Editorial Manager(tm) for Materials and Structures
Manuscript Draft

Manuscript Number:

Title: Robustness as a criterion for use of hollow clay masonry units in seismic zones

Article Type: Scientific Report

Keywords: hollow clay masonry units; robustness; seismic behavior of walls; ductility and energy dissipation capacity; testing; criteria

Corresponding Author: Prof Miha Tomazevic, Ph.D.

Corresponding Author's Institution: Slovenian National Building and Civil Engineering Institute

First Author: Miha Tomazevic, Ph.D.

Order of Authors: Miha Tomazevic, Ph.D.; Polona Weiss, M.Sc.

Abstract: In order to provide adequate seismic behavior of masonry walls, local brittle failure of masonry units in the most stressed zones of structural walls should be prevented. Although robust behavior is required by the code, no specifications are given regarding the criteria to fulfill this requirement. To propose such criteria, a series of 22 masonry walls, built with 6 different types of hollow clay masonry units, currently available on the market, have been tested by subjecting them to cyclic lateral load at two levels of constant precompression. Besides, the strength characteristics of the units, like compressive strength orthogonal and parallel to the bed joints and tensile and shear strength of the units have been determined by standardized and specifically designed testing procedures. By correlating the parameters of seismic resistance of the walls and strength characteristics of the units, no specific indicator for robustness could have been determined on the basis of the mechanical characteristics of the tested units. It has been found that in all cases the level of precompression, i.e. the ratio between the compressive stresses in the walls and the compressive strength of masonry, represents the governing parameter.

ROBUSTNESS AS A CRITERION FOR USE OF HOLLOW CLAY MASONRY UNITS IN SEISMIC ZONES

Miha Tomaževič* and Polona Weiss

Slovenian National Building and Civil Engineering Institute
Dimičeva 12, 1000 Ljubljana, Slovenia

Corresonding author*:
Phone: +386 1 2804 400
Fax: +386 1 2804 484
E-mail: miha.tomazevic@zag.si

ABSTRACT

In order to provide adequate seismic behavior of masonry walls, local brittle failure of masonry units in the most stressed zones of structural walls should be prevented. Although robust behavior is required by the code, no specifications are given regarding the criteria to fulfill this requirement. To propose such criteria, a series of 22 masonry walls, built with 6 different types of hollow clay masonry units, currently available on the market, have been tested by subjecting them to cyclic lateral load at two levels of constant precompression. Besides, the strength characteristics of the units, like compressive strength orthogonal and parallel to the bed joints and tensile and shear strength of the units have been determined by standardized and specifically designed testing procedures. By correlating the parameters of seismic resistance of the walls and strength characteristics of the units, no specific indicator for robustness could have been determined on the basis of the mechanical characteristics of the tested units. It has been found that in all cases the level of precompression, i.e. the ratio between the compressive stresses in the walls and the compressive strength of masonry, represents the governing parameter.

Keywords:

Hollow clay masonry units; robustness; seismic behavior of walls; ductility and energy dissipation capacity; testing; criteria

1. INTRODUCTION

To improve the thermal insulation and load bearing properties of masonry walls, new types of masonry units and improved masonry construction technologies have been developed. Solid bricks have been replaced by hollow units, the shape and materials of which have been designed to meet the demanding energy saving criteria for buildings with minimum additional thermal insulation layers. At present, hollow clay units are usually made of specially developed porous clayey materials. Units with large percentage of voids, thin shells and webs are available on the market. Whereas the load bearing capacity of masonry walls made of such units is adequate for gravity loads, experimental investigations indicated, that the units exhibit local brittle failure if the walls are subjected to in-plane horizontal seismic loads and high level of compressive stresses at the same time [1].

In order to prevent local brittle failure of masonry units, European standard for the design of earthquake resistant structures, Eurocode 8 [2], requires that "masonry units should have sufficient robustness in order to prevent local brittle failure." Whereas the ENV version of the standard determined the maximum volume of holes and minimum allowable thickness of shells and webs for the units for the intended use in seismic zones, no indication is given regarding the properties of robust units in the current standard. The decision on that is left to the National Annexes, which "may select the type of masonry units from EN 1996-1: 2004, Table 3.1 that satisfy this requirement." However, the decision is not simple, because according to Table 3.1, Group 2 clay masonry units are all units, where the volume of holes varies from 25 % to 55 % of the gross volume of the unit, and the thickness of shells and webs is not less than 8 mm and 5 mm, respectively [3].

To identify and quantify the criteria which determine "sufficient robustness", a comprehensive experimental study has been carried out at Slovenian National Building and Civil Engineering Institute, ZAG, in Ljubljana, Slovenia [4]. The results of this study will be discussed and the conclusions drawn presented in this contribution.

2. EXPERIMENTAL PROGRAM

Since all brick manufacturers are following the same goal of optimization of properties of structural masonry regarding the load bearing capacity and thermal insulation, the basic strength and geometrical characteristics of hollow clay masonry blocks, available on the market, do not vary significantly. Consequently, it was not easy to find the units, where the parameters, which influence the robustness of the units (such as percentage of holes, thickness of shells and webs), would be spread in a relatively wide range of values. Nevertheless, five different types of hollow blocks, all belonging to Group 2 masonry units according to Eurocode 6, have been considered interesting enough to be included in the program of testing. In addition to them, the units, representing perforated bricks of double height, belonging to Group 1 units, have been also tested as obvious representatives of robust units.

The experimental program has been carried out in two phases. In the first phase of testing, the mechanical and geometrical characteristics of all types of units have been determined by standardized testing procedures. Then, a series of specific tests has been carried out, by means of which the stress state and failure mechanism of a single unit have been simulated for the case that the units are part of a shear wall, subjected to a combination of vertical load and shear in the case of an earthquake. Namely, when subjected to seismic loads, bending and shear forces develop in structural walls in addition to compression, which may cause crushing and buckling of shells and webs of the units, and, consequently, severe deterioration of ductility and energy dissipation capacity of structural walls [1, 8].

To define the mechanical properties, which determine the behavior and load bearing capacity of the units under a combination of compression and shear, three kinds of tests have been carried out:

- Diagonal compression tests of units to determine the tensile strength of units;
- Splitting tests of units to determine the tensile splitting strength of units, and
- Shear tests of units to determine the shear strength of units.

Whereas 6 units of each type have been tested for compressive strength, as required by the standard [5], 5 units of each type have been tested in the case of each kind of the additional tests. In the case of the shear tests, the units have been subjected to vertical load, causing vertical stresses equal to about 1/3 of the units' compressive strength.

Table 1. Test matrix for testing the masonry walls

Unit	Cyclic shear tests			Compression tests
	Precompression level (σ_o/f_c)			
	0.30	0.20	0.15	
B1	2 walls	-	2 walls	2 walls
B2	2 walls	1 wall	1 wall	2 walls
B3	2 walls	-	2 walls	2 walls
B4	2 walls	2 walls	-	2 walls
B5	3 walls	-	3 walls	2 walls
B6*	2 walls	-	2 walls	1 wall
B6t**	1 wall	-	1 wall	1 wall

* Units type B6 are laid transversally, thickness of walls 24.9 cm
** Units type B6 are laid longitudinally, thickness of walls 12.3 cm.

In the second phase of the program, 28 walls, made with all six types of masonry units, have been tested by subjecting them to a combination of constant vertical and cyclic lateral load. The level of precompression has been chosen by taking into consideration the possible ranges of working stress levels in masonry walls due to vertical loads in an actual structure. Considering the value of partial safety factor for masonry for the case of the uncertain quality control conditions, γ_M = 3, approximately 30 % of the characteristic compressive strength of the masonry can be utilized for the vertical load bearing capacity of masonry walls. This value has been considered as the level of precompression, critical for the brittle local failure of units. To determine the compressive strength of masonry and adjust the precompression level, 2 walls of each masonry type have been tested. The test matrix is given in Table 1.

It has been expected that by correlating the values of the observed parameters of the seismic behavior of the tested walls with the relevant mechanical properties of masonry units, the basic relationships and definitions regarding the robustness of masonry units will be obtained.

