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TESTS OF VERTICALLY SHEARED CLAY BRICK MASONRY WALLS WITH AND WITHOUT BED JOINT REINFORCEMENT

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Abstract

Usually, stiffening (shear) walls of masonry buildings subjected to irregular settlements are built as unreinforced structures. Sometimes this type of masonry has not sufficient mechanical properties. Therefore, it is necessary to look for a method to improve them. One of such methods can use bed joint reinforcement.

The preliminary comparative tests of vertically sheared unreinforced and reinforced clay brick masonry walls carried out in Silesian University of Technology is presented in this paper. The influence of usage of bed joint reinforcement on behaviour, load-bearing capacity, failure model and stiffness such loaded masonry walls was determined. Moreover, the positive influence of used type of bed joint reinforcement on mechanical properties and a significant reducing of crack width were observed.

Key Words

Masonry structures, stiffening walls, vertical shearing, bed joint reinforcement.

1 Introduction

Analysis of masonry walls of buildings situated on terrains subjected to differential settlements is one of main research and theoretical tasks in Poland. This is why a large number of tests of unreinforced masonry walls made of different types of masonry units and sheared in vertical direction were carried out in Department of Building Structures and Bridges of Silesian University of Technology during recent fifteen years. The results of these investigations were presented during several recent years, also by author, at international conferences, e.g. Kubica (1995 and 1998). Synthesis and critical analysis of all earlier tests of unreinforced masonry walls are presented by Kubica (2003). One of the methods of improving masonry mechanical properties is

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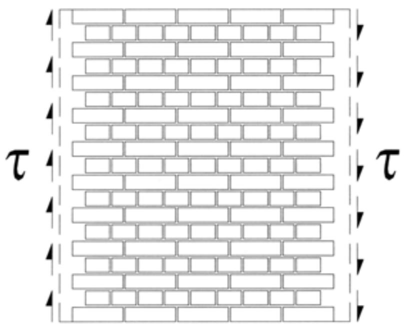
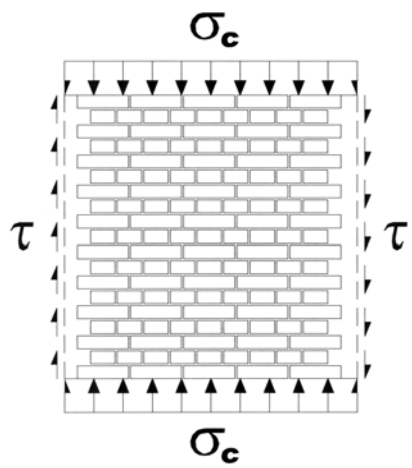
using bed joint reinforcement. It is not so simple to find any detailed information about influence of usage of bed joint reinforcement on behaviour and mechanical parameters of walls subjected to shearing in direction perpendicular to bed joints. At the Silesian University of Technology the first investigations of vertically sheared masonry walls reinforced in bed joints (using smooth bars) were carried out several years ago – see Piekarczyk at al (2002). It is much easier to find information concerning strengthening of existing cracked (damaged) masonry walls using bed joint reinforcement – e.g. Cook at al (1995) or with Fibre Composites – Schwegler (1995). But there is *post factum* action, not so useful for design practice. Therefore we decided to make investigations (the first ones in Poland) of vertically sheared masonry walls with bed joint reinforcement that conform to EN 845-3 (2003) regulations, which is provided to be applied during erection of new buildings (Kubica et al 2002).

2 Brief description of investigations

As a main aim of presented investigations we accepted to obtain as wide as possible information about behaviour and mechanical properties of vertically reinforced masonry walls sheared with and without precompression. Therefore, from all 8 tested models 6 were reinforced and only 2 specimens were unreinforced (one sheared without precompression and another one with compressive stresses σ_c = 0.9 N/mm^2. Generally, the tests programme of main investigations is shown in Table 1 below.

