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Shear modulus and stiffness of brickwork masonry: An experimental perspective

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Abstract. Masonry is a composite non-homogeneous structural material, whose mechanical properties depend on the properties of and the interaction between the composite components – brick and mortar, their volume ratio, the properties of their bond, and any cracking in the masonry. The mechanical properties of masonry depend on the orientation of the bed joints and the stress state of the joints, and so the values of the shear modulus, as well as the stiffness of masonry structural elements can depend on various factors. An extensive testing programme in several countries addresses the problem of measurement of the stiffness properties of masonry. These testing programs have provided sufficient data to permit a review of the influence of different testing techniques (mono and bi-axial tests), the variations caused by distinct loading conditions (monotonic and cyclic), the impact of the mortar type, as well as influence of the reinforcement. This review considers the impact of the measurement devices used for determining the shear modulus and stiffness of walls on the results. The results clearly indicate a need to re-assess the values stated in almost all national codes for the shear modulus of the masonry, especially for masonry made with lime mortar, where strong anisotropic behaviour is in the stiffness properties.

Key words: masonry; shear modulus; stiffness; experimental; compressive test; diagonal test; shear test; static and dynamic loading.

1. Introduction

Current code procedures for determination of the shear modulus assume the behaviour of the masonry follows an isotropic material. The advantages of this assumption are that the simplest theory of elasticity provides the analysis procedures. This isotropic assumption is at best

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questionable for determining the behaviour of historic masonry. Observations of historic mortar show it is traditionally weak in the joints, so the common isotropic assumptions about the theoretical behaviour create a model yielding different results from actual measured behaviour of historic masonry. Masonry made with weak mortar cracks along the line of least resistance rather than along the line of the main principal stresses, negating the isotropic assumption from the lowest applied loads. There is no current harmonized method for the evaluation of the shear modulus of masonry; typically, it is determined from the static or cyclic compressive, diagonal, or shear tests of the masonry specimens. The purpose of the paper is to review recent masonry research and develop guidelines to provide a method to determine an estimate of the shear modulus. The method provides guidance for estimating the shear modulus for design purposes.

2. Experimental procedures

2.1 Introduction

The development of computer technology has improved the resolution of experimental measurements providing an understanding of the material properties at a finer scale than the representative volume element (RVE) (Krajcinovic 1996). Masonry is one of the world's oldest building materials and yet it represents a challenge to analyse the structural performance because of the distinct directional properties of the material (Page 1973, 1983). Masonry is not an isotropic material nor is it anisotropic rather it exhibits a structured orthotropic response to biaxial loading. The modulus of elasticity for this orthotropic material is different for the direction parallel to the bed joint usually termed the *x* direction from the direction perpendicular to the bed joints usually termed the *y* direction. The elasticity equations for an orthotropic material are:

$$\varepsilon_x = \frac{1}{E_x}\tau_x - \frac{v_{xy}}{E_y}\tau_y \tag{1}$$

$$\varepsilon_y = -\frac{\upsilon_{xy}}{E_x}\tau_x + \frac{1}{E_y}\tau_y \tag{2}$$

$$\varepsilon_{xy} = \frac{1}{2G_{xy}}\tau_{xy} \tag{3}$$

$$v_{yx}E_x = \upsilon_{xy}E_y \tag{4}$$

Where $v_{yx}, \upsilon_{xy}, E_x, E_y$ are the Poisson's ratio for the *yx* and *xy* components and the Young's modulus for the *x* and *y* respectively, ε represent the strain components, τ represent the stress components and G_{xy} is the shear modulus. The elastic matrix representation *D* for plane strain (Asteris 2003) is:

$$D = \begin{bmatrix} \dfrac{E_x}{1-\upsilon_{xy}\upsilon_{yx}} & \dfrac{E_x\upsilon_{yx}}{1-\upsilon_{xy}\upsilon_{yx}} & 0 \\ \dfrac{\upsilon_{xy}E_y}{1-\upsilon_{xy}\upsilon_{yx}} & \dfrac{E_y}{1-\upsilon_{xy}\upsilon_{yx}} & 0 \\ 0 & 0 & G_{xy} \end{bmatrix} \tag{5}$$

This reduces to:

$$G_{xy} = \frac{\sqrt{E_x E_y}}{2(1 + \sqrt{\upsilon_{xy}\upsilon_{yx}})} \tag{6}$$

Which results in one of the fundamental Eq. (7) of mechanics for an isotropic material based on the assumption of a RVE and the assumption that the Young's modulus and Poisson's ratio are independent of the basis of the analysis established from Eq. (6):

$$G = \frac{E}{2(1 + \upsilon)} \tag{7}$$

Where G, E and υ are respectively the shear modulus, Young's modulus and Poisson's ratio. A simple design assumption using a Poisson's ratio of 0.25[1], gives the usual isotropic equation for the shear modulus as $G = 0.4E$ (Lekhnitskii 1963). This ratio is also presented in almost all national masonry building codes, including the latest draft of the Eurocode 6 stating "In the absence of a more precise value, it may be assumed that the shear modulus, G, is 40% of the elastic modulus E") (Eurocode 1995).

One example of anisotropic masonry material is masonry made with soft lime mortars, which behave in masonry structures subjected to earthquakes in a significantly different manner to hard brittle mortars (Nichols 1990). The masonry shear modulus, calculated from the effective stiffness obtained from shear tests on masonry walls, may vary from 6 to 25% of the measured elastic modulus of the masonry. The shear modulus is an intrinsic property of a material and the results should not depend on the test method (Turnšek and Čačovič 1971, Krajcinovic 1996, Tomaževič 1999). A significant number of studies have been completed into the stiffness of masonry walls with state of the art reviews presented in two recent studies (Hart *et al.* 1988, Elshafie *et al.* 2002).

The experimental programme to determine a non-isotropic approximation to the shear modulus included several types of experiments including compressive, diagonal, and two types of shear tests of masonry walls (Fig. 1).

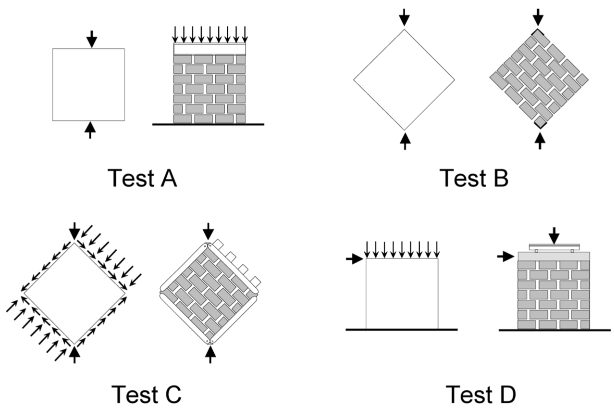


Fig. 1 Various test procedures of masonry structural elements

[1]A mid-range value which is a reasonable starting point for design.

Compressive (Test A) and diagonal tests (Test B) are mono-axial tests, with different inclination of the main stresses towards the bed joints. For diagonal tests it is characteristic that, both normal and shear components of stresses in the bed joints increased with the increasing load. Shear tests (Test C & D) are biaxial tests. The normal stresses in bed joints are almost constant during the shear tests, whilst the shear stresses are applied as a harmonic load.

The experimental procedures detail the mortar and brick types, and provide a set of simple sketches of the test arrangements. The experimental work used extruded clay bricks with a nominal thickness of 120 mm, except for Test D completed using dry pressed clay bricks with a nominal thickness of 110 mm. All joints were finished flush.

2.2 Mortar types

The basic mortars used were a cement mortar (CM), a cement lime mortar (CLM), and a lime mortar (LM). The cement mortar represents an assumed nominal isotropic material typical of modern construction where the mortar is relatively strong; the lime mortar represents a soft historic mortar with nominal anisotropic properties, and the cement lime mortar represents a typical contemporary mortar used extensively in the last five decades. The mixes by volume of sand, cement and lime were CM (4:1:0), CLM (6:1:1), and LM (3:0:1). A fourth mortar type is designated CLMR as this was reinforced with a polymer coated glass-fibres mesh (with square openings up to 5 mm) embedded within the bed joint.

2.3 Test procedures

A summary table lists the test methods for the six tests (Table 1) shown in Fig. 1.

Test A uses an Instron with a capacity of 1MN to apply a slowly increasing monotonic load to the test panels. This test procedure is in accordance with the European prenorm (prEN 1993) (Fig. 2).

