11th Canadian Masonry Symposium, Toronto, Ontario, May 31- June 3, 2009

MODELLING OF SHEAR FAILURE MECHANISM OF MASONRY WALLS

M. Tomaževič[1], M. Gams[1] and S. Lu[2]
[1] Slovenian National Building and Civil Engineering Institute, miha.tomazevic@zag.si, matija.gams@zag.si
[2] Wienerberger A.G., Suikai.Lu@wienerberger.com

ABSTRACT
Shear failure mechanism, characterised by the formation of diagonal cracks, most frequently represents the critical failure mode of unreinforced and confined masonry walls when subjected to lateral in-plane seismic loads. Although other mechanisms are also possible, seismic resistance of a masonry structure depends predominantly on the shear resistance of the walls. The results of cyclic lateral resistance tests of nine unreinforced walls with height/length aspect ratio 0.7, constructed with hollow clay blocks laid in thin layer mortar with vertical joints of mechanical interlocking type, and subjected to three different levels of vertical precompression, have been used to compare the experimentally obtained resistance values with the results of calculations. It has been found that the shear friction failure (step-formed cracks, following the bed and head joints) determines the resistance at precompression less than 10 % of compressive strength and diagonal tension failure (diagonally oriented cracks passing the units) at higher precompression levels. It has been shown that typical equations, used for the calculation of the shear resistance of walls, do not have general validity, because they reflect the type of the shear failure, for which they had been developed. The resistance of the walls has been also assessed by a simple diagonal strut mechanism, where the resistance depends on the vertical load and inclination of struts, determined by dimensions of masonry units and masonry bond. Although surprisingly good correlation between the experimental and calculated values has been obtained at all precompression levels, further development of such model is needed before practical use.

KEYWORDS: unreinforced masonry walls, shear failure mechanism, shear friction failure, diagonal tension failure, modelling, compressive strut, lateral resistance.

INTRODUCTION
As specified in Eurocode 8: Design of earthquake resistant structures [1], European code for earthquake resistant design, besides traditional masonry construction systems, where both bed- and head-joints are fully filled with mortar, systems with partly filled or even dry, mechanical interlocking type of head-joints are also permitted for the construction of masonry structures in seismic zones. However, the conditions of use and possible limitations regarding the application of such systems should be specified in individual countries', members of the European Union, National Annexes to the main standard. Since the experimental information regarding the seismic behaviour of such systems is lacking, a series of walls have been recently tested at

Slovenian National Building and Civil Engineering Institute to study the mechanism of seismic behaviour as well as to determine the values of parameters, needed for the design of the particular type of masonry tested. Besides, the results of tests have been used to verify the validity of typical methods used for the assessment of the shear resistance of unreinforced masonry walls.

MASONRY WALLS, DESCRIPTION OF TESTS AND TEST RESULTS

A series of nine 250 cm long (l), 175 cm high (h) and 30 cm thick (t) unreinforced masonry walls with geometry aspect ratio h/l = 0.7 have been tested. Hollow clay blocks with dimensions 25/25/30 cm (length/height/thickness) have been used for the construction of walls. According to classification of masonry units by geometrical properties, specified in Eurocode 6: Design of masonry structures [2], the units can be classified into group 2 of masonry units (see Figure 1).

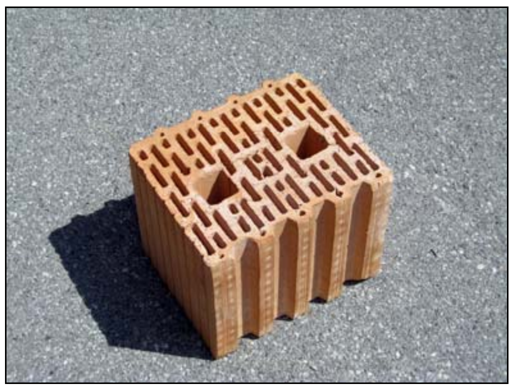


Figure 1: Hollow Clay Unit, Used for the Construction of Walls

Mean compressive strength of units, determined on 6 specimens, was 14.7 MPa. The upper and lower surfaces of units were grinded, so that factory mixed thin layer mortar with mean compressive strength 13.7 MPa in the bed-joints has been used for the construction of specimens. As mentioned, head joints have been of dry, groove and tongue type. Compressive and initial shear strength of masonry has been determined according to European standards, EN 1052-1 and EN 1502-3, respectively. Whereas the mean value of compressive strength of masonry, obtained on three specimens, was 5.98 MPa (characteristic value f_k = 4.98 MPa), the mean value of initial shear strength of masonry, obtained on six specimens, was 0.18 MPa (characteristic value f_{vko} = 0.10 MPa).

