected wall, which had 11 rows of stone compared to 10 rows in the connected wall.

By varying the testing conditions in the cyclic shear tests, various failure mechanisms in the masonry walls were obtained. Lower pre-compression (5.0, 7.5% f_{Mc}) and cantilever boundary conditions provoked rocking. Restraining the rotations enabled a diagonal shear mechanism to

partially develop; this failure mechanism was classified as mixed. A high pre-compression level (15% f_{Mc}) induced diagonal shear behaviour in both boundary conditions, with some toe crushing and the formation of vertical columns along the edges. The type of mechanism activated had the greatest effect on the strength and displacement capacity of the walls. Other characteristic parameters of the walls' in-plane shear hysteresis response (i.e. effective stiffness and shear modulus, energy dissipation, etc.) also differed in response to the damage and failure mechanism developed.

Out-of-plane deformations of the outer leaves seemed not to be as problematic as initially expected. Leaf separation occurred with development of the shear mechanism, but it progressed significantly in the post-peak phases of the tests (i.e. the softening phase). Leaf separation was more apparent under cantilever boundary conditions, regardless of whether or not header stones were present — which differed from the fixed-fixed boundary conditions, where separation of the leaves was limited when header stones were present. It was concluded from the in-plane shear tests that, apart from the aforementioned effect of limiting the post-peak leaf separation, increasing the wall's flexural resistance (by increasing its compressive strength), and providing slightly greater dissipation of energy, the presence of header stones did not have a significant influence on the in-plane shear response of the masonry type tested. It seems that, for such walls, in the case of a shear mechanism the behaviour of walls as single-leaf walls (due to the compact core) is more beneficial than the presence of header stones, since the in-plane failure of walls occurs mainly through the soft mortar, regardless of the presence of header stones. Italian norms (NTC 2018 2018) set a table (C8A.2.2) providing coefficients to correct mechanical parameters according to the quality of the mortar, the presence of courses or borders, and the presence of systematic transverse connections, mortar injections etc. For dressed rectangular stone masonry with transverse connections, this coefficient is set to 1.2. If the wall has a compact core, the shear response of walls as single-leaf masonry may be expected, as in the case of grouted walls (Silva et al. 2014b). The application of an additional correction coefficient due to the transverse connections (as proposed by the Italian code) may, however, be questionable for all mechanical parameters if the monolithic response of the walls is already attained by grouting. An additional improvement in compressive strength could be achieved due to the combined effect of both the grouting (compact core) and transverse connections, as concluded by (Valluzzi, Da Porto, and Modena 2004).

For the type of masonry morphology tested, the presence of header stones seems to have a negligible influence on the ultimate in-plane lateral deformation capacities; the boundary conditions applied and mechanisms of failure obtained are far more important.

Specimens that failed due to rocking exhibited a 3.99% drift on average, compared to 1.88% and 1.53% for the mixed and shear mechanisms, respectively. These values are substantially higher than the values proposed in the code provisions (e.g. values of 0.53% and 1.6% for shear and flexural damage mechanisms for near collapse damage state according to EN1998-3 (EN 1998-3: 2005 2005). Brittle failure did not occur in the case of the shear mechanism; the walls exhibited a considerable post-peak displacement capacity and, on average, $d_{\max}$ was equal to $1.40 d_{F\max}$. The rocking mechanism exhibited very ductile behaviour with a slight decrease in strength (2.8% on average); the mixed mechanism, on the other hand, induced a more brittle failure (average $d_{\max}$ equal to $1.10 d_{F\max}$). The post-peak resistance decrease (at the same amplitude) was highest with the shear mechanism (19.8% on average), but it differed significantly with respect to the type of boundary conditions applied; the average value was 35.2% in the fixed-fixed and 13.2% under cantilever boundary conditions.

Considering energy dissipation, the (diagonal) shear failure proved to be favourable compared to the rocking behaviour; the average equivalent viscous damping coefficients ξ at $d_{F\max}$ were 4.7%, 9.8% and 11.6% with rocking, mixed and shear mechanism, respectively.

It should also be mentioned that a significantly lower shear resistance and displacement capacity were obtained in test no.6 (f_{Mt} 0.087 MPa compared to an average of 0.163 MPa in the other walls; 1.00% drift compared to 1.53%), as a consequence of the thicker horizontal mortar joints. A difference in the shear behaviour was, however, observed when the horizontal mortar joint thickness was increased beyond a certain limit (i.e. the specimen with 10 rows of stones); the shear response of the walls with 12 rows of stones was similar to the response of walls with 11 rows of stones. This emphasizes the importance of evaluating the stone/mortar volume ratio prior to adopting the mechanical properties of the masonry for design purposes, and also the sensitivity of the masonry's behaviour to a parameter that is not usually considered when determining the masonry's characteristics.

Calculating the shear modulus, G_M , from the shear tests proved to be problematic, as previously discussed (Bosiljkov, Totoev, and Nichols 2005). Evaluating G_M from the effective stiffness, K_{ef} , considering the modulus of elasticity, E_M (as obtained from compression tests), produced unrealistic results in some cases; G_M was negative for the higher pre-compression level under cantilever boundary conditions, while the average G_M for fixed-fixed boundary conditions was 128.5 MPa. In order to estimate G_M from shear tests, a realistic relationship between G_M and E_M should be considered, e.g. $G_M = 0.33 E_M$. Through the use of this ratio, more

realistic values were obtained for the higher pre-compression level under cantilever boundary conditions. Furthermore, comparison shows that cantilever boundary conditions are more realistic also for tests with fixed wall rotations, since rotations are obtained through rotations of the masonry units, enabled by the soft mortar being crushed.

The results presented herein may serve as input parameters for the numerical analysis of structures constructed from a similar type of masonry. More importantly, they may serve for the critical assessment of existing and forthcoming analytical models and the deformation performance limits for masonry structures proposed in the second generation of Eurocodes (Kržan and Bosiljkov 2021).

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