Table 1. Programme of main tests

Shape of test specimen and manner of loading	σ_c [N/mm^2]	**Designation of test models**	**Remarks**
	0	MZ-00/1 MZ-00/2 MN-00/1	models only sheared (2 reinforced and 1 unreinforced)
	0.30	MZ-03/1	models sheared with precompression
	0,60	MZ-06/1	
	0.90	MZ-09/1 MN-09/1	models sheared with precompression (1 reinforced and 1 unreinforced)
	1.20	MZ-12/1	models sheared with precompression

All models were subjected to shearing in vertical direction. Three of them were only sheared (without precompression). The remaining five specimens were sheared with different levels of vertical precompression σ_c – as it is shown in Table 1. Simultaneously carrying out tests of walls subjected exclusively to shearing loads as well as the same walls subjected also to shearing but with previously prestressed parts of brickwork, is the correct action. In real situation every masonry load-bearing wall, especially of first storey of building, is subjected also to compressive stresses (created by deadweight loads of higher storeys).
All masonry specimens were built using the same solid clay bricks (class "15" – which corresponds to guarantied by producer compressive strength f_b = 15,0 N/mm^2) and cement-lime mortar class M5 (1 : 1 : 6). Brickwork models (all parts of masonry walls) were made of prefabricated forms manufactured beyond test stand.
Each test specimen had the same shape and overall dimensions – as shown in Fig.1a. Thickness of all models was also constant, i.e. 250 mm. The shape and overall dimensions of masonry walls were determined in earlier research works. This shape of models guarantees undisturbed state of stress and strains inside measurement area of each wall (central part of the wall – see Fig. 1b)

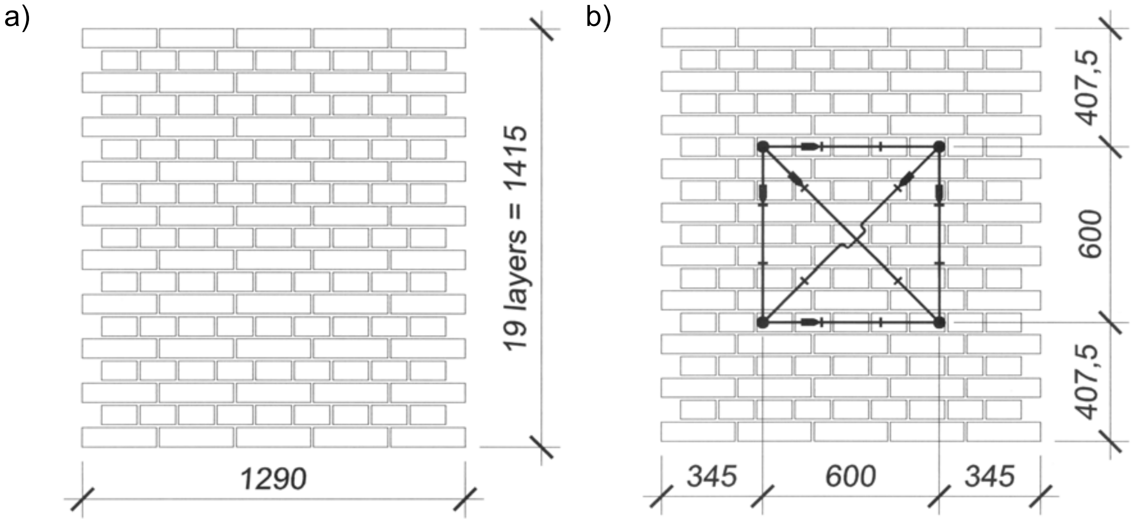


Fig.1 Test specimens: a) - shape and overall dimensions; b) - frame measure arrangement

All reinforced specimens had horizontal truss type reinforcement – according to EN 845-3 (2003) located into five bed joints – see Fig. 2. The main longitudinal steel bars had 5 mm diameter.
The general view of the test stand for investigation of models with prestressed parts of walls is shown in Fig. 2. The principal elements of the stand are two external steel pillars, stiff attached with screws to the floor of the laboratory. Between both external pillars the third steel column is located, which is connected with two horizontal bowstrings to both external pillars. The shearing load “P” was applied by pushing up (using hydraulic jack) the internal steel column. Bowstrings (marked in Fig. 2 by mean of "S" forces) prevented the horizontal movements of upper and bottom parts of the external pillars. The constant value of compressive stresses (σ_c) was provided by using compensating springs mounted on the top of each pair of vertical stiff steel strings (45 mm diameter).
Three (two reinforced and one unreinforded) models that subjected only to shear loads without precompression were also tested in a stand such as presented in Fig. 2, but without prestressing equipment.