3. MASONRY UNITS

Masonry units, used in the study, are shown in Figures 1 and 2. Whereas the hollow blocks belonged to declared strength class B10 (compressive strength 10 MPa), the perforated bricks belonged to strength class B20 (compressive strength 20 MPa) masonry units. The basic geometrical properties of the units are given in Table 2, whereas their strength and other properties are presented in Table 3. All properties have been determined in accordance with European standards (EN 772-1:2002 [5], EN 772-3:1999 [6], EN 772-13:2002 [7], and EN 772-16:2002 [8])

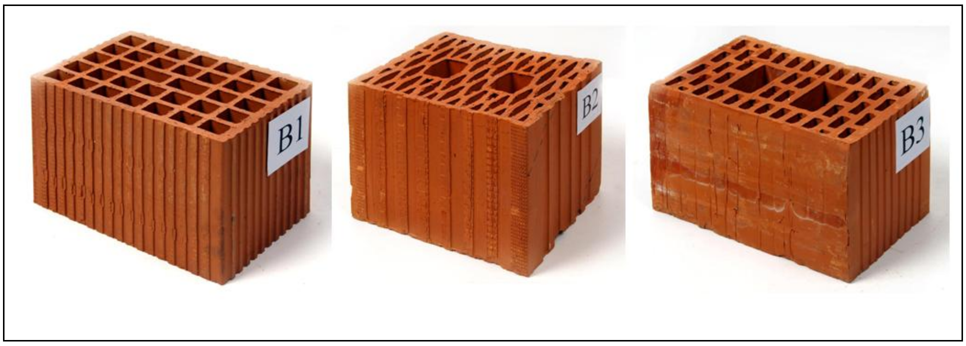

Figure 1. Hollow clay blocks B1, B2 and B3

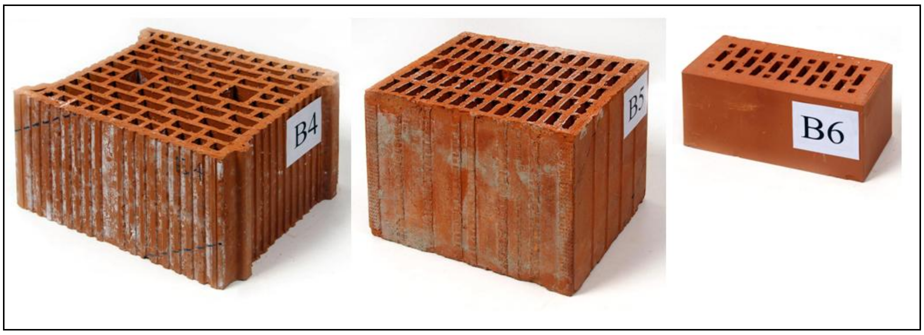

Figure 2. Hollow clay blocks B4 and B5, and perforated brick B6

As can be seen, the geometrical and physical properties of blocks B1 – B5 do not vary significantly, while the properties of the perforated, double height clay brick B6 are quite different.

Table 2. Geometrical properties of the tested masonry units

Designation of unit	B1	B2	B3	B4	B5	B6
Length, l (mm)	188	238	189	331	244	254
Width, w (mm)	288	282	292	292	297	122
Height, h (mm)	189	234	188	189	236	121
Volume of holes (%)	58	55	53	54	51	25
Thickness of shells (mm)	9.8	10.8	11.4	11.7	11.8	21.6
Thickness of webs (mm)	6.5	6.7	7.2	7.4	6.8	7.3
Combined thickness of shells and webs - transversal (% of width)	20	41	35	33	35	46
Combined thickness of shells and webs - longitudinal (% of length)	24	18	24	21	24	48

Table 3. Mechanical and other physical properties of the tested masonry units

Designation of unit	B1	B2	B3	B4	B5	B6
Mean compressive strength of unit, $f_{b,m}$ (MPa)	18.2	11.4	12.8	11.4	10.2	29.1
Shape factor, δ	1.14	1.14	1.14	1.07	1.13	1.04
Normalized compressive strength of unit, f_b (MPa)	20.7	13.0	14.6	12.2	11.5	30.3
Compressive strength of unit parallel to bed joints, $f_{b,h}$ (MPa)	5.0	3.0	1.5	3.8	5.9	16.0
Gross density (kg/m^3)	806	811	880	863	860	1446
Net density (kg/m^3)	1941	1798	1860	1866	1756	1925
Water absorption coefficient (%)	11.0%	11.6%	14.2%	13.4%	13.8%	11.4%

The layout of diagonal compression, splitting, and shear tests of individual units is shown in Figures 3, 4, and 5, respectively. In the case of the diagonal compression tests, steel angles (loading shoes) have been placed at the top and bottom of the specimen, with supporting length not exceeding 20 % of the unit's height. Gypsum was used to ensure good contact between the loading shoes and the unit at both corners. The diagonal tensile strength of the unit, $f_{bt,d}$, has been calculated by equation:

$$f_{bt,d} = \frac{0.707\ V}{A_{w,d}}, \qquad (1)$$

the splitting tensile strength of the unit by equation:

$$f_{bt,s} = \frac{2\,V}{\pi\,h\,w}, \tag{2}$$

and the shear strength of the unit by equation:

$$f_{bs} = \frac{H}{l\,w}, \tag{3}$$

where:

$f_{bt,d}$, $f_{bt,s}$, and f_{bs} = the diagonal tensile strength, splitting tensile strength and shear strength of the unit, respectively,
V = the vertical load at failure of the unit,
H = the shear load at failure of the unit,
$A_{w,d}$ = the area of the cross section of the unit along the diagonal,
l, h, and w = the length, height, and width of the unit, respectively.

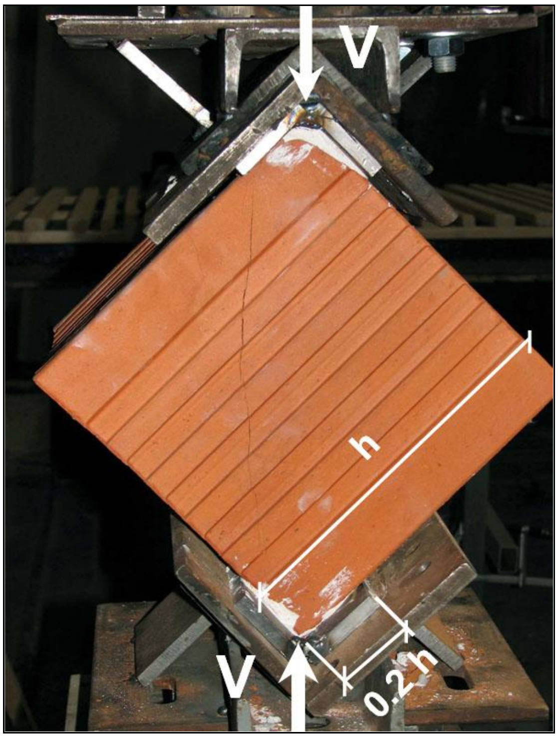

Figure 3. Diagonal compression test of unit B2

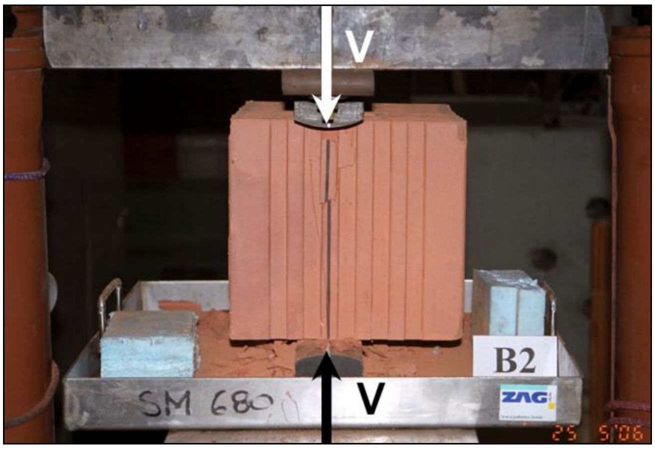

Figure 4. Splitting test of unit B2

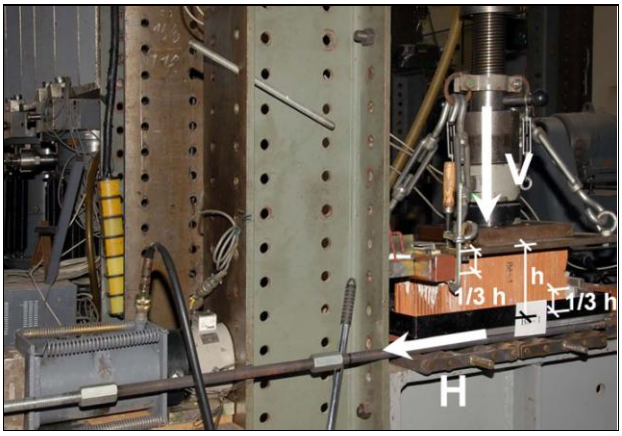

Figure 5. Disposition of shear test

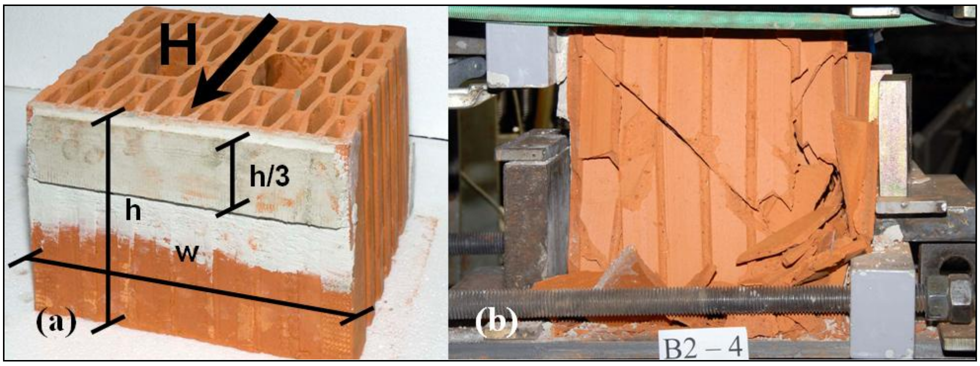

Figure 6. Hollow block type B2 ready for shear test (a) and (b) hollow block type B2 after the shear test

No preparation of units tested by vertical splitting was needed. In the case of the specimens, tested for shear, about 10 mm thick layer of cement mortar has been placed on the contact areas along the units' width, 1/3 of the units' height high, in order to uniformly induce the shear force into the unit, without local crushing of protruding parts of the units' shells (Figure 6a). In the case of the shear tests, all units have been subjected to vertical precompression equal to 30 % of the mean compressive strength of the type of the unit under consideration. The units have been laterally fixed at the top and supported on the roller bearings at the bottom to reduce friction.