Test B uses an Instron with a capacity of 250 kN to test in accordance with ASTM standards (ASTM 1998) in a monotonic test procedure. Eight deformeters with the accuracy of one μm and three Linear Velocity Displacement Transducers (LVDT) with the accuracy of ten μm measured the local and global deformations in two perpendicular directions. Two LVDT patterns measured the changes in properties of the panels with the applied load. “Big cross” refers to the vertical and horizontal deformations measured with the LV10 and average values from LV9 and LV11 and the

Table 1 Test methods for the experimental program

Test	Test method basis	Type of test protocol
A	European pre-norm prEN 1052-1	Compressive tests of masonry wallettes with monotonic loading in a displacement-controlled mode (0.3 mm/min)
B	ASTM C1391	Monotonic loading with a displacement-controlled mode (0.3 mm/min)
C	Nichols (2000)	Square masonry shear walls at different levels of precompression and with dynamically applied shear The shear load was also sinusoidal with various frequencies
D	Shear tests with harmonic seismic frequency	On the masonry cantilever walls, with the constant level of precompression and with imposed lateral loading history in sinusoidal cyclic manner

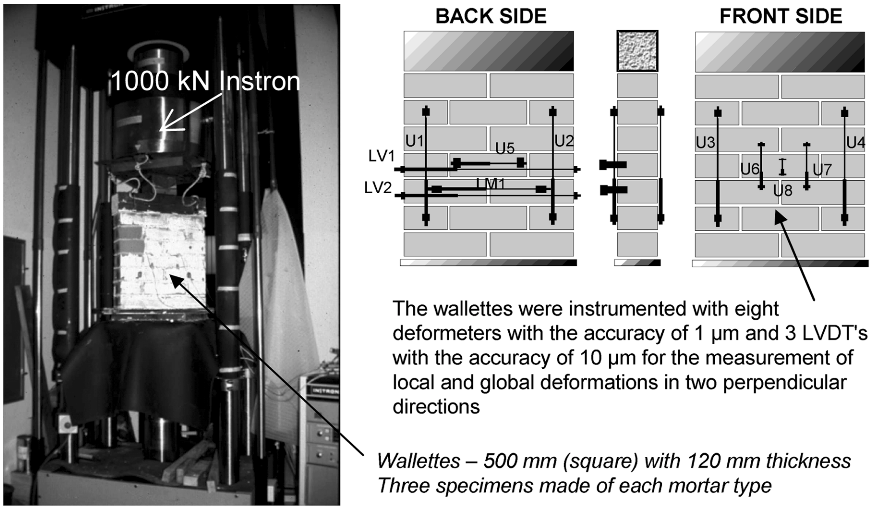


Fig. 2 Test type A – Setup

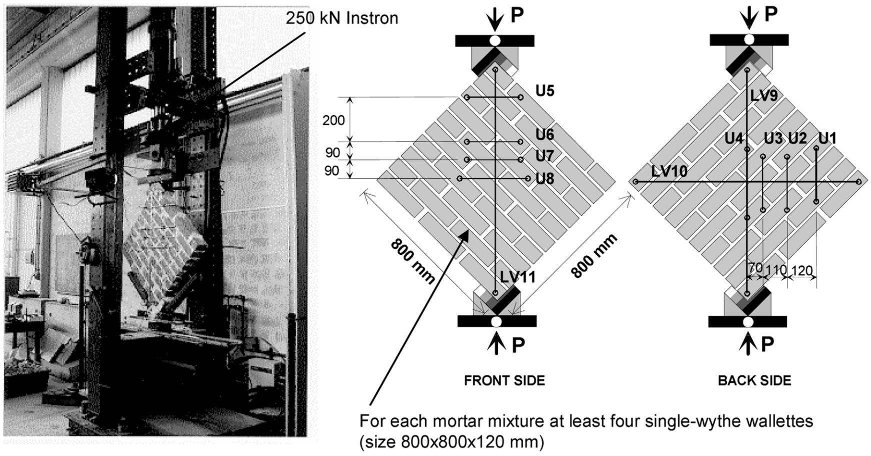


Fig. 3 Test type B – Setup

term “Small cross” refers to the measurements of the deformations from measuring devices U4 and U8 (Fig. 3).

Fig. 4 provides the defining sketch for calculating the stress and strain data for test setup type B, where P is the applied force, the breadth is b, the depth of the section is t, the diagonal length is d, the vertical diagonal shortening is δ_v, and the horizontal diagonal extension is δ_h.

Where the shear stress, τ_{xy}, the shear strain, γ_{xy} and a shear modulus, G_d, is defined as:

$$\tau_{xy} = P/\sqrt{2}bt \tag{8}$$

$$\gamma_{xy} = \left(\tan\alpha + \frac{1}{\tan\alpha}\right)\left(\frac{\delta_v + \delta_h}{2d}\right) \tag{9}$$

$$G_d = \frac{\tau_{xy}}{\gamma_{xy}} \tag{10}$$

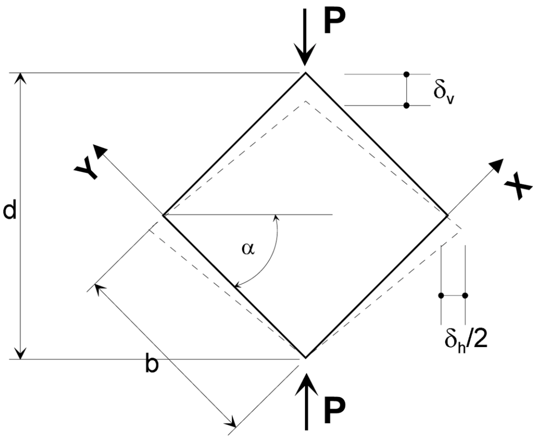


Fig. 4 Shear stress and strain – Definition sketch

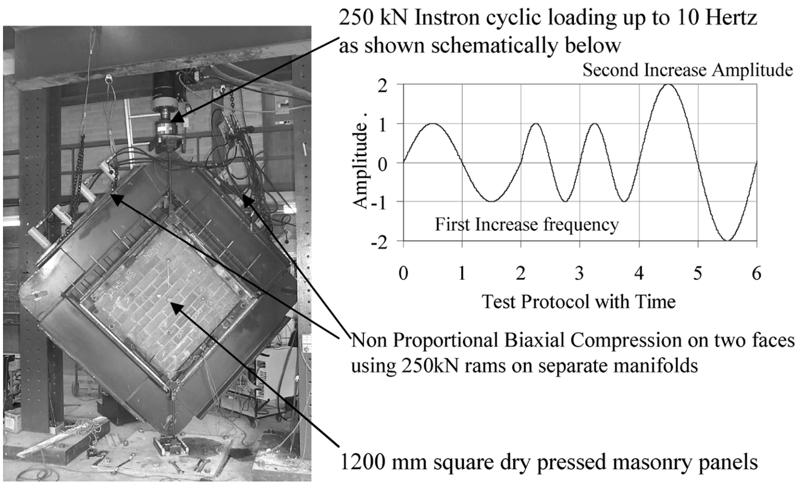
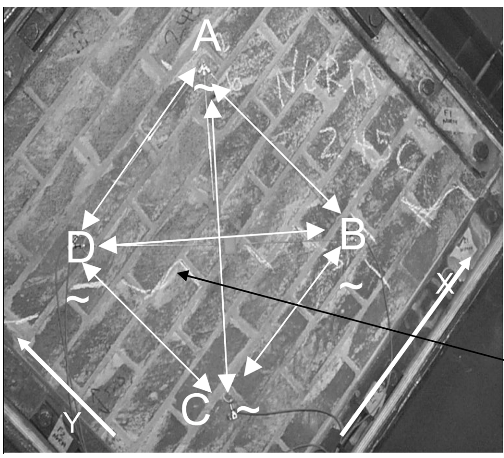


Fig. 5 Type C setup



The shear strain was determined from the change in lengths of AC, AD and AB using the standard rosette equations (Megson, 1991,2002) and the second shear strain from BD, AB and AD. (The ~ marks each measurement point). The chalk failure line is evident in the photograph.