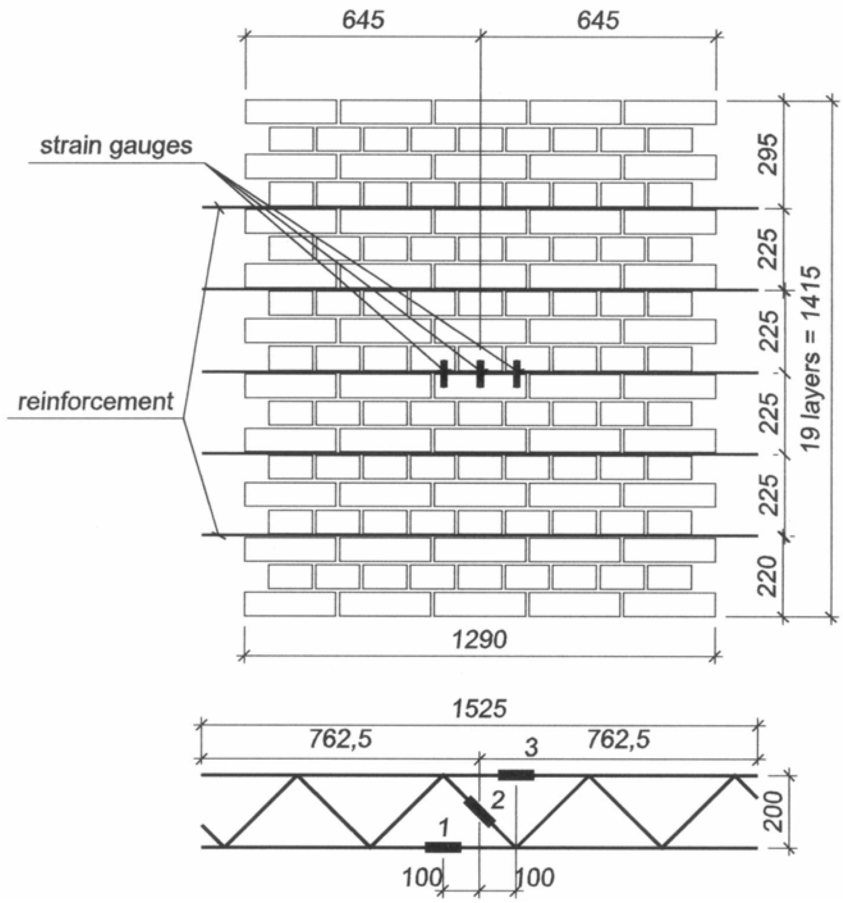


Fig. 2 Reinforcement location in five bed joints of each reinforced masonry wall

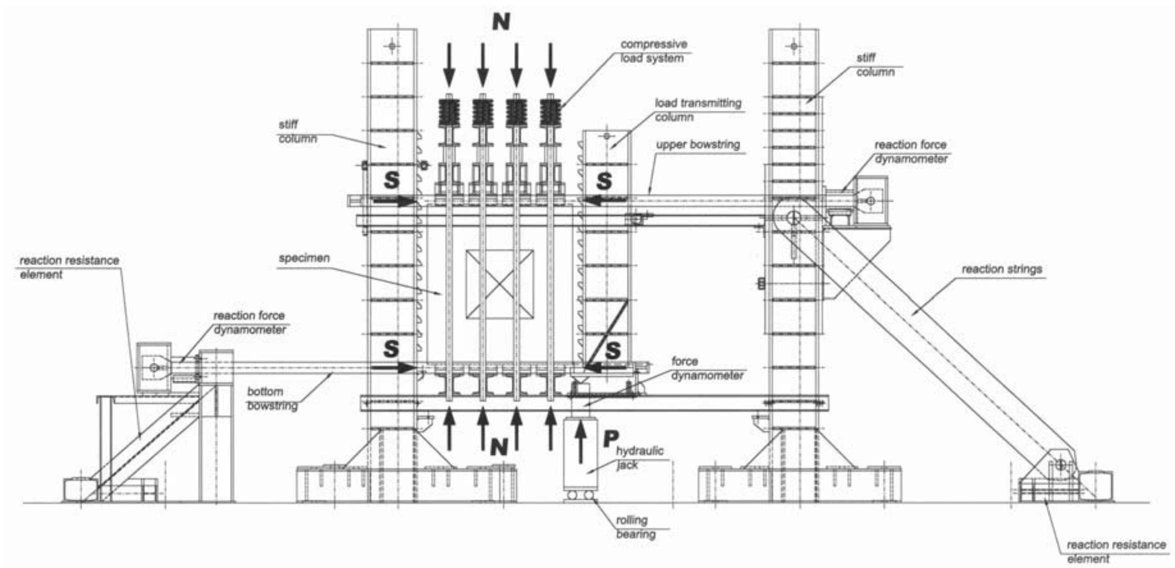


Fig. 3 General view of the test stands for investigation of models with prestressed part of wall