Whereas splitting of specimens across the compression plane typically occurred during the diagonal compression and splitting tests, the failure plane of the units, tested for shear, was in most cases oriented diagonally (see Figure 6b). In a few cases, the units crushed.

The results of diagonal compression, splitting, and shear tests are summarized in Table 4. Average values obtained by testing 5 specimens of the same type by different testing methods, are given in the table. Data regarding the volume of holes and compressive strength of the units orthogonal and parallel to bed joints are included to complete the information.

Table 4. Mean values of mechanical properties of the tested masonry units

Unit	Volume of holes (%)	$f_{b,m}$ (MPa)	$f_{b,h}$ (MPa)	$f_{bt,d}$ (MPa)	$f_{bt,s}$ (MPa)	f_{bs} (MPa)
B1	58	18.2	5.0	0.57	0.35	2.02
B2	55	11.4	3.0	0.88	0.59	2.28
B3	53	12.8	1.5	0.63	0.59	2.12
B4	54	11.4	3.8	0.56	0.59	2.15
B5	51	10.2	5.9	0.86	0.53	2.14
B6	25	29.1	16.0	1.49	1.27	6.10
B6t	25	29.1	-	0.51	0.63	5.22

As expected, the values of all parameters are significantly higher in the case of the perforated, double height clay bricks, units B6, tested in both, longitudinal and transverse directions. On the basis of the results given in Table 4, it can be concluded that the units belonging to Group 1, according to Eurocode 6, are stronger and more robust than the units, belonging to Group 2. It can be also seen that, the values of the tensile and especially shear strength of the hollow clay blocks B1–B5, belonging to Group 2 units, do not vary significantly. No distinct correlation between the strength of the units and their shape can be observed. In other words, on the basis of these particular types of tests it is not possible to propose a value of a strength parameter, which would determine the limit between the robust and brittle behavior of masonry units.

Since the quality of masonry blocks is declared by their compressive strength, the correlation between the mean compressive strength of units, normal to the bed joints, $f_{b,m}$, on the one hand, and compressive strength, parallel to the bed joints ($f_{b,h}/f_{b,m}$), as well as the tensile ($f_{bt,d}/f_{b,m}$; $f_{bt,s}/f_{b,m}$) and shear strength ($f_{bs}/f_{b,m}$), on the other, has been analyzed. The results are shown in Figure 7, where the ratio between the various strength parameters and compressive strength of the tested hollow blocks is plotted against the volume of holes. As can be seen, only the ratio between the compressive strength, parallel to the bed joints, and the compressive strength, normal to them, indicates a trend of increase with the decreased volume of holes. In the case of the units B3 the compressive strength, parallel to the bed joints, which is not located on the trend line, is below the value, allowed by Eurocode 8 ($f_{b,hmin}$ = 2.0 MPa). At the same time, however, the ratio obtained in the case of the perforated bricks B6, belonging to Group 1 units, where the volume of holes was only 25 %, was the same as in the case of the hollow unit B5 with 51 % of holes.

The trend in the case of the shear strength/compressive strength ratio is not so clear, whereas the diagonal tensile- and splitting strength/compressive strength ratio only slightly depends on the volume of holes.

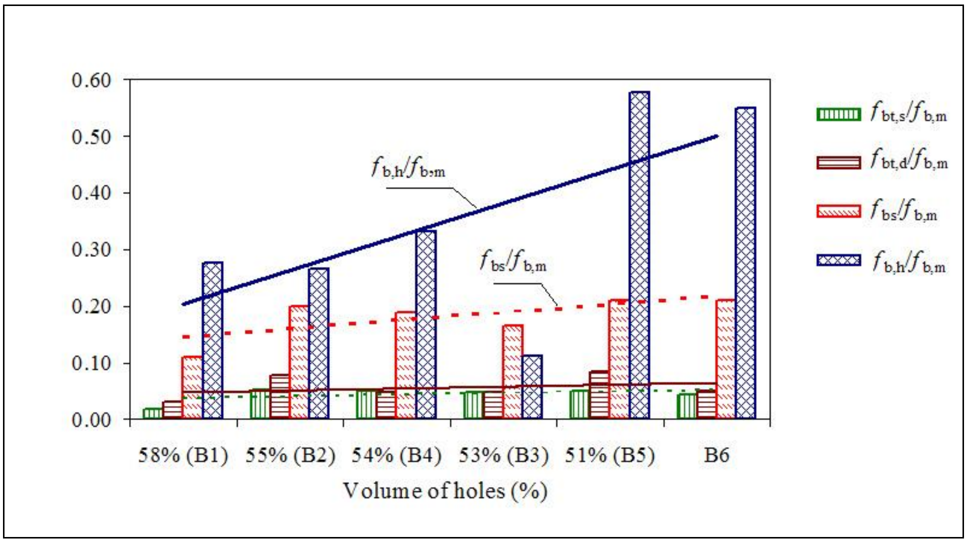

Figure 7. Correlation between different strength parameters and the volume of holes

4. TESTING OF MASONRY WALLS

To complement the information obtained by testing the masonry units as single elements, the behavior of the same units, however as constituent parts of the masonry structural walls, has been investigated by subjecting a series of masonry walls of the same size to simulated seismic loads (see Table 1). All walls have been built on reinforced concrete foundation blocks, with a bond-beam on the top of the walls. General purpose pre-mixed mortar Kemamix G (producer Kema Puconci, Slovenia), mortar class M5, has been used for the construction of specimens. In this study, vertical joints between the units have been fully filled with mortar.

4.1 Compression tests

To adjust the level of vertical precompression, acting on the walls under simulated seismic loading conditions, the compressive strength of the masonry has been determined following the testing procedure, specified in the standard [10]. However, only two specimens of each type have been tested in a 5000 kN testing machine. Testing arrangement can be seen in Figure 8, whereas the average dimensions of specimens for the compression tests and the test results are summarized in Table 5. The mean values of the compressive strength of the units, $f_{b,m}$, and actual compressive strength of mortar, f_m, determined on prisms 4/4/16 cm on the day of the testing of the walls, are also given in the table.

Table 5. Dimensions and material properties of the walls, tested in compression, and test results (compressive strength of masonry, f_c, average values)

Unit	Number of walls	l (cm)	h (cm)	t (cm)	$f_{b,m}$ (MPa)	f_m (MPa)	f_c (MPa)	$1.2f$ (MPa)
B1	2	100.7	142.4	28.4	18.2	7.69	5.7	8.3
B2	2	101.3	150.4	28.0	11.4	4.82	5.8	5.2
B3	2	101.0	143.2	28.9	12.8	5.79	5.4	6.0
B4	2	99.9	142.5	28.6	11.4	4.92	5.7	5.0
B5	2	101.9	150.1	30.0	10.2	5.0	4.1	4.8
B6	1	107.3	146.5	24.9	29.1	5.70	6.5	12.1
B6t	1	105.7	146.8	12.3		2.13	6.6	9.0

The meaning of the symbols in Table 5 is as follows:

l, h, t = the length, height and thickness of the walls, respectively,
f_c = mean compressive strength of masonry, obtained by testing,
f = characteristic compressive strength of masonry, calculated according to Eurocode 6.

For comparison, the compressive strength of masonry has been evaluated also by empirical equation, proposed for the determination of the characteristic strength of masonry, f, in Eurocode 6 [3], which is a function of the normalized compressive strength of masonry blocks, f_b, and compressive strength of masonry, f_m:

$$f = K\, f_b^{\alpha}\, f_m^{\beta}. \tag{4}$$

K, α, and β are constants. In the case of the units belonging to Group 2 and general purpose mortar, the respective values are K = 0.45, α = 0.7, and β = 0.3. In the case of the units B6, which belong to Group 1, however, K = 0.55.

To be compared with the experimental mean values, the calculated characteristic values have been multiplied by 1.2, following the definition of the characteristic strength, which is the mean value, obtained by testing, divided by 1.2. It can be seen that good agreement between the actual and EC 6 proposed values has been obtained except in the case of the walls made with hollow blocks type B1. In the case of the walls made with Group 1 units B6, the code values highly overestimated the actual compressive strength.

Figure 8. Disposition and instrumentation of walls during compression tests

Analyzing the observed behavior of the walls during the compression tests (see Figure 9), no specific conclusion can be made as regards the robustness of the hollow units tested within this study. Whereas shells of the hollow blocks buckled and separated from the units in the case of the walls made with hollow block B1–B5 (Fig. 9a), typical vertical cracks along the whole height of the walls have been observed before collapse in the case of the walls made with perforated bricks B6 (Fig. 9b). In the case of the hollow blocks, crushing of webs in the inside of the walls has been also identified after the tests.