Fig. 6 Type C – Failure mode

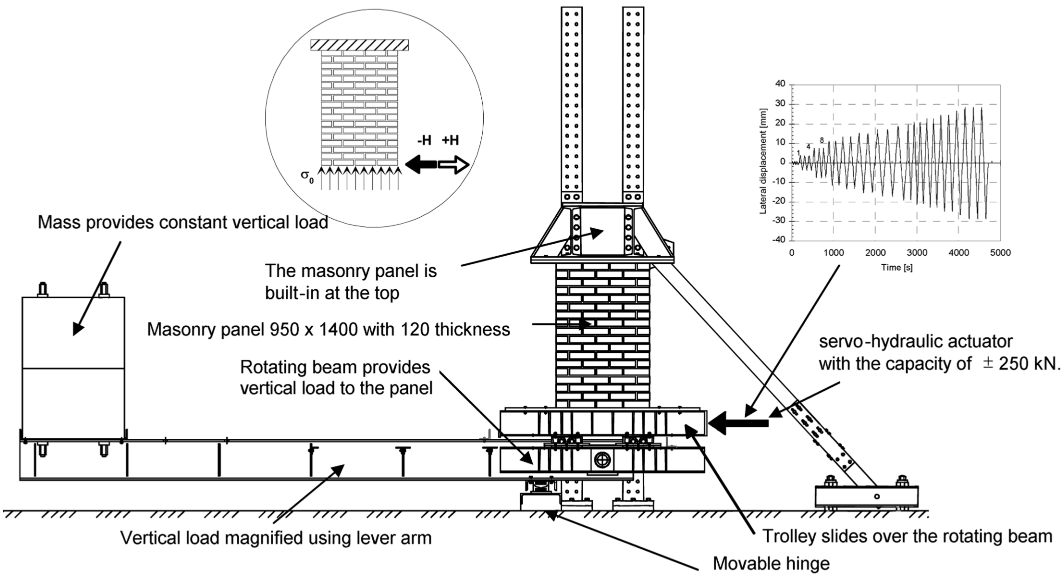


Fig. 7 Type D setup

Test D uses an Instron with a capacity of 250 kN to test masonry panels using cyclic frequencies ranging from 0.1 to 10 Hertz (Nichols 2000) (Fig. 5 and Fig. 6).

The rig has two independent mechanisms. One mechanism combines a rectangular pin jointed compression frame with four 250 kN rams on two sides providing the bi-axial, non-proportional, static compression perpendicular and parallel to masonry bed joints. Another mechanism consists of a harmonic shear load rig attached at the top to a 250 kN INSTRON actuator applying a cyclic load of varying amplitude and frequency. The design of the system with the applied compression stress in the masonry means that the system has the first principal stress in compression at the start point of each test run. The applied harmonic shear drives the first principal stress into tension across the plane BD (Fig. 6) ultimately causing failure along the chalked line.

Test setup D allowed the tests of the masonry cantilever walls, with the constant level of precompression and with imposed lateral loading history in sinusoidal cyclic manner (Fig. 7). The testing procedure allowed the rotation and horizontal displacement to be free (released) on the lower edge of the panel. The vertical compression was applied first and then the horizontal harmonic load was applied as a second load. This shear test measured the resistance of the walls to the seismic loading.

The level of constant vertical precompression for the tests was selected at a sixth of the mean compressive strength for each type of masonry. This relationship is termed the "same relative level of precompression". Three masonry panels with dimensions 950×1400 mm and thickness of 120 mm were constructed using each of the different mortars. In the case of the CLM a larger number of specimens allowed for a greater number of shear tests using five different absolute levels of precompression.

In-plane deformations of the wall panel were measured and monitored with 12 deformeters with the accuracy of 1/100 mm and 11 LVDT's with the accuracy of 1/10 mm. With deformeters U1 and U2 the overall vertical deformations were monitored. On the record of these deformeters the opening of the flexural crack were monitored. With U3-U6 the boundary conditions were monitored

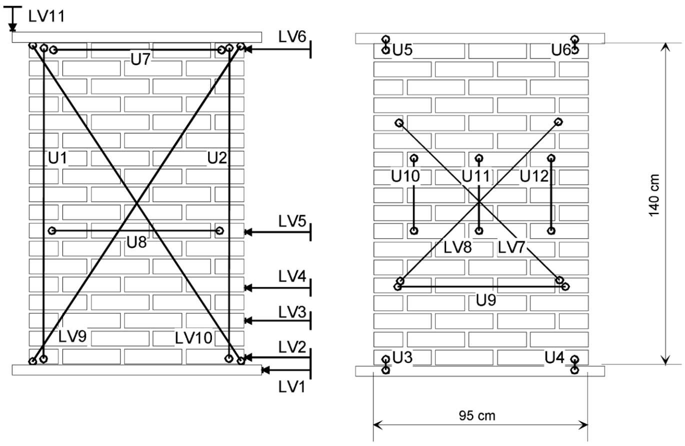


Fig. 8 Instrumentation of shear tests of masonry panels

as well as potential de-bonding of the masonry panel from the steel plates. With the deformeters U7-9 in the combination with LVDT's LV4-6 the potential slippage and the opening of the shear cracks were monitored. Vertical deformations in the middle third of the specimens were monitored with the deformeters U10-12. With those measuring the changes of the boundary conditions to the axis of symmetry of the panel were monitored. LVDT's LV1-6 measured the in-plane curvature of the specimen. The difference between LV1 and LV2 was indicator for potential slippage between the panel and steel plate. The measuring pair of devices LV7-8 ("Small cross") and LV9-10 ("Big cross") are standard measuring position (Fig. 8).

3. Experimental results

3.1 Experiment type A - results

The results of testing, which are presented in detail elsewhere (Bosiljkov *et al.* 2000, 2003), are in short presented in following table:

Table 2 Results of compressive tests

Mortar type	CM	CLM	LM	CLMR
Young's modulus (GPa)	12.6 ± 0.75	12.5 ± 1.2	1.8 ± 0.2	10.3 ± 1.1
Coefficient of variation (%)	6	10	12	11
Poisson's ratio	0.07 ± 0.03	0.25 ± 0.03	0.4 ± 0.3	0.28 ± 0.11
Coefficient of variation (%)	43	12	78	41
Shear modulus (GPa)	5.9	5.0	0.64	4.0

There are two important considerations in determining Young's modulus and Poisson's ratio and thus the shear modulus of masonry. The first is the appropriate position of the measuring devices for both vertical and lateral deformations to minimize the effect of confinement and the second is the level of loading at which Poisson's ratio should be evaluated. To obtain representative values of the vertical deformations the average of four results (U1-4) was used (global vertical deformations). The response of measurement devices U6-7, in the early stage of loading was almost zero. At the load level of approximately 60% of the compressive strength, the influence of the mortar/brick ratio in the gauge length of deformeters becomes significant. The values for the elastic modulus obtained from U6-7 were much lower than the values calculated from global vertical deformations read from U1-4, which is typical for confined compressive tests. According to our experience it is advisable for the measurement of lateral deformation to take the average of the two measurements (LV1 and LV2) in two consecutive courses, because the deformations gained through LV1 were slightly higher since its gage length contained more mortar head joints.

The level of compressive loading at which Poisson's ratio, $\nu_{w,(30\%)}$, is determined usually correlate with the level at which the secant modulus of elasticity, $E_{w,(30\%)}$, is determined. Most of the researchers choose the level of 30% or 60% of the achieved compressive strength of the masonry. However, the first cracking of masonry could occur before 30% of the compressive strength is achieved. This could result in unrealistic values for the Poisson's ratio of more than 0.5.

In our analysis Poisson's ratio was determined either at 30% of the compressive strength or at the beginning of cracking, whichever happened first. The shear modulus, G_c, in the direction of the bed joint was calculated according to the formula for elastic isotropic material.

$$G_c = \frac{E_{w,\,(30\%)}}{2 \cdot (1 + \nu_{w,\,(30\%)})} \tag{11}$$

Comparing the calculated values of the shear modulus and elastic modulus (Fig. 9), we can see that their ratio is very close to the value of 0.4 which is stated in most national codes for masonry structures. Note also that the experimentally evaluated Poisson's ratio has the high scattering and thus a high coefficient of variation (C.O.V.) of results. Nichols and Totoev (1997) in a similar set of experiments using masonry prisms determined a range of Poisson's ratios from 0.17 to 0.29 for a CLM mortar, which yielded a range of 0.38 to 0.42 for the shear modulus to Young's modulus ratio.