To be transported from the construction site to the testing floor, the walls have been built on reinforced concrete foundation blocks, provided with holes to accommodate the bolts to anchor the blocks into the testing floor during the tests. The bolts have been prestressed in order to prevent any lateral motion or rocking of foundation blocks. At the top of the walls, reinforced concrete bond beams have been constructed for application of constant vertical as well as the in-plane acting cyclic lateral load.

The walls have been instrumented with a set of load cells to measure the vertical and lateral loads and LVDTs to measure the displacements and control the tests. They have been tested as vertical cantilevers fixed to the testing floor and subjected to constant vertical load. 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. Test set-up consisted of a steel testing frame and three hydraulic actuators, fixed to the frame in order to simulate constant gravity loads (two 1000 kN capacity jacks) and lateral in-plane seismic loads (one two-way acting 500 kN capacity programmable actuator). The test set-up is presented in Figure 2.

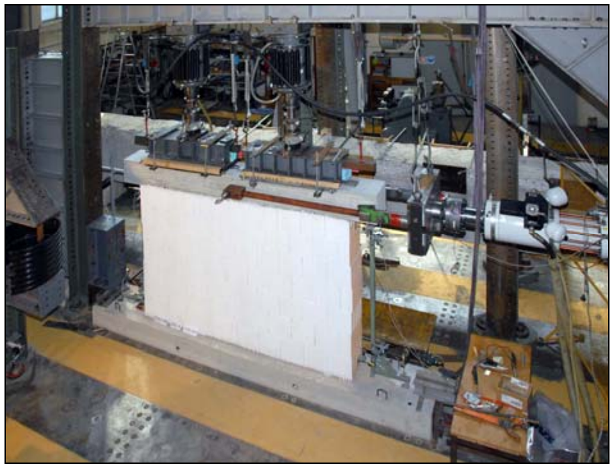
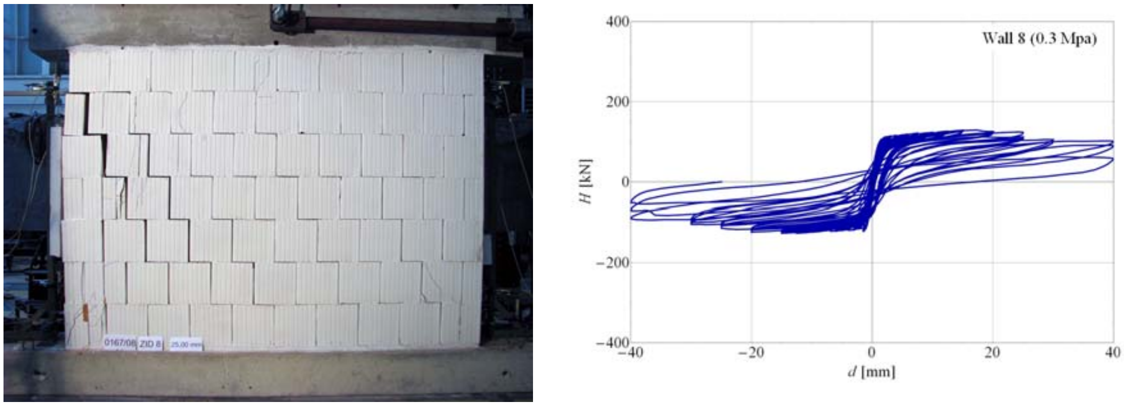


Figure 2: Test Set-up

The walls have been tested at three levels of vertical precompression, amounting to 0.92 MPa, 0.62 MPa, and 0.34 MPa. Three specimens have been tested at each level, which, taking into account the mean value of compressive strength of masonry, f = 5.98 MPa, amounted to 15 %, 10 %, and 5 % of compressive strength of masonry, respectively.