The values of pushing force (designated in Fig. 2 as „P") and forces in vertical tension members ("N" forces in steel strings) were measured using adequate electro resistance dynamometers.

All models were subjected to temporary loads, within the range of one cycle, while performing the current checking, both the forces, which pushed the middle pillar and forces in prestressing system. The main loading force "P" was increased with gradation ΔP = 10 kN in every 5 to 10 minutes.

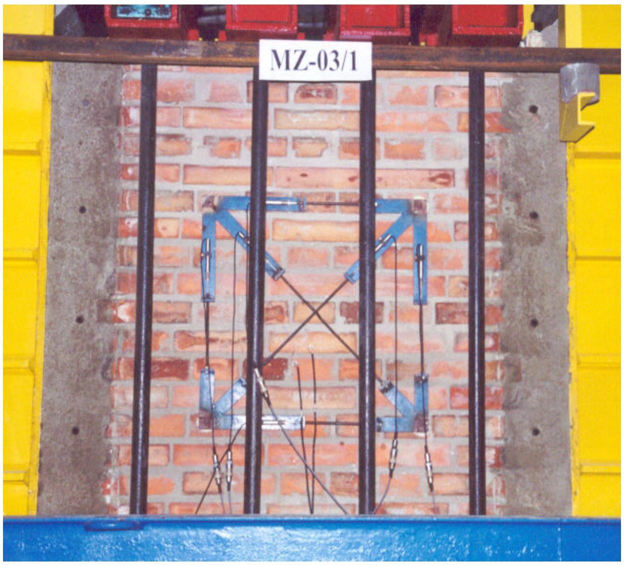


Fig.4 View of central area of test model with completed measurement equipment

The deformations of each test specimen were measured using the square system of displacement measuring devices presented in Fig. 1b., whereas the view of central area of test model with used complete measurement equipment is shown in Fig. 4.
The inductive indicators (with accuracy of 0,0002 mm) for measuring the displacement were used.
All these indicators were connected to special automatic registration apparatus.
Moreover, the values of strains were registered at all load levels by means of strain gauges placed on bars of bed joint reinforcement (see strain gauges location shown in Fig. 2).

3 Results of tested models with bed joint reinforcement

According to expectation a similar failure of sheared masonry walls with and without precompression was observed. The shape of failure of all test models was one or several diagonal cracks. As a criterion of failure – from the not satisfying serviceability limit state point of view – the situation was taken into consideration when the first diagonal crack had appeared. Presented investigations have proved that the appearance of the first crack in reinforced models was not equal to physical destruction of the wall – like it was observed in case of unreinforced masonry walls. It means the cracking load is not equal with the ultimate load. After appearance of first cracks, the load level was increased until destruction of the model was achieved. In phase of destruction, the failure had form of one wide crack (especially in sheared models without precompression, i.e. $\sigma_c = 0$ – see Fig. 5a) or many cracks parallel to each other of different widths (e.g. shown in Fig. 5b wall sheared with precompression – the maximal level of compressive stresses $\sigma_c = 1{,}2\ \text{N/mm}^2$).
The essential results of investigations are compared in Table 2. The values of scratching shear stresses (τ_{cr}) and the ultimate shear stresses (τ_u) were determined for each tested specimen.
In case of models sheared without precompression the shear stresses (τ_{cr}) and (τ_u) were calculated as average values of both tested specimens. The ultimate values of the non-dilatational strain angles, corresponding to (τ_{cr}) and (τ_u) stresses are also presented in adequate columns in Table 2.
The relation between shearing stresses corresponding to cracking (τ_{cr}) and ultimate shear stresses (τ_u) are presented in the last column of the above mentioned Table. The comparison of values shows that it is difficult to determine on the basis of these results if differences between (τ_{cr}) and (τ_u) stresses are greater for models sheared with precompression than for models being only sheared. Practically, for all tested

specimens ultimate shear stresses were greater by ca. 20% than stresses corresponding to the first crack appearance. Of course, these investigations covered only 6 reinforced specimens. Only one model was tested for each level of compressive stresses σ_c, except for two models sheared without precompression. From quantitative point of view more reliable analysis will be available after performing a series of additional tests of test models.