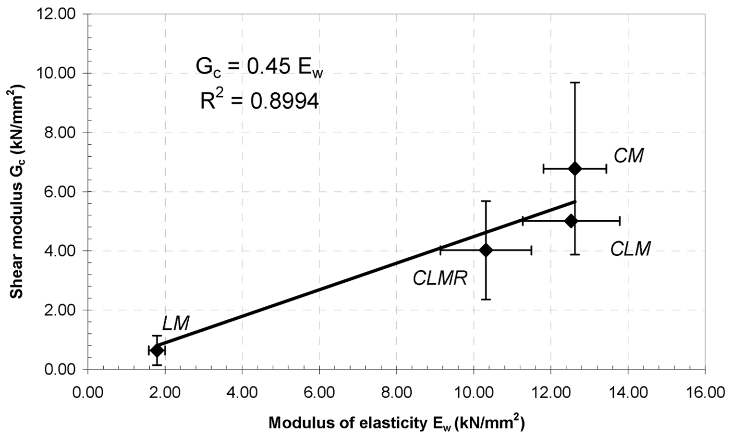
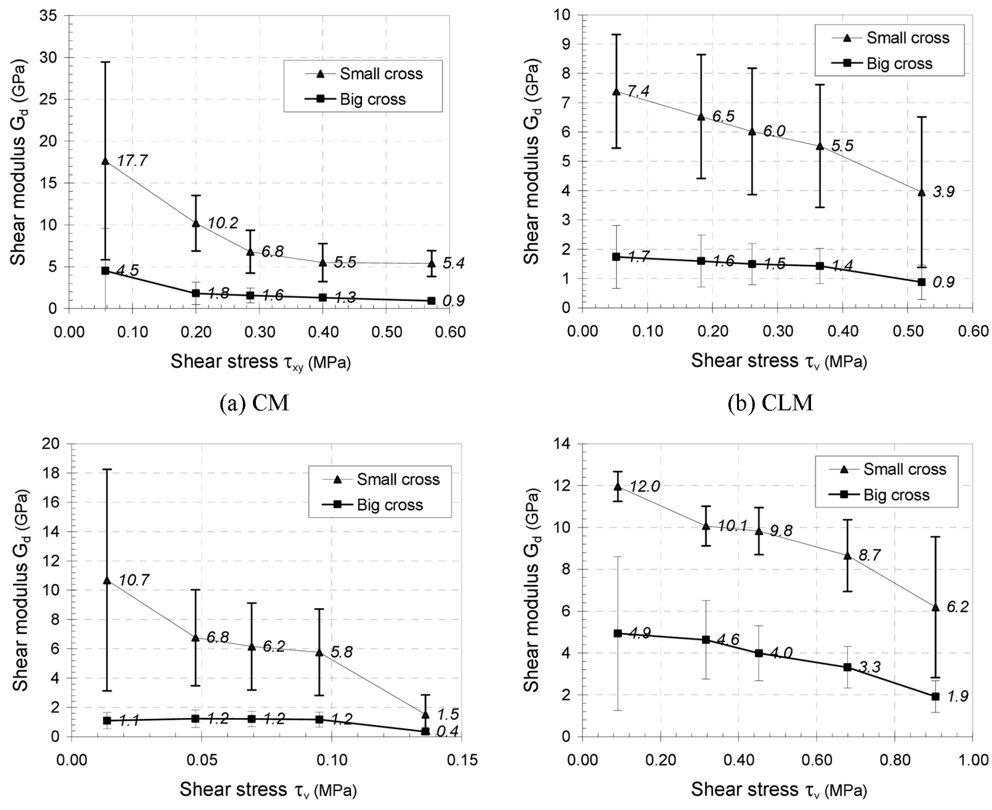


Fig. 9 Ratio between elastic and shear modulus – compressive tests

3.2 Experimental type B - results

Considering the failure modes of diagonally loaded masonry specimens we have to note that both stiff mortar mixtures (CM and CLM), showed very brittle failure. Most ductile behaviour was observed for reinforced masonry CLMR. Also very ductile behaviour was observed for lime mortar masonry (LM). It was primarily due to the propagation of cracks within the mortar joints and not at the brick-mortar interface as it was the case with other mortar mixes. Note, that the achieved shear strength (Fig. 10) of the LM was also much lower in comparison to other types of masonry. The effective shear stiffness was calculated from the measurements by the big and small cross for each type of masonry at various levels of shear stress (10, 35, 50, 70 and 100% of τ_{xy}). Fig. 10 shows the averaged values for the effective shear modulus.

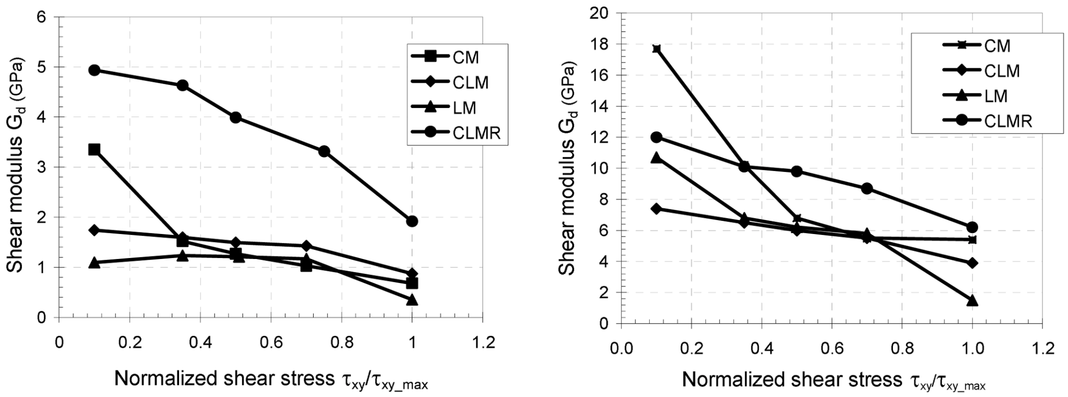
Measurements of deformations on a small area in the middle of the masonry panel, where stress is almost uniform, give values that represent properties of masonry as a construction material. The shape of the diagrams for the effective shear stiffness obtained through the big cross (Fig. 11) shows, that for almost all mortar mixtures there exists a plateau between 35% and 70% of achieved shear strength, which give us some idea of realistic design value for the shear modulus of masonry panels that could be expected due to shear loading.



(a) CM (b) CLM

(c) LM (d) CLMR

Fig. 10 Averaged dependent shear modulus – shear stresses for various mixes



(a) Big cross (b) Small cross

Fig. 11 Shear modulus vs. normalized shear stresses

The influence of the reinforcement on the shear modulus for reinforced masonry can be seen in Fig. 10 and Fig. 11. These results show that the values for shear modulus of reinforced masonry are much higher than for unreinforced masonry (URM) made from the same mortar (CLMR vs. CLM), as well as the effective shear stiffness constant declines with the increase in the shear stresses. The question of suitability of these test methods (diagonal tests) thus arises for the estimation of mechanical parameters of reinforced masonry.

3.3 Experiment type C - results

The complete results of testing are presented elsewhere (Nichols 2000). The typical results are presented in form of the shear modulus degradation curves in Fig. 12.

The degradation of the shear modulus is related to the increasing strain. Because the loading frequency in these tests was 0.4 Hz, which can be regarded as quasi static, the average initial value of the modulus of 7.74 GPa can be taken as the shear modulus of this masonry undamaged. These results are in good agreement with the small cross results of the diagonal tests for CLM (Fig. 10).

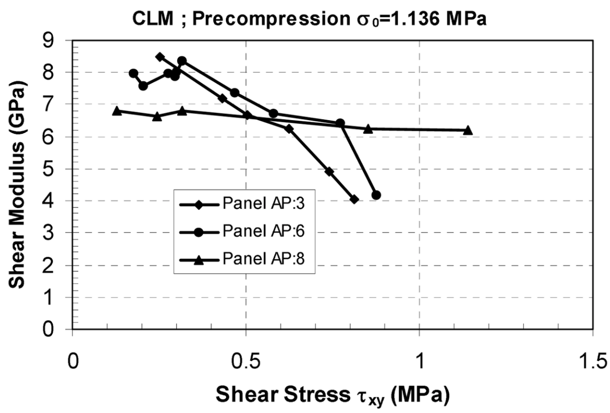
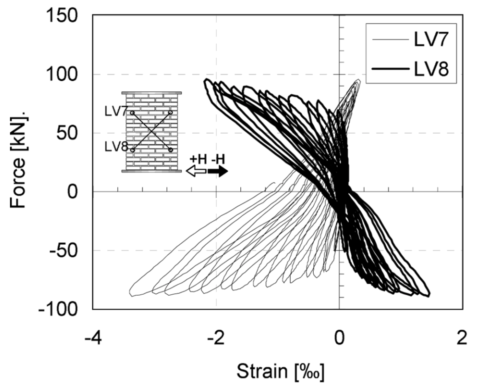


Fig. 12 Effective shear modulus degradation curves

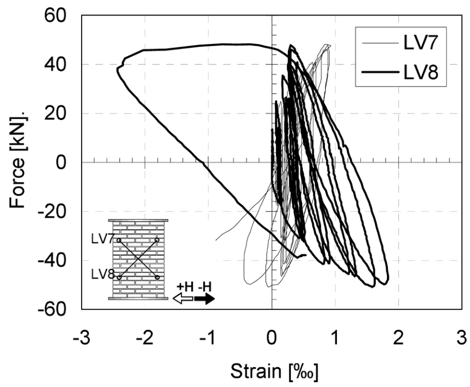
3.4 Experiment type D – results

Each specimen was subjected to a prescribed lateral displacement history under a constant vertical load. The first three cycles were repeated at three different levels (elastic loading) and were the same for each type of masonry. After this the displacement was increasing in a stepwise manner. The magnitude of those steps was different for each type of masonry. The complete results of testing are presented elsewhere (Bosiljkov 2000). The small cross results in the forms of hysteresis loops were hard to interpret, especially for the specimens made from weaker mortar (Fig. 13). Further discussion is based on results recorded from the big cross.