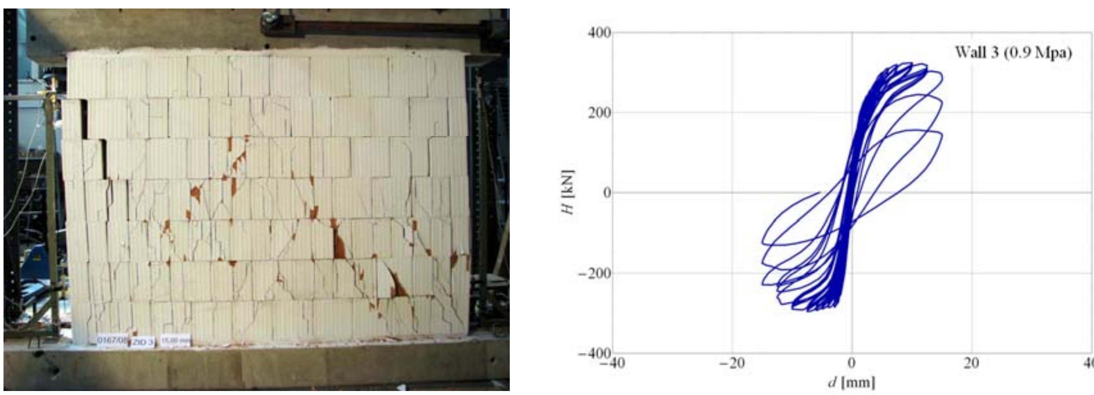
Although some rocking of the walls on the foundation block has been observed in the initial phases of tests, the predominant mechanism of behaviour was of typical shear type, as expected. Diagonally oriented cracks formed, uniformly distributed over the entire surface of the walls. However, whereas diagonal orientation of cracks can be clearly identified at all precompression levels, the step-formed cracks passed the bed- and head-joints at low precompression level (Figure 3a), but passed the units at high precompression (Figure 4a). Also, at low precompression the units rotated at increased amplitudes of imposed lateral displacements, whereas at high precompression, the rotation was prevented and the units started cracking and crushing. Displacement and energy dissipation capacities were higher at low precompression level (Figures 3b and 4b).

Test results are summarized in Table 1, where the resistance and displacement values are given at typical limit states, defined in the behaviour of the tested walls, such as crack (damage) limit state, determined by the lateral load and displacement (H_{cr}, d_{cr}), where the first cracks occur in the walls, causing evident changes in the stiffness of the wall; maximum resistance, determined by the maximum attained resistance of the wall and corresponding displacement ($R_{w,exp} = H_{max}$, d_{Hmax}), and ultimate state (collapse), determined by the maximum attained displacement of the wall and corresponding lateral resistance (d_{max}, H_{dmax}). For easier correlation, the values of displacements are given also in the nondimensional form of rotation $\Phi = d/h$ (in %). It is to note that the following vertical loads, acting on the walls, correspond to individual precompression levels: σ_o = 0,92 MPa – V = 690 kN, σ_o = 0,62 MPa – V = 465 kN, and σ_o = 0,34 MPa – V = 255 kN. Average values, obtained by testing a group of three specimens, are given in the table.



a) **b)**

Figure 3: Typical Damage Pattern (a) and (b) Hysteresis Loops, Obtained by Testing the Wall at Low Precompression Level (0.3 MPa)



a) **b)**

Figure 4: Typical Damage Pattern (a) and (b) Hysteresis Loops, Obtained by Testing the Wall at High Precompression Level (0.9 MPa)

Table 1: Test Results

Walls	σ_o (MPa)	Crack limit			Maximum resistance			Collapse		
		H_{cr} (kN)	d_{cr} (mm)	Φ_{cr} (%)	H_{max} (kN)	d_{Hmax} (mm)	Φ_{Hmax} (%)	H_{du} (kN)	d_u (mm)	Φ_u (%)
1, 2, 3	0,92	235	1,8	0,09	303	8,0	0,41	131	16,7	0,84
4, 5, 6	0,62	208	2,3	0,12	221	8,9	0,45	111	25,0	1,26
7, 8, 9	0,34	119	2,3	0,12	130	14,0	0,71	71	40,0	2,02