a) b)

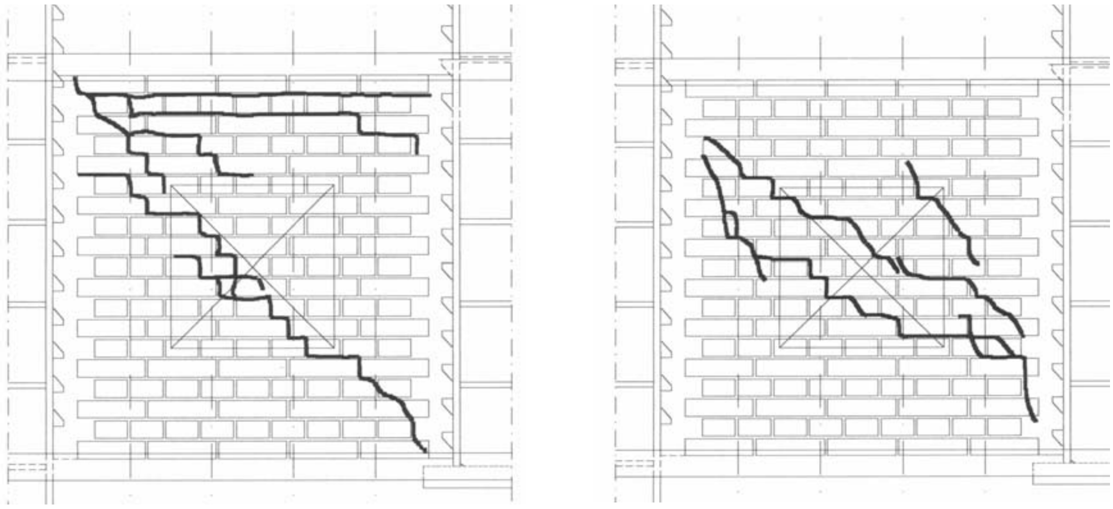


Fig. 5 Crack pattern of both surfaces for model: a) MZ-00/1 – vertically sheared without precompression (σ_c = 0); b) MZ-12/1 – vertically sheared with maximal level of precompression (σ_c = 1,2 N/mm^2)

Table 2: Main results of investigations

Model	σ_c [N/mm^2]	$\tau_{cr,i}$ [N/mm^2]	$\Theta_{cr,i}$ [mm/m]	$\tau_{u,i}$ [N/mm^2]	$\Theta_{u,i}$ [mm/m]	$n = \frac{\tau_{u,i}}{\tau_{cr,i}}$
MZ-00/1 / **MZ-00/2**	0	$\frac{0.57}{0.40}$ 0.49	$\frac{0.30}{0.25}$ 0.28	$\frac{0.68}{0.45}$ 0.57	$\frac{0.55}{0.32}$ 0.44	**1.16**
MZ-03/1	0.30	0.79	0.83	0.96	4.69	**1.21**
MZ-06/1	0.60	0.62	0.47	0.85	2.51	**1.37**
MZ-09/1	0.90	0.99	1.03	1.19	3.38	**1.20**
MZ-12/1	1.20	0.96	1.65	1.19	4.66	**1.14**

Fig. 6 presents comparison of shear stresses (τ_i) – non-dilatational strain angles (Θ_i) relationships determined for all tested models and shear stresses within the range of $0 \div \tau_{u,i}$. No differences are observed up to shear stresses level approximately equal ($\tau_{cr,i}$) – the first crack appearance. After occurrence of cracking, the character of analysed relationships is significantly connected with level of precompression. For models with the highest values of σ_c stresses (MZ-09/1 and MZ-12/1) these relationships are smooth –

quite different like for models with lower levels of precompression, where an effect of sliding into the cracks was observed .