The resistance of the wall gained through the shear tests is usually presented in the shape of hysteresis loops. In order to quantify the overall behaviour of biaxially loaded masonry the first step is evaluation of the envelopes of the hysteresis loops and its idealisation (Fig. 14). The piecewise linear idealisation is traditionally used for hysteresis loops (Tomaževic 1999). The bilinear idealisation of the experimentally gained hysteresis envelope was evaluated in our analysis. It was



(a) CM specimen



(b) LM specimen

Fig. 13 Local response of the specimens recorded with devices LV7 and LV8

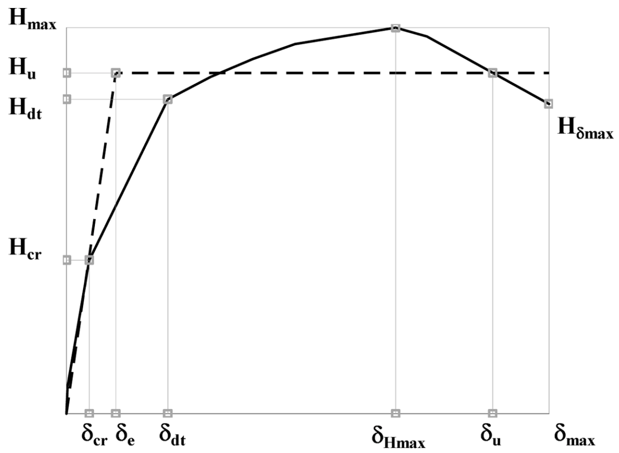
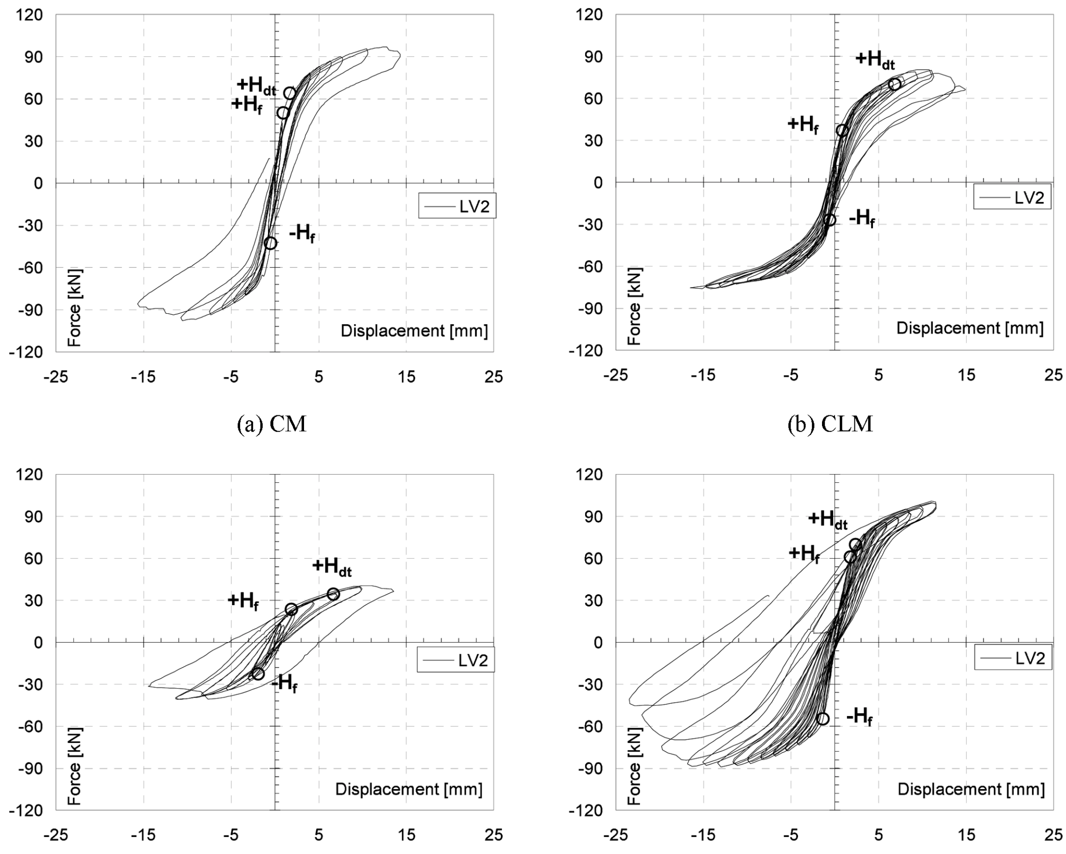


Fig. 14 Hysteresis envelope and its bilinear idealisation

done on the basis of an equal energy dissipation capacity (the areas below the actual and bilinear idealised) of actual and idealised panels.

Four limit states for the tested panels were defined on the idealised envelope curve as follows: (i) Crack or elastic limit, determined by displacement δ_{cr} and resistance H_{cr}; (ii) Shear crack state of resistance was determined by the occurrence of the first shear crack (H_{dt} and displacement δ_{dt}).; (iii) Maximum resistance determined by $H_{\max}$, recorded during test, and corresponding displacement $\delta_{H\max}$; (iv) Ultimate state determined by maximum displacement recorded during a test $\delta_{\max}$ and corresponding resistance $H_{\delta\max}$.

The bilinear idealised curve was defined by idealised resistance H_u and effective stiffness K_{ef} defined by $K_{ef} = H_{cr}/\delta_{cr}$. The criteria for selecting the point H_{cr} determine the effective stiffness of the wall, and so one of the aims of this analysis was also to investigate the influence of the chosen criteria and its impact on the calculated values of effective stiffness. Three selection criteria were reviewed. The first criterion for the term $K_{ef}(1)$ used was $H_{cr} = 0.33H_{\max}$ maintaining the standard definition. The second criterion for the term $K_{ef}(2)$ was chosen as performance perspective, as the resistance of the wall at first flexural crack ($H_{cr} = H_f$) and thus $K_{ef}(2) = H_f/\delta_f$. The third criteria was



(a) CM (b) CLM

(c) LM (d) CLMR

Fig. 15 Typical experimental lateral displacement - lateral resistance hysteresis loops for different types of mortar at the same relative level of precompression chosen as the occurrence of the first shear (diagonal) crack ($H_{cr} = H_{dt}$) and thus $K_{ef}(3) = H_{dt}/\delta_{dt}$. It can be seen from Fig. 15 that all specimens made from stiffer mortar mixtures (CM, CLM and CLMR) exhibited a characteristic rocking behaviour at the same relative level of precompression.

Slightly better dissipation of energy was observed in the CLMR specimens, where in softening range the reinforcement enabled more ductile behaviour of the specimens. In addition to this, the masonry made with weak lime mortar (LM), almost from the beginning of loading exhibited ductile behaviour with considerable energy dissipation. On the other hand, the level of the resistance of LM specimens was much lower in comparison to the stiffer mortar mixtures. In general, for the specimens tested under the same relative level of precompression, the occurrence of the flexural cracks did not change the stiffness of the panels. Shear diagonal cracks were predominantly opened before the beginning of rocking. The failure of the specimens was attributed either to the simultaneous opening of the shear crack with the toe crushing or to the softening of the masonry in the mid-height of the specimen.

The results of testing for different types of masonry in the terms of idealised resistance and stiffness depending on the chosen criteria are presented in Table 3. Comparison of the effective stiffness results (Table 3) for all masonry types according to ANOVA method (Bosiljkov 2000) shows that there is no significant statistical difference between values obtained between the first two criterion. It should be noted that for the purpose of further analysis and discussion that differences between effective stiffness for different types of the masonry at the same relative level of precompression do not necessarily translate into similar differences at absolute levels. A set of tests has been carried out for one mortar type (CLM) in order to analyse the influence of the different absolute levels of precompression both on the stiffness of masonry walls as well as the failure modes.