ASSESSMENT OF SHEAR RESISTANCE OF MASONRY WALLS

Various methods and equations have been already proposed for the assessment of the shear resistance of unreinforced masonry walls, characterised by diagonal cracking. Turnšek and Čačovič [3] introduced the hypothesis that the tensile strength of masonry, conventionally defined as the principal tensile stress developed at the attained maximum resistance of a masonry wall, assuming that the wall is elastic, homogeneous and isotropic panel, determines the shear resistance of the wall. Following this idea, the situations where the diagonal cracks pass either the mortar joints or masonry units, or both, is covered by the same equation. According to Mann and Müller [4], however, the shear resistance is calculated depending on the path of the diagonally oriented cracks. In the case where the cracks pass the vertical and bed joints (step-formed cracks - friction failure of the bed joints), the resistance is defined by the friction law introducing cohesion and friction coefficients as the critical parameters. In the case where the cracks pass the units, however, the tensile strength of the unit is critical and the equation for the calculation of the shear resistance of the wall is of the same form as the one proposed by Turnšek and Čačovič. Recently, Calderini, Cattari and Lagomarsino [5] summarized the development of models for the calculation of the in-plane resistance of masonry walls.

Although substantial amount of experimental and analytical research to study the behaviour of masonry walls subjected to shear has been carried out, the recent European standard for the design of masonry structures, Eurocode 6 [2], requires that only sliding shear failure mechanism (similar as the friction failure of the bed joints according to Mann and Müller [4]), with initial shear strength instead of cohesion and prescribed value of the friction coefficient as the governing parameters, be used for the assessment of the shear resistance of unreinforced and confined masonry walls.

The results of cyclic shear tests of walls described in the previous chapters have been used to compare the experimentally obtained values with the values, calculated by using various shear failure models and assumptions. According to Eurocode 6, the design shear resistance of a masonry wall, $R_{\mathrm{dw,EC6}}$, is calculated by:

$$R_{\mathrm{dw,EC6}} = \frac{f_{\mathrm{vk}}}{\gamma_{\mathrm{M}}} t\, l_{\mathrm{c}} \ , \qquad (1)$$

where $f_{\mathrm{vk}} = f_{\mathrm{vko}} + 0.4\ \sigma_{\mathrm{o}}$, and $l_{\mathrm{c}} = 3\left(\frac{l}{2} - e\right)$, where f_{vk} = the characteristic shear strength of masonry, γ_{M} = partial safety factor for masonry, t = the thickness of the walls, l_{c} = the length of the compressed part of the wall, σ_{o} = the average vertical stress over the compressed part of the wall that is providing shear resistance, f_{vko} = the characteristic initial shear strength of masonry at zero compression, $e = H\,h/V$ is the eccentricity of the vertical load, h = the height of the wall. The expression for l_{c} should be considered in the case where the eccentricity of axial load, e, exceeds 1/6 of the wall's length.

To compare the calculated results with experiments, mean values of the initial shear strength (f_{vo} = 0.18 MPa) without reduction (γ_{M} = 1.0) have been taken into account. The results of calculations are given in Table 2. The bottom (left-hand column) and the mid-height section (right-hand column) of the walls have been verified. As can be seen, in the particular case tested the section at the mid-height level can be considered as fully compressed. As expected, the results of calculations depended on the compressed length of the section.

Table 2: Shear Resistance of Walls According to EC 6

σ_o (MPa)	0.92		0.62		0.34	
l_c (cm)	144	250	125	250	107	241
σ_o (MPa)	1.59	0.92	1.24	0.62	0.79	0.35
f_v (MPa)	0.82	0.55	0.67	0.43	0.50	0.32
$R_{w,EC6}$ (kN)	354	411	254	321	160	232
$R_{w,exp}$ (kN)	303		221		130	

Although the actual failure mechanism observed during the tests did not confirm the assumptions of calculations recommended by Eurocode 6 (see Figures 4 and 5), relatively good correlation between the calculated and experimental results has been obtained in the case where the compressed length at the bottom section of the walls has been taken into account. The calculated values overestimated the experimental ones by approximately 20 %. In the case where the non-reduced section has been considered as fully compressed, the results significantly overestimated the actual situation.