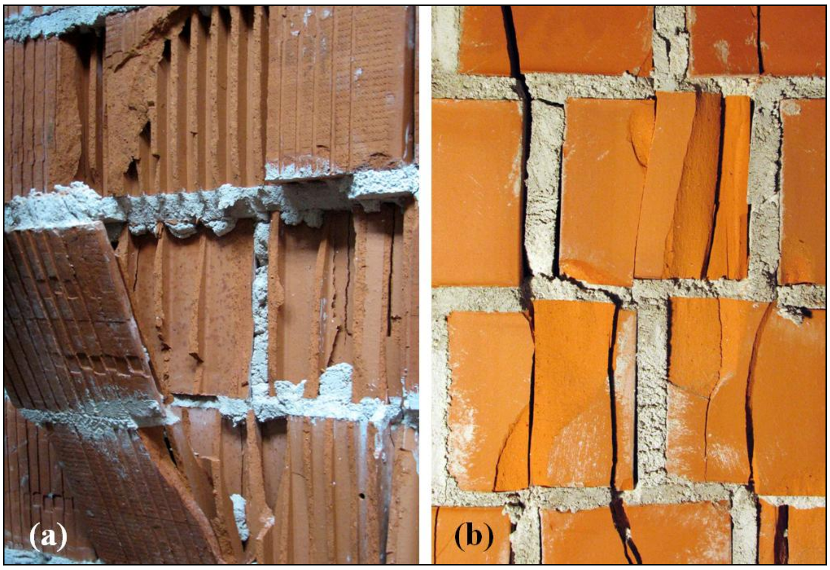

Figure 9. Details of failure mechanisms at compression. a) Buckling of shells in the case of the hollow block B4. b) vertical cracking of units B6

The difference between the ductile and brittle behavior of perforated bricks (Group 1 unit B6) and hollow blocks (Group 2 units B1–B5), respectively, can be clearly seen in Figure 10, where the stress-strain diagrams, obtained during the compression tests of the walls, are compared. However, no specific conclusion can be made regarding the parameters which define the robustness of the units under consideration.

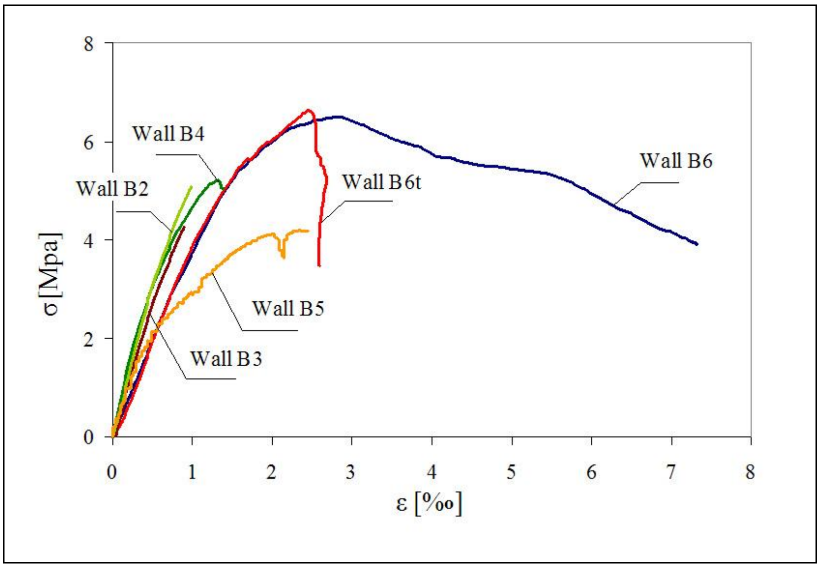

Figure 10. Typical stress-strain relationships, obtained by compression tests of walls

The difference in the observed behavior of walls B6 and B6t, made with the same units, is the result of orientation of the units and consequent different slenderness ratios of the walls. Namely, the thickness of wall B6, where the units have been laid transversally, was 24.9 cm, whereas the thickness of wall B6t with units laid in longitudinal direction, was 12.3 cm.

4.2 Cyclic shear tests of walls

To study the behavior of different types of units as constituent elements of structural walls subjected to seismic loading, the walls have been tested as vertical cantilevers, subjected to constant precompression, simulating the gravity loads, and programmed, cyclically acting lateral displacements, simulating the horizontal seismic loading. Both types of loads have been imposed at the level of the bond-beam on the top of the walls. The reinforced concrete foundation blocks on which the walls have been constructed, have been fixed to the strong floor by means of bolts, prestressed in order to prevent lateral motion and rocking. The test set-up consisted of a steel testing frame and hydraulic actuators, fixed to the frame in order to simulate constant gravity loads and lateral in-plane seismic loads. All walls have been instrumented with displacement transducers (LVDT-s) to measure horizontal displacements and rotation, as well as deformation of the masonry in the diagonal direction of the walls' surface. Load cells have been placed between the wall and actuators in order to measure the actual forces acting on the wall. The testing arrangement and instrumentation of the walls can be seen in Figure 11.

Figure 11. Test set-up and instrumentation of walls for cyclic lateral resistance tests

The dimensions of the walls and testing parameters, such as vertical load acting on the walls, V, compressive stresses in the walls, σ_o, and the resulting level of precompression, σ_o/f_c, are given in Table 6.

Table 6. Dimensions of the walls and testing parameters for cyclic shear tests (average values)

Unit	Wall	Number of walls	l (mm)	h (mm)	t (cm)	f_c (MPa)	V (kN)	σ_o (MPa)	σ_o/f_c
B1	B1/1	2	100.4	142.6	28.5	5.7	551.	1.92	0.34
	B1/2	2	99.9	143.1	28.5		274.8	0.96	0.17
B2	B2/1	2	101.9	150.4	28.1	5.8	490.2	1.71	0.29
	B2/1a	1	100.7	151.0	28.1		388.2	1.37	0.24
	B2/2	1	101.4	150.5	28.1		268.0	0.94	0.16
B3	B3/1	2	100.8	141.2	29.0	5.4	488.5	1.67	0.31
	B3/2	2	100.6	142.3	29.2		259.2	0.88	0.16
B4	B4/1	2	99.5	142.3	28.9	5.7	464.7	1.62	0.28
	B4/2	2	99.6	141.3	28.7		261.7	1.00	0.18
B5	B5/1	3	102.7	151.2	30.0	4.1	367.7	1.19	0.29
	B5/2	3	102.9	151.1	30.0		183.9	0.60	0.15
B6	B6/1	2	106.3	146.9	25.2	6.5	524.3	1.95	0.30
	B6/2	2	107.2	146.1	25.2		273.9	1.01	0.16
B6t	B6t/1	1	105.2	147.0	12.0	6.6	259.5	2.05	0.31
	B6t/2	1	105.3	146.0	12.2		136.5	1.06	0.16

The walls have been subjected to constant vertical load and imposed horizontal displacements, caused by a programmable hydraulic actuator, fixed on one of the columns of the testing frame at the mid-height level of the bond-beam, and connected to the wall at the middle of the length of the bond-beam with a steel connector, pinned at both ends (Figure 11). Cyclic lateral displacements with step-wise increased amplitudes, repeated three times at each displacement peak, have been used to simulate the in-plane lateral seismic loads. The shape of displacement pattern and amplitudes, used to control the actuator's motion, was the same in all cases: ± 0.25 mm, ± 0. 5 mm, ± 0.75 mm, ± 1.0 mm, ± 1.5 mm, ± 2.0 mm, ± 2.5 mm, ± 3.0 mm, ± 4.0 mm, ± 5.0 mm, ± 6.0 mm, ± 8.0 mm, ± 10.0 mm, etc. The uniform imposed displacement pattern made possible easier comparison of seismic behavior of different types of walls. Average values of quantities, measured on the walls of the same type at the same displacement amplitudes, at positive and negative direction of loading, have been analyzed.

All walls failed in shear, as expected. Because of the type of the masonry under consideration (unreinforced masonry), geometry of the specimens (height to length ratio h/l = 1.45), precompression ratio (σ_o/f_c = 0.15–0.32) and constraint conditions (vertical cantilever), a rocking type of behavior has been observed in the beginning of test. As a rule, horizontal cracks developed at the tensioned side of the walls in the connection plane between the wall and foundation block. In a few cases, however, horizontal cracks occurred in the bed joints at the top of the first course of units. Then, diagonally oriented cracks developed in the walls, which, depending on the level of precompression, passed either through the mortar joints (low precompression) or through the units (high precompression), or both. Cracks in the units in the compressed toes of the walls have been also observed. At the increased displacement amplitudes, local buckling and crushing of outer shells of the units took place along the diagonal shear cracks. In some cases, the propagation of shear cracks was accompanied by crushing of the units at the compressed toes of the walls. Ultimately, individual units started crushing in the middle part of the walls so that the walls collapsed or the resistance degraded substantially because of the local failure of the units. In the case of all types of walls, crushing of units was more severe when subjected to higher level of precompression.