The different absolute level of precompression had a significant influence on the observed failure modes (Fig. 16). For the lowest level of σ_0, despite the beginning of rocking behaviour in the upper

Table 3 Results of shear tests at the same relative level of precompression σ_0

Criteria	Type of mortar	CM	CLM	LM	CLMR
	σ_0 (MPa)	2.72	2.00	1.16	2.54
1	Loading point (kN)	86	63	38	81
	C.O.V. (%)	1	6	11	10
	Effective stiffness (1) (kN/mm)	57	42	24	41
	C.O.V. (%)	6	2	31	16
2	Loading point (kN)	90	64	40	82
	C.O.V. (%)	6	6	13	10
	Effective stiffness (2) (kN/mm)	39	40	18	33
	C.O.V. (%)	51	5%	53%	1%
3	Loading point (kN)	92	-†††	84	92
	C.O.V. (%)	-††	-†††	10	-††
	Effective stiffness (3) (kN/mm)	32	-†††	23	32
	C.O.V. (%)	-††	-†††	13	-††

††calculated on the bases of the results for two specimens only
†††energy requirement were not satisfied

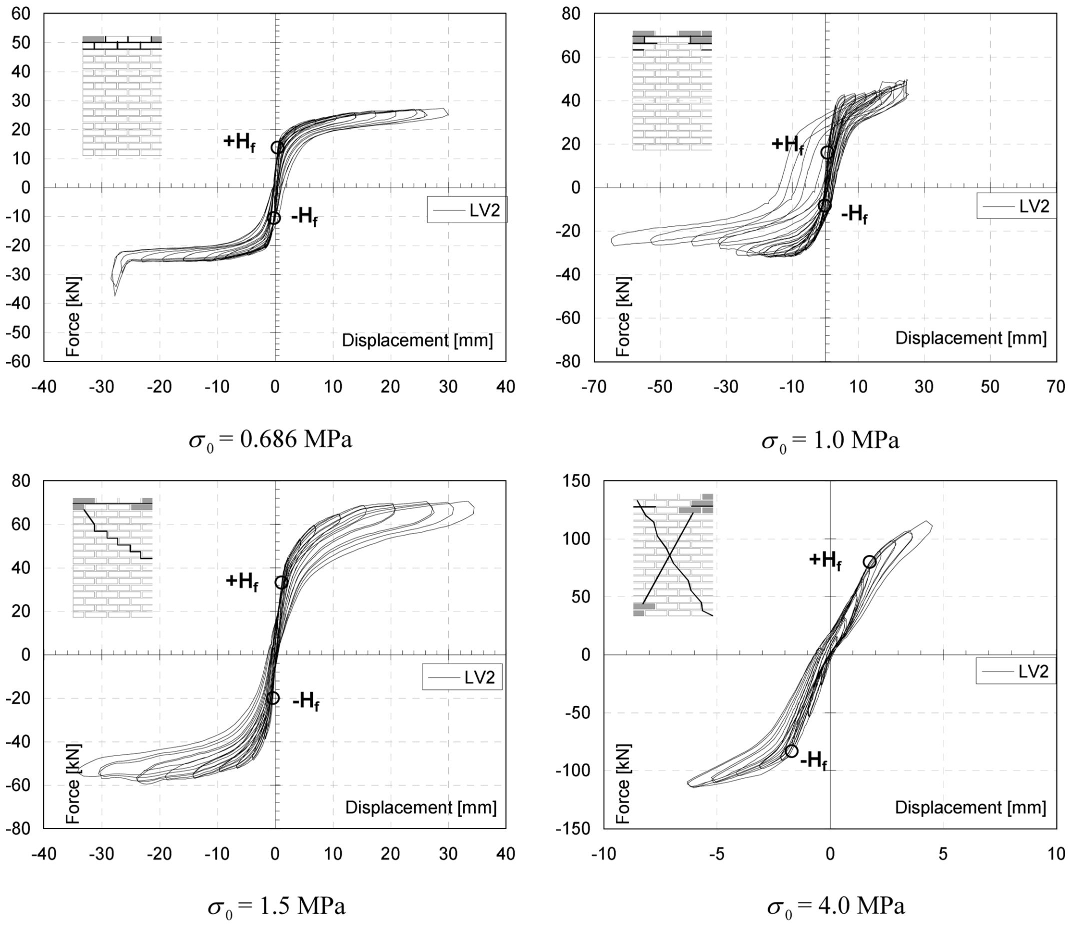


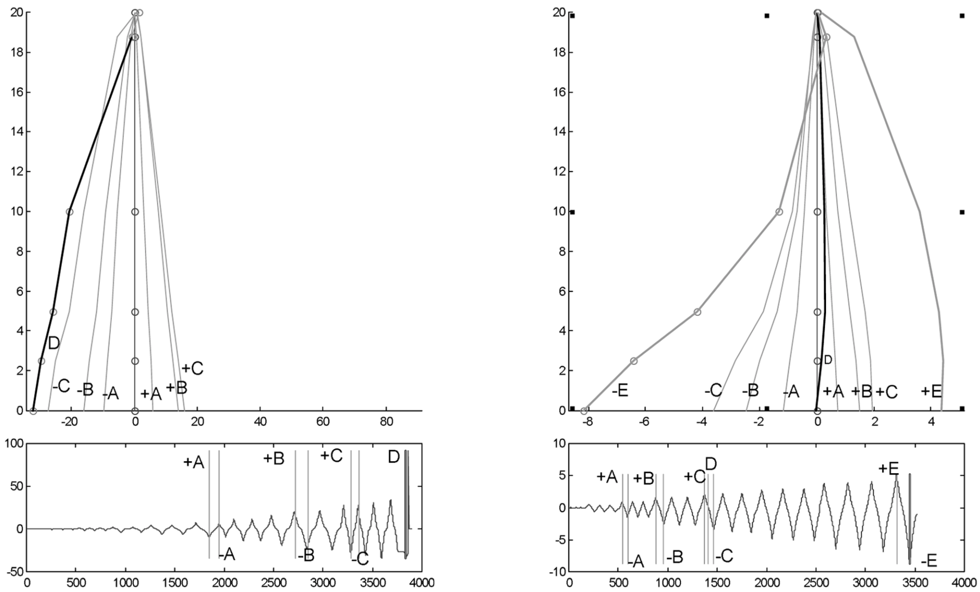
Fig. 16 Typical experimental lateral displacement - lateral resistance hysteresis loops for CLM at different level of precompression

edge of the specimen and the opening of the flexural cracks through the length of the specimen, the failure of the specimen was due to sliding (σ_0 = 0.686 MPa) with shear-friction resistance in the plastic range. For higher level of precompression the beginning of typical rocking behaviour has been observed with crushed masonry in the upper corners of the specimen (σ_0 = 1.0 MPa). The failure of the specimen in the plastic range appeared to be a combined rocking-sliding failure. The first shear crack was developed at the precompression of 1.5 MPa, due to the crushing masonry in the upper corner of the specimen. At the highest level of precompression (σ_0 = 4.0 MPa) a sudden fragile shear failure was observed, with an almost simultaneous opening of the shear crack and crushing of the masonry in the upper and lower corners of the specimen.

From the results of testing at different levels of precompression which are presented in Table 4 and are consistent with the results of some other authors (Shing *et al*. 1991), it is shown that the level of precompression has a significant influence in the determination of the effective stiffness of the masonry wall. When comparing the influence of different types of mortar (Table 3) versus level of precompression at which the stiffness has been determined (Table 4), then at the same relative

Table 4 Results of shear tests at the different levels of precompression

Criterion	σ_0 (MPa)	0.686	1.0	1.5	4.0
	No. of spec.	1	1	1	2
1	Effective cracking load (kN)	24	43	65	103
	C.O.V. (%)	-	-	-	2
	Effective stiffness (1) (kN/mm)	26	21	31	61
	C.O.V. (%)	-	-	-	2
2	Effective cracking load (kN)	24	-	66	107
	C.O.V. (%)	-	-	-	1%
	Effective stiffness (2) (kN/mm)	21		26	50
	C.O.V. (%)	-	-	-	2



(a) lowest level of precompression (b) highest level of precompression

Fig. 17 Deformed shapes of the specimens for the lowest and the highest level of precompression

level of precompression the failure mechanisms of walls made from different types of masonry were similar. However, for one type of masonry at different absolute levels of precompression both, the mechanisms of failure and the effective stiffness were different.

Furthermore, the different types of deformation modes of specimens were observed under different absolute levels of precompression. Some modes of deformation of specimens recorded with the measurement devices LV1-6 for the lowest (σ_0 = 0.686 MPa) and the highest (σ_0 = 4.0 MPa) level of precompression are presented in Fig. 17.