For comparison, the shear resistance of the walls has been also calculated by the Mann and Müller [4] friction failure of the bed joint formula:

$$R_{w,fr} = \tau A_w = (k' + \mu' \sigma_o) A_w, \tag{2}$$

where $A_w = tl$ = the area of the horizontal cross section of the wall, k' = the reduced cohesion, and μ' = the reduced friction coefficient. It has been assumed that cohesion is equal to the initial shear strength of masonry, $k = f_{vo}$= 0.18 MPa, and the friction coefficient is the same as assumed in Eurocode 6, μ = 0.4. Reduced values have been calculated by taking into account the actual shape of masonry units, resulting into cohesion and friction coefficient reduction factors:

$\dfrac{1}{1+\dfrac{2\Delta x}{\Delta y}} = 0.33$, where $\Delta x = \Delta y$ = 250 mm are the length and height of the unit, respectively.

Following the proposal of Mann and Müller [4], the values of k' = 0.06 MPa and μ' = 0.13 have been obtained. The calculated values of the shear resistance, assuming that the friction failure of the bed joints is critical, are compared with the experimental values in Table 3. Without reduction of values of cohesion and friction coefficient, identical values of the shear resistance are obtained as in the case of the Eurocode 6 method, where the whole length of the wall is under compression.

Table 3: Shear Resistance of Walls According to Mann and Müller (Friction Failure of the Bed Joints)

σ_o (MPa)	0.92	0.62	0.34
$R_{w,fr}$ (kN)	135	105	78
$R_{w,exp}$ (kN)	303	221	130

As can be seen, the results of calculations underestimate the experimental values even in the case of low precompression, where the sliding of the units on the bed-joints (step-formed diagonal cracks) has been actually observed. No data regarding the tensile strength of masonry units have been available to verify the Mann and Müller [4] proposal for the case where the shear resistance of the walls is determined by the failure due to cracking of the units:

$$R_{w,cr} = A_w \left(\frac{\beta_{zst}}{2.3} \sqrt{1 + \frac{\sigma_o}{\beta_{zst}}} \right), \tag{3}$$

where β_{zst} = the tensile strength of masonry units. Therefore, β_{zst} has been evaluated from the results of tests of walls, subjected to maximum precompression where the cracking of units has been actually observed. The value of β_{zst} = 0.58 MPa has been obtained, which yields the values of the shear resistance of the walls, given in Table 4.

Table 4: Shear Resistance of Walls According to Mann and Müller (Failure Due to Cracking of Units)

σ_o (MPa)	0.92	0.62	0.34
$R_{w,cr}$ (kN)	303	272	238
$R_{w,exp}$ (kN)	303	221	130

Obviously, identical values as obtained experimentally have been obtained in the case of the walls, used for the evaluation of the tensile strength of units. In the case of the low precompression, the calculations significantly overestimate the resistance.

The method to determine the tensile strength, f_t, as defined by Turnšek and Čačovič [3] is not standardized. However, it has been already shown that comparable results can be obtained by cyclic lateral resistance tests, simple racking tests as well as diagonal compression tests of masonry walls [6]. The specimens with geometry aspect ratio h/l = 1.5 are usually tested. Unfortunately, in the specific case studied, no specimens have been tested to determine the tensile strength. Consequently, the value has been determined by evaluating the results of tests of walls, which exhibited clear shear behaviour, i.e. the walls, where diagonally oriented cracks formed passing through the units (precompression level σ_o = 0.92 MPa). The mean value of f_t = 0.18 MPa has been obtained in this particular case.

According to Turnšek and Čačovič [3], design shear resistance of an unreinforced masonry wall, $R_{dw,ft}$, is determined by:

$$R_{\mathrm{dw,ft}} = A_{\mathrm{w}} \frac{f_{\mathrm{tk}}}{\gamma_{\mathrm{M}}} \frac{1}{b} \sqrt{\frac{\gamma_{\mathrm{M}}}{f_{\mathrm{tk}}} \sigma_{\mathrm{o}} + 1}\,, \tag{4}$$

where b = the shear stress distribution factor, which depends on the geometry of the wall and the ratio between the vertical load, V, and maximum horizontal load, $H_{\max}$. In the case where the aspect ratio is equal to or greater than $h/l = 1.5$, the value of $b = 1.5$ can be assumed. However, in the case of squat walls, $b = 1.1$. This value has been taken into account when evaluating the tensile strength and calculating the shear resistance of the walls under consideration. To compare the calculated results with experiments, mean value of the tensile strength without reduction ($\gamma_{\mathrm{M}} = 1.0$) has been taken into account. The results of calculations are given in Table 5.