As representative, damage occurred to the walls made of hollow blocks B3 and B4, tested at high and low level of precompression, is shown in Figures 12–15. In the same figures, the hysteretic relationships between the lateral load and rotation of the walls, i.e displacement at the top/height of the wall ratio, $\Phi = d/h$ (in %), are also shown. The difference in the displacement and load-bearing capacity of the walls, subjected to different levels of precompression, can be clearly seen.

For comparison, the damage at ultimate state, occurred to the walls, made of the perforated bricks B6, and hysteretic behavior, observed during the tests at high and low levels of precompression, is shown in Figures 16 and 17, respectively. Against expectations, based on the results of tests of single units, relatively brittle behavior of the walls has been observed also in the case of the walls type B6, at both levels of precompression. Moreover, the displacement capacity of the walls type B6 did not exceed the displacement capacity of the walls made of the obviously more brittle hollow blocks.

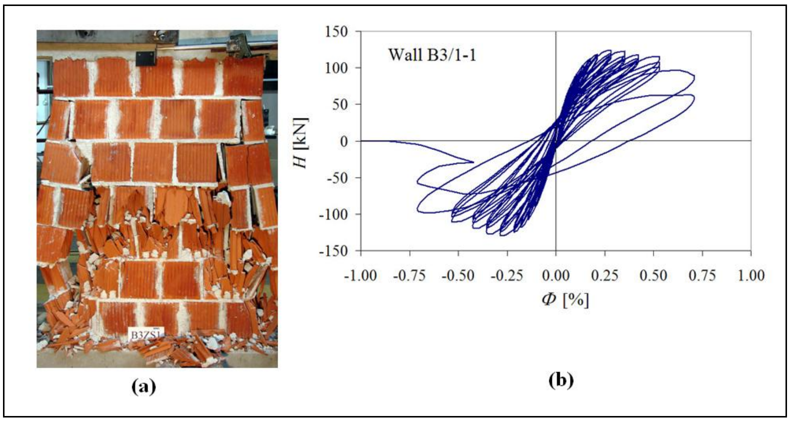

Figure 12. Typical damage at ultimate state a) and b) hysteresis loops, obtained by testing the wall type B3 at 0.28 f_c

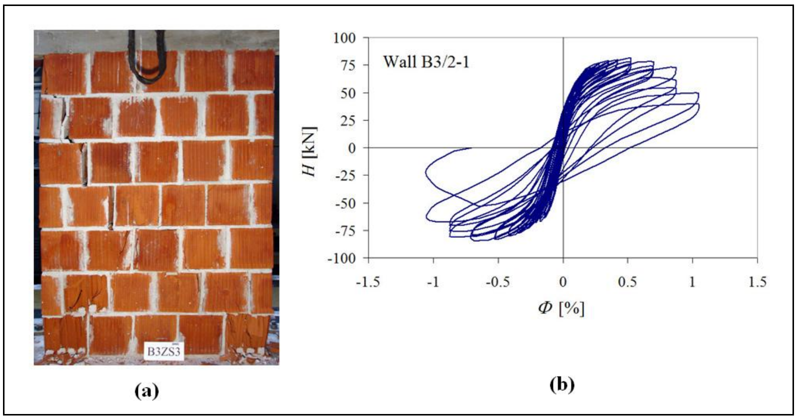

Figure 13. Typical damage at ultimate state a) and b) hysteresis loops, obtained by testing the wall type B3 at 0.15 f_c

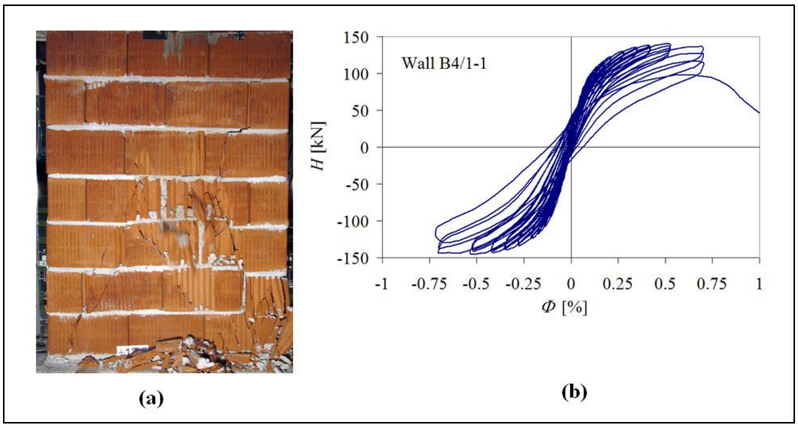

Figure 14. Typical damage at ultimate state a) and b) hysteresis loops, obtained by testing the wall type B4 at 0.31 f_c

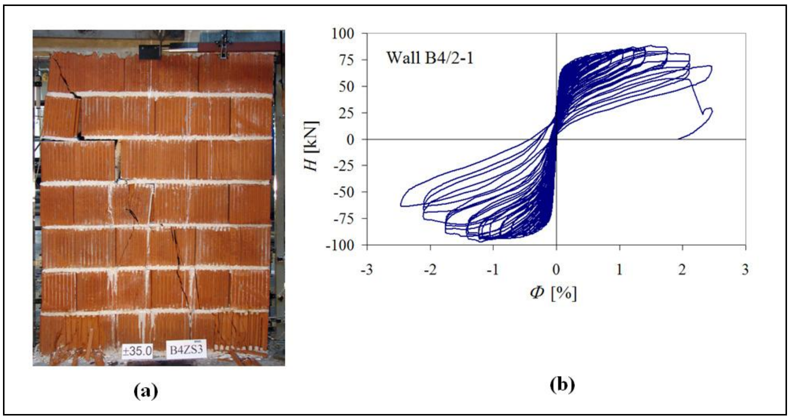

Figure 15. Typical damage at ultimate state a) and b) hysteresis loops, obtained by testing the wall type B4 at 0.18 f_c

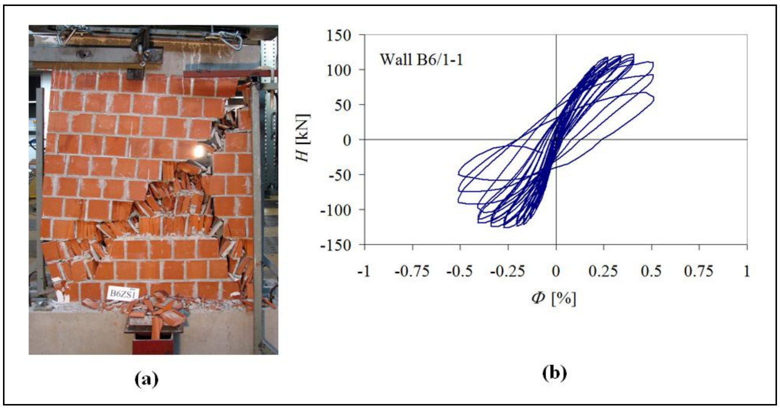

Figure 16. Typical damage at ultimate state a) and b) hysteresis loops, obtained by testing the wall type B6 at 0.31 f_c

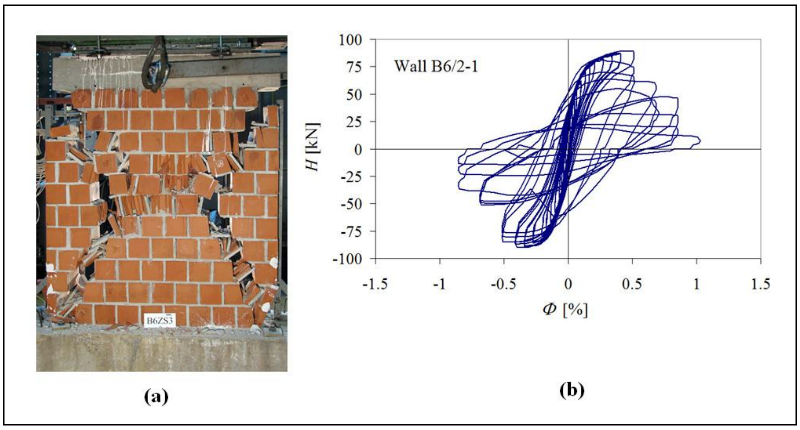

Figure 17. Typical damage at ultimate state a) and b) hysteresis loops, obtained by testing the wall type B6 at 0.16 f_c

By comparing the damage patterns and displacement capacity of the walls, subjected to different levels of precompression, it can be seen that the local brittle failure of units, which took place at high compressive stresses in the walls, adversely affected the displacement and energy dissipation capacity of the walls when subjected to cyclic lateral loading. The effect of precompression on the displacement capacity of the walls can be clearly seen in Figure 18, where for each type of the tested walls the average envelopes of hysteretic relationships between the horizontal force and displacements, given in percent of the wall's height, are presented for the high and the low level of precompression, respectively. It has to be borne in mind, however, that because of the testing procedure and masonry type (vertical cantilever wall with relatively high height/length ratio, quite rigid unreinforced masonry), rocking of the walls on the foundation blocks took place in the initial phases of testing, the amount of which depended on the level of precompression. Consequently, typical S-shaped hysteresis loops have been recorded in the beginning of tests (Figures 13–17). In this regard, the behavior of walls type B6, made of Group 1 units, did not differ from the behavior of the walls, made of Group 2 units B1–B5. Unfortunately, the attempts to take account for rocking and correct the values of displacements measured at the top of the walls, did not yield realistic values. However, after the formation of diagonal cracks the rocking of the walls was no more observed.