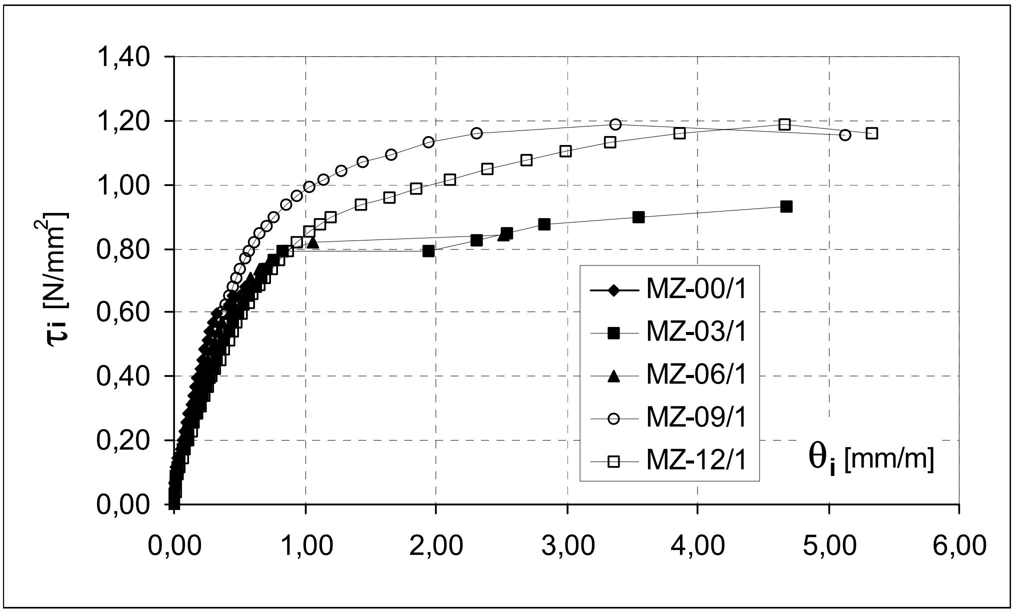


Fig. 6 Shear stresses (τ_i) – non-dilatational strain angles (Θ_i) relationships obtained for reinforced models

Additionally, the measurement of steel strains (ε_i) of longitudinal and skew rods of the used reinforcement was carried out. The *(τ_i) – (ε_i)* relationships obtained for longitudinal steel rods are shown in Fig. 7.

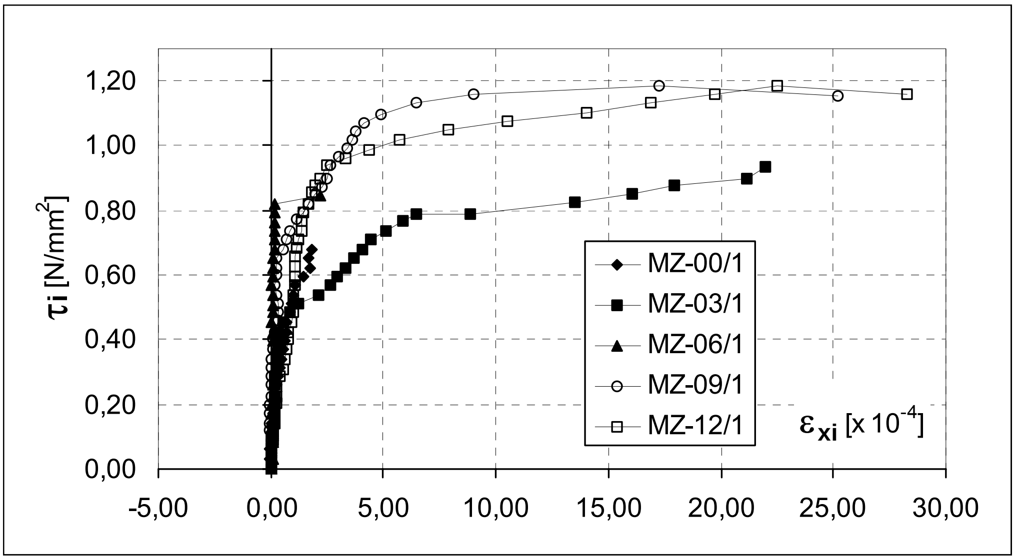


Fig. 7 Shear stresses (τ_i) – steel strains (ε_i) relationships for longitudinal rods of reinforcement

Practically, up to the first diagonal cracks appearance the values of steel strains were very small and well corresponded with horizontal deformations of masonry specimens. Situation has changed after cracking. Strains and stresses in longitudinal rods connected with them were growing up. In effect, the crack width was reduced.