The load-time history is presented in the lower part of the Fig. 17 while in the upper part recorded modes of deformations at different stages of loading are shown. For the lowest level of

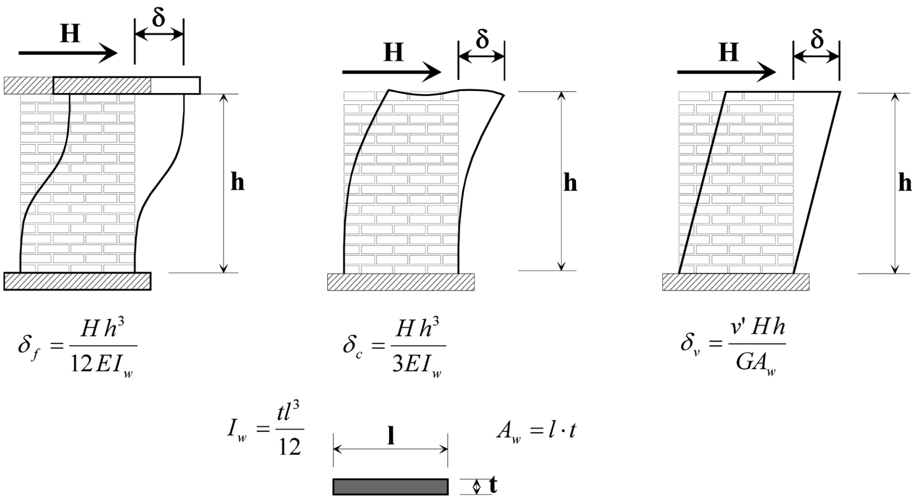


Fig. 18 Elastic deflections of masonry walls

precompression typical shear behaviour (Fig. 17a) was observed. After the formation of the flexural cracks (point B), the mode of deformation remained unchanged and symmetrical about the reference line. It remained symmetrical until the start of rocking and crushing of units at the upper corners of the specimens (point C) which provoked the formation of a plastic hinge in the upper part of the specimens and further sliding failure (point D) of the specimen. For the highest level of precompression typical flexural behaviour was observed (Fig. 17b). The formation of the shear cracks (point A) and the softening of the masonry in the mid-height of the specimen (point B) were two major reasons for distinguished asymmetric behaviour of the specimen (point C) and some residual deformations (point D). Due to the further propagation of the shear cracks and beginning of rocking the final shape of the specimen is typical for flexural behaviour (point E).

Stiffness of structural elements such are masonry walls is an important parameter which determine the overall dynamic behaviour of the structure and distribution of the seismic forces. The stiffness of the structural element according to the theory of elasticity depends on the mechanical properties, geometry of the element, boundary restraints and is defined by the action that causes a unit displacement. For in-plane laterally loaded masonry elements the unit displacements/deflections are presented in following Fig. 18. The coefficient υ' is the shear deformation coefficient which for the rectangular cross-section is equal to 1.2.

In seismic analysis of laterally loaded walls one of the assumptions is that the resulting deformation δ_Σ is a result of the combined action of bending and shear. Thus for walls with both ends fixed, the δ_Σ is equal to sum of δ_f and δ_v, and for cantilever walls $\delta_\Sigma = \delta_c + \delta_v$. The elastic stiffness is taken as $K_e = l/\delta_\Sigma$ Introducing the coefficient k' which describes the applied restraint conditions of the element (for both ends fixed is equal to 0.83 and for cantilever walls $k' = 3.33$), the elastic stiffness of the wall can be calculated as:

$$K_e = \frac{GA_w}{1.2h\left[1 + k'\frac{G}{E}\left(\frac{h}{l}\right)^2\right]} \tag{12}$$

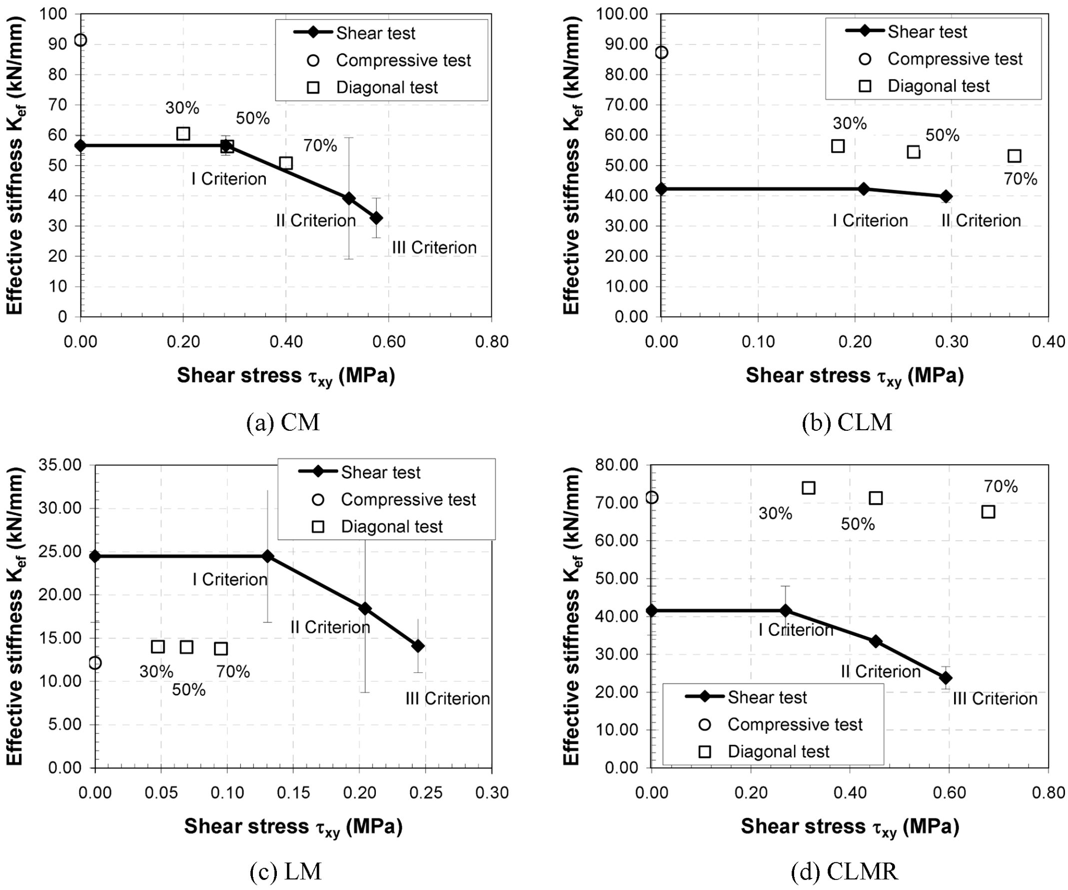
It can be seen from Eq. (12) that knowing the geometry and boundary conditions of masonry structural element it is necessary to determine mechanical properties of masonry such are G and E in order to calculate the design elastic stiffness.

An alternative form for a stiffness equation for the lateral stiffness of cantilevered shear walls is the standard FEMA 356 document (Federal Emergency Management Agency 2000):

$$k = \frac{1}{\dfrac{h_{eff}^3}{3E_m I_g} + \dfrac{h_{eff}}{A_v G_m}} \tag{13}$$

Where h_{eff} is the effective wall height, A_v is the shear area, I_g is the uncracked moment of inertia, E_m is the masonry elastic modulus, and G_m is the masonry shear modulus.

Following the results from our tests, the elastic modulus was determined from confined compressive tests. The shear modulus was determined from compressive (G_c) and diagonal tests (G_d). These values were used to calculate effective stiffness. Then analytically calculated elastic stiffness was compared (Fig. 19) to the effective stiffness obtained experimentally.



(a) CM (b) CLM

(c) LM (d) CLMR

Fig. 19 K_{ef} and K_e vs. shear stresses for different types of mortar at the same relative level of precompression

The results of the comparisons undeniably show that the shear modulus of the masonry obtained from compressive tests is not appropriate in prediction of the stiffness of the laterally loaded masonry structural elements. On the other hand, the shear modulus obtained from diagonal tests with big cross can be used for the evaluation of the stiffness of masonry elements made from stiff mortars (CM and CLM). This is not valid for reinforced masonry (CLMR), where once again we have to confirm unsuitability of the diagonal tests for the evaluation of the both mechanical and stiffness parameters for reinforced masonry.

As for the weak mortar (LM) results, both compressive and diagonal tests are not suitable for prediction of the stiffness of the laterally loaded masonry elements and for the LM specimens the absolute levels of shear stresses at which the stiffness was calculated from the results of diagonal testing were much lower in comparison to the results of shear testing.