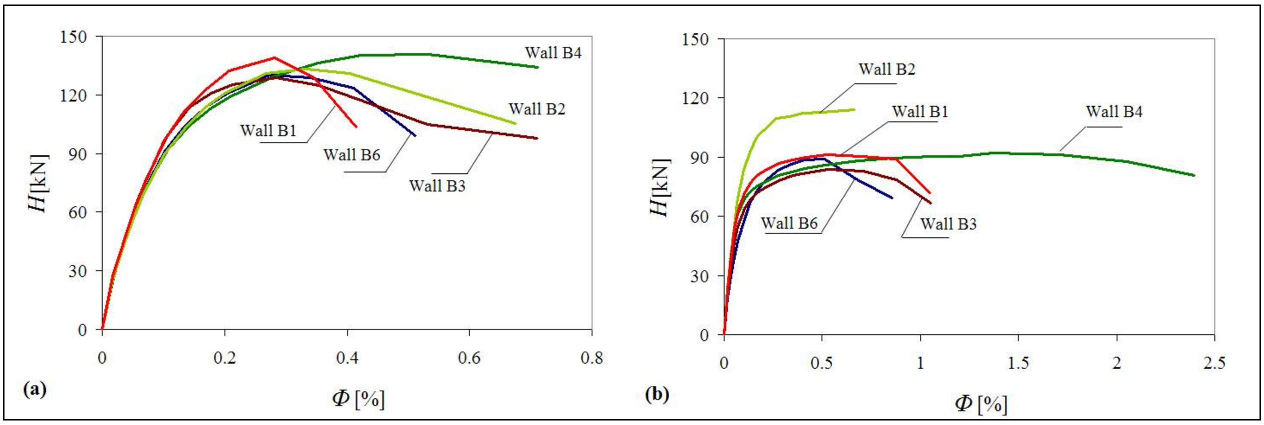

Figure 18. Lateral load - rotation angle relatioships, obtained by testing the walls at 0.3 f_c (a) and b) at 0.15 f_c

As can be seen, no general conclusion regarding the influence of different shapes and mechanical properties of the units on the lateral resistance of the walls can be drawn. As the results indicate, the level of precompression was the governing parameter in all cases. By increasing the precompression, the resistance of the walls increased, but the displacement capacity decreased. The walls subjected to lower level of precompression exhibited significantly greater displacement and ductility capacity than the walls tested at higher level.

Taking advantage of similarity of resistance envelopes, obtained at each level of compression level, shown in Figure 18, the average envelopes of all walls, tested at the same level of precompression, are plotted in Figure 19. To obtain the average envelope for the low precompression level, only walls type B1, B3, and B 6 have been considered.

Based on the observed damage propagation and lateral load-displacement relationships, three limit states have been defined on the resistance curves, which characterize the seismic behavior of the tested walls. Namely:

- Crack (damage) limit state, defined by the displacement, d_{cr}, or rotation, $\Phi_{cr} = d_{cr}/h$ (in %), of the walls, where the first cracks occur in the walls, causing evident changes in stiffness of the walls;
- Maximum resistance, H_{max}, i.e. maximum lateral load, attained during the test;
- Ultimate limit, defined by the maximum attained displacement (rotation) of the wall just before collapse d_u.(Φ_u).

The limit states are indicated in Figure 19.

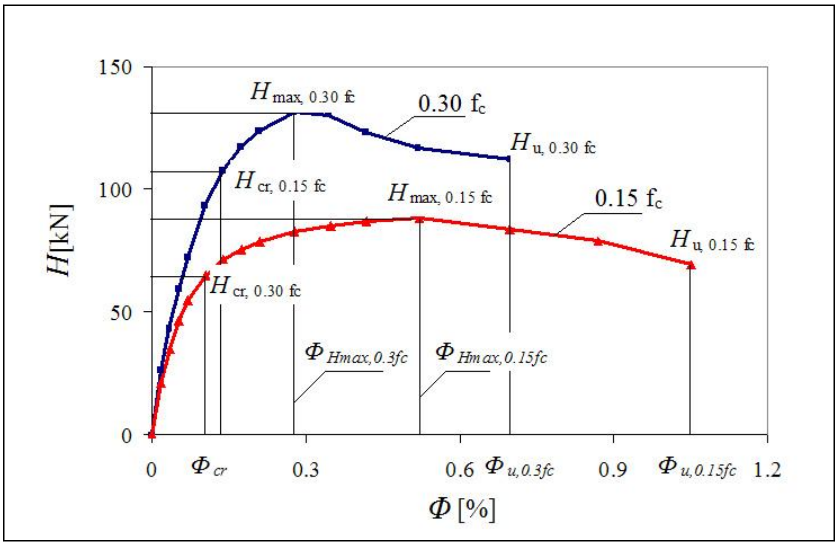

Figure 19. Comparison of average lateral load - rotation angle envelope curves, obtained by testing the walls at high and low precompression

To analyze the load bearing and displacement capacity, the average values of the measured displacements (rotations) at each characteristic limit state have been compared for both high and low level of precompression. The values are given in Table 7.

Table 7. Correlation between the displacements at characteristic limit states for the case of the low and high precompression of the tested walls

σ_o/f_c	Φ_{cr} (%)	Φ_{Hmax} (%)	$\frac{\Phi_{Hmax}}{\Phi_{cr}}$	Φ_u (%)	$\frac{\Phi_u}{\Phi_{cr}}$
0.30	0.14	0.28	2.01	0.70	5.07
0.15	0.11	0.52	4.97	1.05	10.01

No collapse and damage limitation requirements should be considered in the design by verifying the ultimate and usability limit states of the structure under consideration. When the structure is verified for damage limitation requirement, the interstory drift is limited to an allowable level [13, 14 and 15]. Previous research has indicated the following ranges of interstory drifts (relative story displacements) of masonry structures at characteristic limit states [after 15]:

- At crack limit: Φ_{cr} = 0.2–0.4 %;
- At maximum resistance: Φ_{Hmax} = 0.3–0.6 %;
- At design ultimate state: Φ_{u} = 1.0–1.2 %;
- At limit state of collapse: Φ_{u} = 1.2–5.4 %.

As can be seen, the values, given in Table 7, are in good agreement with the actually observed values at crack limit and maximum resistance. However, at ultimate state, only the displacement capacity of the walls, tested at lower level of precompression, can be considered acceptable.

Table 8. Correlation between the actual and expected increase in lateral resistance of the tested walls due to precompression

Wall	σ_o/f_c	H_{max} (kN)	$K_{\sigma o} = \sqrt{\sigma_{o1} / \sigma_{o2}}$ Actual	$K_{\sigma o} = \sqrt{\sigma_{o1} / \sigma_{o2}}$ Expected
B1/1	0.34	140.6	1.53	1.41
B1/2	0.17	92.0		
B2/1	0.29	133.7	1.13	1.10
B2/1a	0.24	118.0		
B2/2	0.16	90.9	1.47	1.35
B3/1	0.31	128.7	1.53	1.39
B3/2	0.16	84.2		
B4/1	0.28	141.7	1.51	1.25
B4/2	0.18	93.9		
B5/1	0.29	106.7	1.81	1.39
B5/2	0.15	59.0		
B6/1	0.30	131.0	1.43	1.37
B6/2	0.16	91.6		
B6t/1	0.31	73.1	1.54	1.39
B6t/2	0.16	46.9		

To verify the possible influence of local brittle behavior of units on the lateral resistance, the expected effect of the increase of compressive stresses on the shear resistance of the walls has been estimated by calculation. For this purpose, the equation for the calculation of the shear resistance of an unreinforced masonry wall, R_w, proposed by Turnšek and Čačovič [11], has been used:

$$R_w = A_w \frac{f_t}{b} \sqrt{\frac{\sigma_o}{f_t} + 1}, \tag{5}$$

where f_t = the tensile strength of masonry, conventionally defined by the principal tensile stress developed at the attained maximum resistance of a masonry wall, assuming that the wall is elastic, homogeneous and isotropic panel, A_w = the horizontal cross-sectional area of the wall, and b = shear stress distribution coefficient. The validity of this equation has been recently discussed in [12]. Following this equation, the expected effect of the increase of vertical precompression on the shear resistance of the walls can be easily evaluated as a ratio: $K_{\sigma o} = \sqrt{\sigma_{oH} / \sigma_{oL}}$, where σ_{oH} and σ_{oL} are the average compressive stresses in the horizontal cross-sectional area of the walls at high and low level of precompression, respectively. The results of such analysis are given in Table 8.

As can be seen, the values of coefficient are within the expected range of scattering of resistance values. No trend can be observed as regard the adverse influence of the lack of robustness of the units on the lateral resistance of the walls.