Table 5: Shear Resistance of Walls According to Turnšek and Čačovič (Diagonal Tension Failure)

σ_o (MPa)	0.92	0.62	0.34
$R_{\mathrm{w,ft}}$ (kN)	303	259	209
$R_{\mathrm{w,exp}}$ (kN)	303	221	130

Again, identical values as obtained experimentally have been obtained in the case of the walls, used for the evaluation of the tensile strength. Against expectations, however, the correlation between the calculated and experimental values at low level of precompression is not good. All calculated values are compared in Table 6.

Table 6: Shear Resistance of Walls: Comparison

	Precompression (MPa)					
	0.92		0.62		0.34	
	R (kN)	$R_{\mathrm{cal}}/R_{\mathrm{exp}}$	R (kN)	$R_{\mathrm{cal}}/R_{\mathrm{exp}}$	R (kN)	$R_{\mathrm{cal}}/R_{\mathrm{exp}}$
Experimental R_{exp}	303		221		130	
Eurocode 6	354	1.17	254	1.15	160	1.23
Mann- Müller (friction)	133	0.44	104	0.47	74	0.57
Mann- Müller (unit cracking)	303*	1.00	272	1.23	238	1.83
Turnšek-Čačovič (diag. tension)	303*	1.00	259	1.17	209	1.61

*mechanical properties evaluated from tests of the same walls

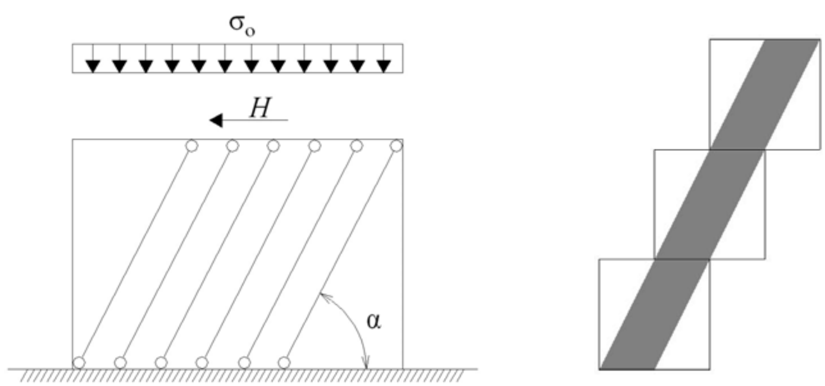
Although sliding shear (friction failure) mechanism has not been observed during experiments, the calculations based on such mechanism (Eurocode 6) yielded acceptable, though overestimated values at all levels of precompression. Similar values of the shear resistance at high and medium precompression has been obtained also in the case where the equations based on either diagonal tension failure (Turnšek and Čačovič) or cracking of units criteria (Mann and Müller) have been used. This type of mechanism actually occurred at medium and high precompression levels, i.e. at $\sigma_o = 0.62$ MPa (10 % of compressive strength) and $\sigma_o = 0.92$ MPa (15 % of compressive strength). At low level of precompression, i.e. at $\sigma_o = 0.34$ MPa (5 % of compressive strength), where diagonal cracks passed the bed and head joints, these methods by far overestimated the actual shear resistance.

COMPRESSIVE STRUT MODEL

On the basis of the observed damage pattern (see Figure 4a) the idea to model the ultimate state of the wall with a system of compressive struts, oriented diagonally and separated by cracks, which form in the walls in the non-linear range, has been used to calculate the shear resistance (Figure 5a). Such mechanism offers maximal resistance when the resultant of the horizontal and vertical loads aligns with the direction of struts. The inclination angle, α, depends on the units' height/length ratio and overlap of the head joints (Figure 5). In the particular case tested, the units' height/length ratio was 1.0 with 50 % of the unit's length overlap, resulting into $\alpha = \text{arc tan}(2) \approx 63°$. As can be seen by comparing crack patterns, shown in Figures 4a and 5a, in the particular case tested the inclination of cracks did not depend on the mechanism, i.e. on the level of precompression. It was the same in the case of cracks passing the units as was in the case of cracks passing the head and bed joints.