The results of testing the CLM specimens under different levels of precompression, which are presented in Fig. 20, clearly show the influence of the level of precompression on the evaluated effective stiffness of the unreinforced masonry elements. This phenomena has been also reported and discussed by other authors (Drysdale *et al.* 1994, Shing *et al.* 1991) for reinforced masonry and by Sinha and Hendry (Mayes *et al.* 1975) for unreinforced masonry. While for different types of masonry under the same relative level of precompression the failure modes of the structural elements were the same, for CLM specimens under the different absolute level of precompression both the failure modes as well as the experimentally gained stiffness were different. The use of the shear modulus derived from diagonal tests in Eq. (12), produces the elastic stiffness K_e which is a rough approximation for the effective stiffness K_{ef} of laterally loaded masonry element. This can be easily understood since both diagonal tests and the shear tests under the highest levels of precompression (2.0 and 4.0 MPa) caused the same type of diagonal tensile failure of the specimens.

The stiffness of the elements is a function of the shear stresses. According to our results, it is obvious that the shape of the stiffness degradation curve for cantilever elements depends on the level of precompression (Fig. 20). It is also clear that in the inelastic analysis of the masonry elements, where large displacements and shear stresses are likely to occur, it is more appropriate to consider the effective stiffness to be constant up to the point of first limit state.

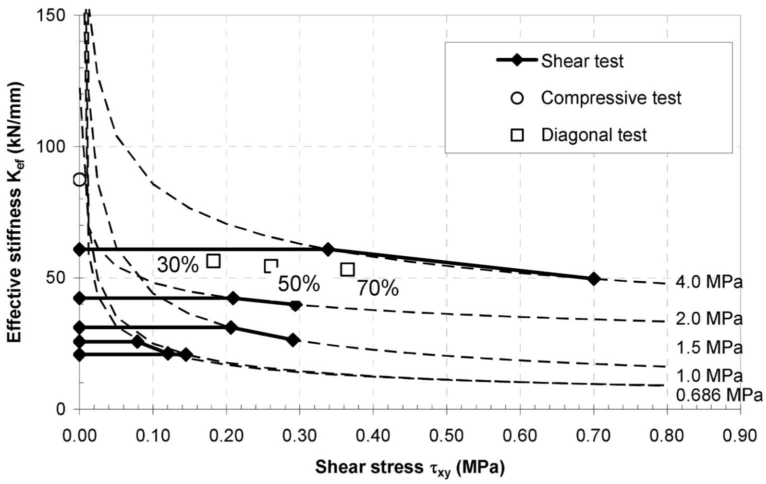


Fig. 20 Stiffness degradation vs. shear stresses for different levels of precompression

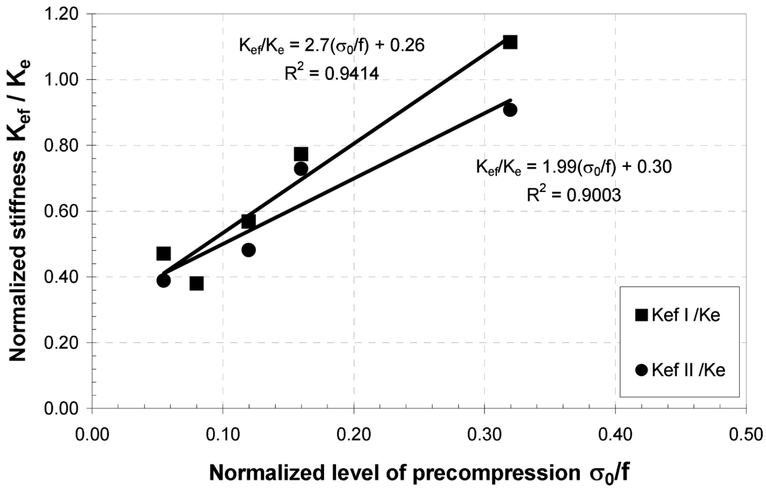


Fig. 21 Dependence of the stiffness of cantilever masonry walls due to applied level of precompression

The effective stiffness is a complex parameter associated with all restoring forces of structural system and depends upon several factors including: ductility, different failure mechanisms and for seismic design also dissipation of energy. Unlike K_{ef} the elastic stiffness K_e is a solid parameter which depends on the geometry of the wall, mechanical parameters of the masonry (G and E) and boundary restraints. It is possible to correlate those two parameters using Eq. (12), the shear modulus obtained from diagonal tests and considering an appropriate level of precompression. When predicting the actual effective stiffness K_{ef} from the elastic stiffness K_e and considering the level of precompression for two criteria used to evaluate effective stiffness, two empirical formulas have been suggested (Fig. 21).

4. Analysis

Shear modulus of the masonry gained through compressive tests showed good correlation with the results of other authors as well as with the provisions of different national codes for the masonry. For stiffer mortar mixtures it also good correlates with the results for shear modulus gained through diagonal and dynamic shear tests. Note also that for all those three types of tests the deformations of the masonry were measured on relatively small area of the specimens. However it clearly showed inefficiency in the prediction of the effective stiffness K_{ef} of the laterally loaded masonry structural elements. While for the stiff mixtures it overestimate the experimentally gained effective stiffness, for masonry made from weak mortars it underestimate experimentally gained values. The latter conclusion was mainly attributed to the strong anisotropic behaviour of the masonry which was made from weak mortars.

Both different types of mortar/masonry as well as different levels of precompression strongly influence the effective stiffness of masonry elements. While for different types of masonry the failure mechanisms under the same relative level of precompression remain the same, the effective stiffness as well as the overall resistance of the walls depends on the type of the masonry. On the other hand for the one type of masonry, different levels of precompression provoke different failure mechanisms as well as different overall resistance of the masonry walls and thus different effective stiffness.

In order to obtain shear modulus for the prediction of effective stiffness of the masonry walls by simple testing procedure, the most efficient were diagonal tests under monotonic loading. By using the results for the shear modulus gained through diagonal tests into the equation for calculation of the elastic stiffness we have approached the experimentally gained effective stiffness for masonry made from stiff mortar mixtures fairly well. On the other hand diagonal tests showed unsuitability in obtainment of mechanical and stiffness parameters for reinforced masonry.

Following the results of tests on laterally loaded masonry walls and calculating shear modulus from the effective stiffness (Eq. (12)) Tomaževic (1999), recommends different ratio for shear modulus G against elastic modulus E as a value between 6% to 25%. For cantilever walls this equation for calculation of the shear modulus from effective stiffness is derived in Eq. (14).

$$G_v = \frac{K_{ef}}{\frac{A}{1.2h} - \frac{4K_{ef}}{1.2E}\left(\frac{h}{l}\right)^2} \tag{14}$$

Implementing into Eq. (14) the results for K_{ef} gained from the shear tests of masonry made from stiffer mortar mixtures under higher levels of precompression it is observed that this ratio for shear modulus G vs. elastic modulus E is a realistic estimate. However, according to the test results and from Eq. (14), it is observed that this second approach has two major deficiencies: The second approach does not consider the influence of the level of precompression and for slender walls and for masonry with low modulus of elasticity (that could be expected for historic masonry) it gives unrealistic values for the shear modulus. The latter represent a realistic case for the masonry made from weak mortar with strong anisotropic behaviour. Some recent results from ISPRA and University of Pavia (Magenes and Calvi 1997), confirm our results and emphasize the inconsistency of the results for the deformation and stiffness properties for the masonry made from weak (lime) mortars.

5. Conclusions

This paper aimed at re-evaluating the values for the shear modulus stated in many national codes considering different experimental techniques for its determination. Analysis of the results of an extensive testing programme allows drawing following conclusions:

1. Present provision for the shear modulus of the masonry (G = 0.4E) stated in many national codes as well as in new Eurocode 6 is correct if the compressive loading is predominant one. It also correlate well with the results of measurement of deformations of masonry as a construction material (on a small area of the specimens) gained through diagonal (small cross) and dynamic shear tests. However it can largely overestimate the stiffness of masonry structural elements.
2. For brickwork masonry made with stiffer mortar mixtures the results of simple diagonal tests can be effective for calculating the effective stiffness of masonry elements.
3. Diagonal tests are unsuitable for evaluating both mechanical and stiffness properties for reinforced masonry.

4. Due to the strong anisotropic behaviour the stiffness of the brickwork made with weak mortars (historic masonry) should be evaluated solely through shear tests of the specimens of the size of structural elements.

Since the effective stiffness gained from shear tests of cantilever walls can depend from the level of precompression, the determination of the shear modulus from that type of testing remains questionable and in that sense more harmonized approach in determination of the overall shear resistance of masonry structural elements should be defined.

Acknowledgements

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