Therefore, besides ductility capacity, the correlation between the input and dissipated hysteretic energy would provide more reliable information regarding the influence of brittle local failure of units on the seismic behavior of walls. To analyze the energy dissipation capacity, the cumulative input energy, E_{inp}, needed to deform the wall from the beginning of test to the displacement amplitude under consideration, and dissipated hysteretic energy, have been correlated. Cumulative input energy has been defined as the cumulative work of the hydraulic actuator, needed to deform the wall up to the amplitude peak under consideration, following the programmed displacement path. In one cycle of loading, i.e. for pushing the wall to maximum displacement amplitude of the cycle, and pushing it from the unloaded position to the same displacement amplitude in the opposite direction, the work of the actuator has been evaluated by equation:

$$\Delta E_{inp,i} = \int_{(H=0)_i}^{(H=0)_{i+1}} dE_{inp} \,, \tag{6}$$

where:

$$dE_{inp} = \begin{cases} 0, & \text{if } H\mathrm{d}u \le 0 \text{ and} \\ H\mathrm{d}u, & \text{if } H\mathrm{d}u > 0 \end{cases},$$

where u = displacement of the actuator. Cumulative input energy, however, from the beginning of test to the displacement amplitude under consideration, has been evaluated as a sum:

$$E_{\mathrm{inp}} = \sum_{i=1}^{k} \int_{(H=0)_i}^{(H=0)_{i+1}} dE_{\mathrm{inp}} \,. \tag{7}$$

Dissipated energy has been evaluated on the basis of the measured lateral load-displacement hysteretic relationships. The amount of dissipated energy in one cycle of loading has been defined as the area of the hysteresis loop between two consecutive displacement peaks, the cumulative dissipated energy, however, as a sum of dissipated energy in all loading cycles from the beginning of test until the end of cycling at the displacement amplitude under consideration. The calculated values of the input and dissipated hysteretic energy at characteristic limit states are given in Table 9.

Table 9. Cumulative input and dissipated hysteretic energy at the characteristic limit states

		Crack limit		Maximum resistance		Ultimate state	
Wall	σ_o/f	E_{inp} (kNm)	E_{hys} (kNm)	E_{inp} (kNm)	E_{hys} (kNm)	E_{inp} (kNm)	E_{hys} (kNm)
B1/1	0.34	2.12	0.57	4.20	1.07	11.57	4.28
B1/2	0.17	7.94	1.84	10.29	2.36	22.88	6.64
B2/1	0.29	3.16	0.88	5.91	1.61	17.00	5.59
B2/1a	0.24	1.44	0.39	1.44	0.39	2.15	1.19
B2/2	0.16	0.68	0.18	0.68	0.18	1.77	0.51
B3/1	0.31	1.79	0.54	5.00	1.34	14.35	4.74
B3/2	0.16	9.52	2.01	9.52	2.01	23.70	6.79
B4/1	0.28	10.20	2.25	12.19	2.73	19.04	4.73
B4/2	0.18	31.27	7.19	50.26	11.76	72.03	18.78
B5/1	0.29	8.95	1.35	8.95	1.35	27.36	6.52
B5/2	0.15	10.58	1.56	10.78	1.62	43.29	9.73
B6/1	0.30	3.54	0.97	4.85	1.37	13.41	5.40
B6/2	0.16	2.39	0.71	12.42	3.63	20.07	7.86
B6t/1	0.31	2.56	0.77	2.56	0.77	5.56	2.20
B6t/2	0.16	1.83	0.46	5.10	1.37	15.67	6.27

The ratio between the cumulative input and dissipated hysteretic energy at maximum resistance and ultimate state is graphically presented for both levels of precompression in Figures 20a and b. The ratio between the cumulative input energy, needed to attain the maximum resistance and ultimate state, at 0.15 f_c and energy at 0.30 f_c, however, is presented in Figures 21a and b. In the same figure, the ratio between the energy, dissipated at 0.15 f_c and energy, dissipated at 0.30 f_c, at maximum resistance and ultimate state, is also plotted. It can be seen that in the case of all wall types, more input energy was needed to attain the maximum resistance and ultimate limit states in the case of the low than in the case of high precompression, although one could have expected that more energy will be needed to deform the walls subjected to higher level of precompression. It can be also seen, that at both limit states, more energy has been dissipated in the case where the walls have been subjected to low than in the case of the high precompression. This is the expected consequence of brittle behavior of units at higher level of precompression.

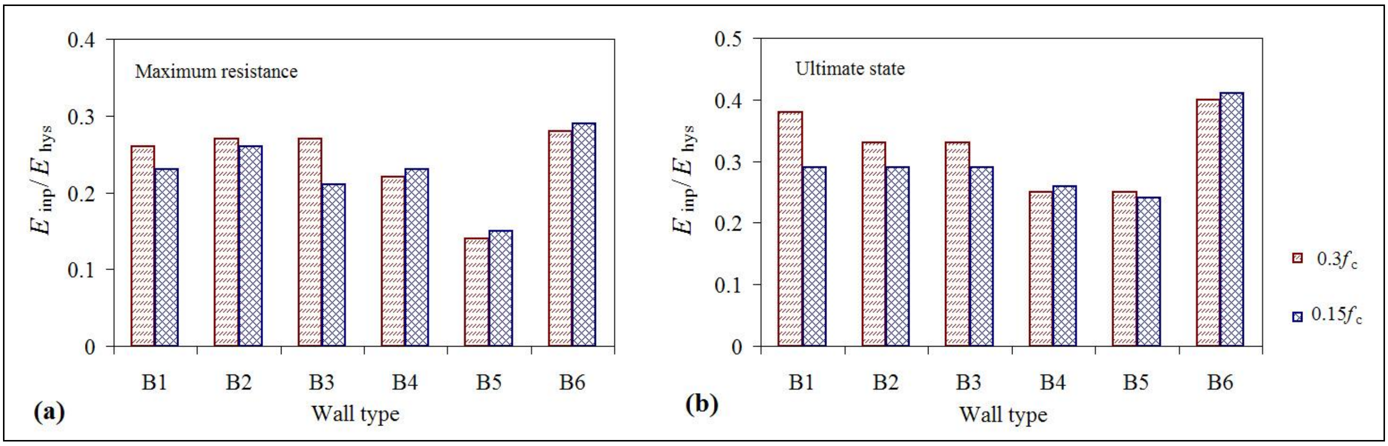

Figure 20. Input/dissipated energy ratio at maximum resistance (a) and b) at ultimate state

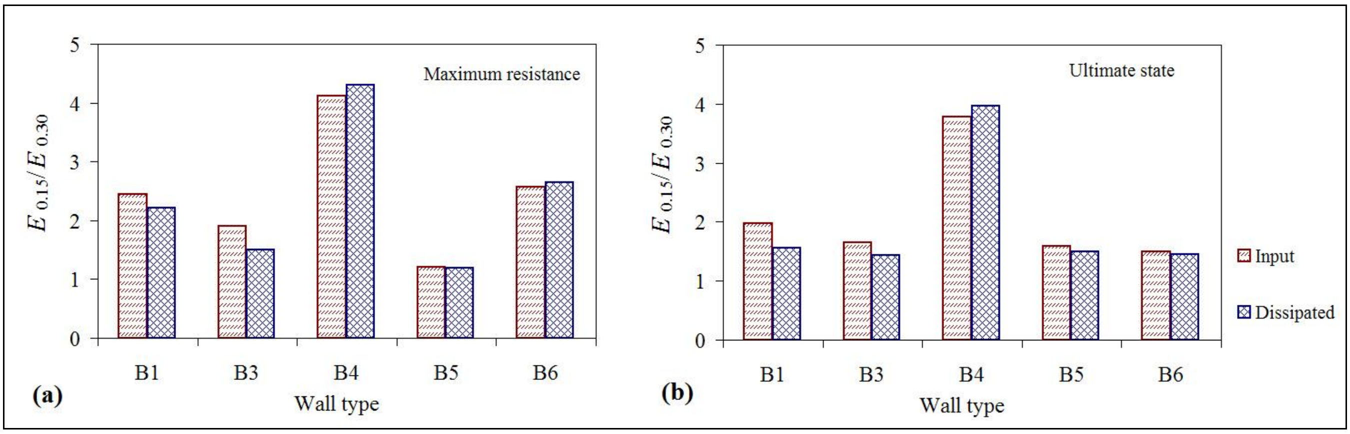

Figure 21. Ratio between the input energies at 0.15 f_c and 0.30 f_c, and ratio between the dissipated energies at 0.15 f_c and 0.30 f_c, at maximum resistance (a) and b) ultimate state

Interestingly, the ratio between the input and dissipated hysteretic energy did not depend significantly on the level of precompression. As can be seen in Figure 20, the ratio was slightly higher at high than at low precompression in the case of the walls type B1, B2, and B3, whereas it was practically the same at both levels of precompression in all other cases. On the average, 22 % (24 %) of the input energy has been dissipated by the hysteretic behavior at maximum resistance, and 30 % (32 %) at ultimate limit state in the case of the walls tested at lower (higher) level of precompression. The cumulative input/dissipated hysteretic energy ratio was highest in the case of the walls made of Group 1 units B6.