The lateral resistance of such system, $R_{\mathrm{w,ds}}$, is given simply by:

$$R_{\mathrm{w,ds}} = \frac{\sigma_{\mathrm{o}} A_{\mathrm{w}}}{\tan \alpha}. \tag{5}$$



a) **b)**

Figure 5: Compressive Strut Model

Table 7: Shear Resistance of Walls Calculated by Compressive Strut Model

σ_{o} (MPa)	0.92	0.62	0.34
$R_{\mathrm{w,strut}}$ (kN)	346	234	128
$R_{\mathrm{w,exp}}$ (kN)	303	221	130

As can be seen in Table 7, where the calculated results are compared with experimental values, good correlation between the experiments and calculations has been obtained at all cases of precompression. This indicates that the idea has some validity. However, further research is needed to answer the question at which geometry characteristics of walls and boundary conditions such mechanism actually develops. Also, the model should be further developed by introducing the strength characteristics of masonry in a proper way. Obviously, the resistance of the wall does not depend only on its geometry and the level of precompression.

CONCLUSIONS

The analysis of results of cyclic lateral resistance tests of nine equal walls, tested at different levels of precompression indicated that, although the walls failed in shear, shear mechanisms depended on the level of precompression. At precompression, higher than 10 % of the compressive strength of masonry, diagonally oriented shear cracks developed in the wall, passing mainly through the units. At precompression, equal to only 5 % of the compressive strength, however, shear cracks passed the bed and head joints. In both cases, the inclination of cracks was the same. Good correlation between the experimental results and calculations, using diagonal tension strength based equations, has been obtained in the case of the shear mechanism, where the diagonal cracks passed the units. Against expectations, however, similarly good correlation between experimental and calculated results has been obtained also by the calculations based on the sliding shear mechanism (Eurocode 6), although the actual failure mechanism observed during the tests did not confirm such assumptions. This indicates that after decades of research and testing, additional work is still needed before making final conclusions regarding the modelling of the shear mechanism and assessing the shear resistance of masonry walls.

A simple, compressive strut model has been proposed to assess the shear resistance of the tested walls. It is based on the assumption that maximal shear resistance is attained when the resultant of the horizontal and vertical loads aligns with the direction of struts formed in the wall in the non-linear range. The inclination angle depends on the units' height/length ratio and overlap of the head joints. Although good correlation between the experimental results and calculated values at all levels of precompression indicates the validity of idea, further research is needed to determine at which geometry, boundary conditions, and strength characteristics of walls such mechanism develops.

REFERENCES

1. Eurocode 6 (2004) "Design of masonry structures - Part 1-1: Common rules for reinforced and unreinforced masonry structures". EN 1996-1-1, CEN: Brussels.
2. Eurocode 8 (2004) "Design of structures for earthquake resistance, Part 1: General rules, seismic actions and rules for buildings". EN 1998-1. CEN, Brussels.
3. Turnšek, V., Čačovič, F. (1970) "Some experimental results on the strength of brick masonry walls" Proc. of the 2nd International Brick Masonry Conference, Stoke-on-Trent.
4. Mann, W., Müller, H., (1980) "Failure of shear-stressed masonry – An enlarged theory, tests and application to shear-walls" Proc. of the International Symposium on Loadbearing Brickwork, London, 1-13.
5. Calderini, C., Cattari, S., Lagomarsino, S., (2008) "In-plane strength of unreinforced masonry piers" Earthquake Engng. Struct. Dyn., Published on-line at http://www3.interscience.wiley.com/journal/121502319/abstract.

6. Bernardini A, Modena C, Turnšek, V, Vescovi, U (1980) "A comparison of three laboratory test methods used to determine the shear resistance of masonry walls". Proc. of the 7th World Conference on Earthquake Engineering. International Association for Earthquake Engineering, Istanbul, Vol.7: 181-184.