Almost 2-times more input energy than at high (4-times in the case of the walls type B4) has been needed to attain the maximum resistance and ultimate limit states at low precompression level (Figure 21). Similarly, up to 4-times more energy has been dissipated at low than at high level of precompression. The relationships obtained in the case of the walls type B6, made of Group 1 units, did not differ significantly from the relationships, observed in the case of the walls, made of Group 2 units B1–B5.

This indicates that the level of compressive stresses in structural walls should be considered as the critical parameter to determine the terms for sufficient robustness. As indicated by this study, the shape and mechanical properties of the units are important, but are implicitly taken into consideration in this, general conclusion. The values of strength parameters, used in the design of masonry structures, are based on the gross-sectional areas of the unit and walls, and not on the actual net, load-bearing areas of the material. In Table 10, the net-values obtained under the assumption that the reduction of the gross load-bearing area of the units is proportional with the volume of holes, are correlated.

Table 10. Correlation between the volume of holes and gross and net* values of compressive strength of units and masonry

Wall	Volume of holes (%)	$f_{b,m}$ (MPa	$f_{b,m,net}$ (MPa)	f_c (MPa)	$f_{c,net}$ (MPa)	$f_c/f_{b,m}$
B1/1	58	18.2	43.3	5.7	13.6	0.31
B1/2	58	18.2	43.3	5.7	13.6	0.31
B2/1	55	11.4	25.3	5.8	12.9	0.51
B2/1a	55	11.4	25.3	5.8	12.9	0.51
B2/2	55	11.4	25.3	5.8	12.9	0.51
B3/1	53	12.8	27.2	5.4	11.5	0.42
B3/2	53	12.8	27.2	5.4	11.5	0.42
B4/1	54	11.4	24.8	5.7	12.4	0.50
B4/2	54	11.4	24.8	5.7	12.4	0.50
B5/1	51	10.2	20.8	4.1	8.4	0.40
B5/2	51	10.2	20.8	4.1	8.4	0.40
B6/1	25	29.1	38.8	6.5	8.7	0.22
B6/2	25	29.1	38.8	6.5	8.7	0.22
B6t/1	25	29.1	38.8	6.6	8.8	0.23
B6t/2	25	29.1	38.8	6.6	8.8	0.23

* Calculated by assuming that the load-bearing area of the unit is proportional with the volume of holes

As can be seen, no direct correlation between the volume of holes and the strength of masonry materials can be observed except in the case of the walls made of hollow blocks B2 and B4 and perforated bricks B6.

5. CONCLUSIONS

Although substantial amount of different types of tests of single hollow clay blocks as well as masonry walls have been carried out, no straightforward conclusion regarding the influence of shape and mechanical properties of masonry units on the robustness of units can be made, and, consequently, determine and quantify the criteria which determine "sufficient robustness".

The tests on single units indicated that only the ratio between the compressive strength, parallel to the bed joints, and the compressive strength, normal to them, indicates a trend of increase with the decreased volume of holes. However, the same ratio as in the case of the hollow unit, belonging to Group 2 units, with 51 % of holes, has been obtained also in the case of the perforated bricks, belonging to Group 1 units, where the volume of holes was only 25 %. Other parameters, which determine the mechanical properties of units, such as the shear strength/compressive strength, diagonal tensile- and splitting strength/compressive strength ratios only slightly depended on the volume of holes. Practically the same relationships between different strength parameters have been obtained in the case of both, hollow blocks and perforated bricks.

Whereas no firm conclusion regarding the robustness can be made on the basis of tests of single units, compression tests of masonry walls pointed out the brittle failure of the walls made of hollow blocks against rather ductile compression failure of walls made of perforated bricks. However, again no significant differences in the behavior of the walls made of different types of hollow blocks has been observed.

Against expectations, the shape and mechanical properties of individual units did not affect the seismic behavior of walls. When subjected to a combination of vertical and cyclic horizontal loads, however, the level of precompression, i.e. the working compressive stress/compressive strength of masonry ratio, proved to be the governing parameter. Namely, the behavior of the same units, which exhibited monolithic behavior at low level of precompression, became brittle when subjected to higher level of precompression. Although not obtained within the framework of the present project, a good example of this observation is shown in Figure 22. Whereas the hollow units, similar to type B5 units, studied within this study, rocked as solid bodies when subjected to low compressive stresses, and diagonally oriented cracks passed mainly through the mortar joints, they crushed when subjected to higher compressive stresses. As can be seen, the slope of the shear cracks followed the block laying pattern in both cases.

Figure 22. Typical crack patterns obtained by testing long walls made of units type B5 at 0.05 f_c (a) and b) at 0.15 f_c (adapted from [12])

The analysis of correlation between the input and dissipated hysteretic energy also confirmed the predominant effect of precompression. Displacement and energy dissipation capacity of the walls only slightly depended on the shape and mechanical properties of the units. The influence of the level of precompression was predominant in all cases.

On the basis of the results of the particular study, presented in this paper, it can be concluded that the requirements and recommendations for sufficient robustness of hollow clay blocks for the intended use in seismic regions are only partly function of the units' type. The working stress level in structural walls is the governing parameter. In this regard, the following conclusion can be made: if the units, belonging to Group 2 units according to Eurocode 6 are used for the construction of masonry structures in seismic zones, in no case the design compressive stresses in the walls should exceed 15 % of the characteristic compressive strength of masonry ($\sigma_d/f < 0.15$). In the case of Group 1 units, however, the allowable design working stress/compressive strength of masonry ratio may be increased to 0.20 ($\sigma_d/f < 0.20$). As the results of tests indicated, no additional requirements regarding the shape and mechanical properties of masonry units, besides those specified in the codes [2 and 3], are needed.

7. ACKNOWLEDGEMENT

The research, presented in this paper, was carried out within the framework of the research project L2-6417 and research program P2-0274, financed by the Slovenian Research Agency in the years 2003-2008. The research was co-financed by the brick manufacturer, Goriške opekarne, Ltd., from Renče, Slovenia. The contribution of the authors' former colleague Dr Vlatko Bosiljkov, and Dr Francesca da Porto from University of Padua, who visited the Institute on a NATO scholarship grant, to the success of the experimental phase of the project, is gratefully acknowledged.

8. REFERENCES

[1] Tomaževič M, Lutman M, Bosiljkov V (2006) Robustness of hollow clay masonry units and seismic behaviour of masonry walls. Construction and Building Materials 20: 1028–1039

[2] CEN (2004) Eurocode 8: Design of structures for earthquake resistance, Part 1: General rules, seismic actions and rules for buildings. EN 1998-1:2004, Brussels

[3] CEN (2005) Eurocode 6: Design of masonry structures - Part 1-1: Common rules for reinforced and unreinforced masonry structures. EN 1996-1-1:2005. Brussels

[4] Tomaževič M, Weiss P (2008) The influence of robustness of hollow clay blocks on seismic behaviour of masonry walls. Test results. Report ZAG/0964/04-1, Slovenian National Building and Civil Engineering Institute, Ljubljana

[5] CEN (2000) Methods of tests for masonry units - Part 1: Determination of compressive strength. EN 772-1:2000. Brussels

[6] CEN (1998) Methods of tests for masonry units - Part 3: Determination of net volume and percentage of voids of clay masonry units by hydrostatic weighing. EN 772-3:1998. Brussels

[7] CEN (2002) Methods of tests for masonry units - Part 13: Determination of net and gross dry density of masonry units (except for natural stone). EN 772-13:2002. Brussels

[8] CEN (2002) Methods of tests for masonry units : Part 16: Determination of dimensions. EN 772-16:2002. Brussels

[9] Bosiljkov V, Tomaževič M, Lutman M (2004) Optimization of shape of masonry units and technology of construction for earthquake resistant masonry buildings. Research report ZAG, Ljubljana

[10] CEN (1998) Methods of tests for masonry - Part 1: Determination of compressive strength. EN 1052-1:1998, Brussels

[11] Turnšek V, Čačovič F (1971) Some experimental results on the strength of brick masonry walls. Proceedings of the 2nd international brick-masonry conference. British Ceramic Society, Stoke-on-Trent: 149-156

[12] Tomaževič M, Gams M, Lu S (2009) Modelling of shear failure mechanism of masonry walls. 11th Canadian Masonry Symposium. CD-ROM. McMaster University, Canada Masonry Design Centre, Toronto: Paper A1-5.

[13] Zonta D, Zanardo G, Modena C (2001) Experimental evaluation of the ductility of a reduced-scale reinforced masonry building. Materials and Structures, 34: 636–644

[14] Alcocer SM, Arias JG, Flores LE (2004) Some developments on performance based seismic design of masonry structures. Performance-Based Seismic Design. Concepts and Implementation. Eds. Fajfar P, Krawinkler H. PEER Report 2004/05. Berkeley: 33–44

[15] Tomaževič M (2007) Damage as a measure for earthquake-resistant design of masonry structures: Slovenian experience. Canadian Journal of Civil Engineering. 34 (11): 1403–1412