4 Comparison of results obtained for all tested models

The comparison of results obtained from investigations of masonry walls with truss type reinforcement into five bed joints subjected to vertical shearing with results of loaded in this same way models without any reinforcement is given in Table 3. The values of shear stresses τ_i and non-dilatational strain angles Θ_i are compared.

Table 3 Results of comparative tests

Values obtained for level of first cracks appearance						
σ_c [N/mm^2]	Unreinforced models $\tau_{cr,i-N}$ [N/mm^2]	$\Theta_{cr,i-N}$ [mm/m]	Reinforced models $\tau_{cr,i-Z}$ [N/mm^2]	$\Theta_{cr,i-Z}$ [mm/m]	$n = \frac{\tau_{cr,i-Z}}{\tau_{cr,i-N}}$	$n = \frac{\Theta_{cr,i-Z}}{\Theta_{cr,i-N}}$
0	0.49	0.31	0.59	0.32	**1.20**	**1.03**
0.90	0.99	0.78	1.09	1.03	**1.10**	**1.38**
Ultimate values (failure level)						
σ_c [N/mm^2]	Unreinforced models $\tau_{u,i}$ [N/mm^2]	$\Theta_{u,i}$ [mm/m]	Reinforced models $\tau_{u,i}$ [N/mm^2]	$\Theta_{u,i}$ [mm/m]	$n = \frac{\tau_{u,i-Z}}{\tau_{u,i-N}}$	$n = \frac{\Theta_{u,i-Z}}{\Theta_{u,i-N}}$
0	0.57	0.45	0.68	0.55	**1.19**	**1.22**
0.90	0.95	1.39	1.19	3.38	**1.09**	**2.43**

The graphic comparison of non-dilatational strain angles (Θ_i) changes in co-relation with shear stresses (τ_i) level for specimens sheared without precompression is presented in Fig. 8.

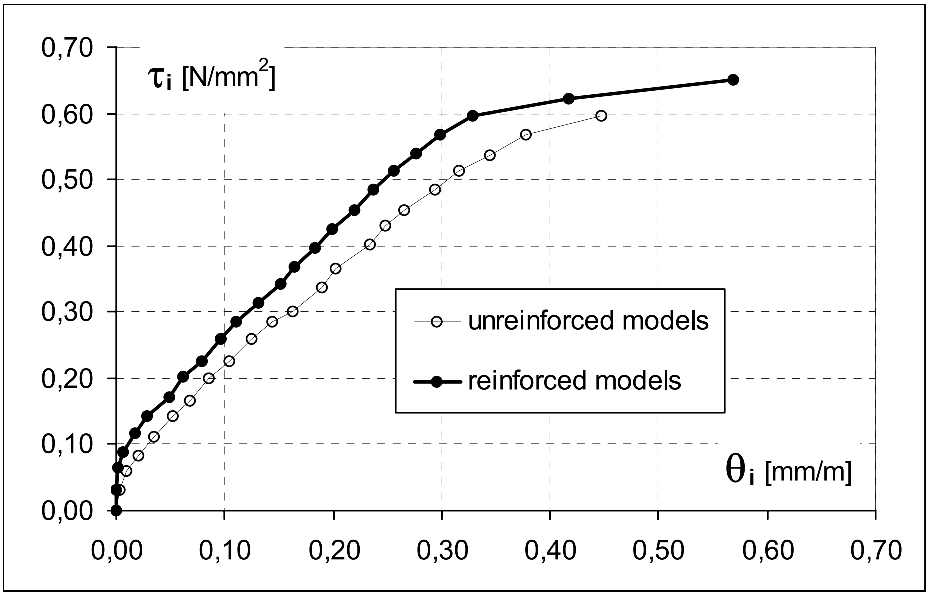


Fig. 8 Shear stress (τ_i) – non-dilatational strain angle (Θ_i) relationships for models only subjected to vertical shearing (σ_c = 0)

The same graphs for walls simultaneously sheared and compressed in vertical direction are shown in Fig. 9.

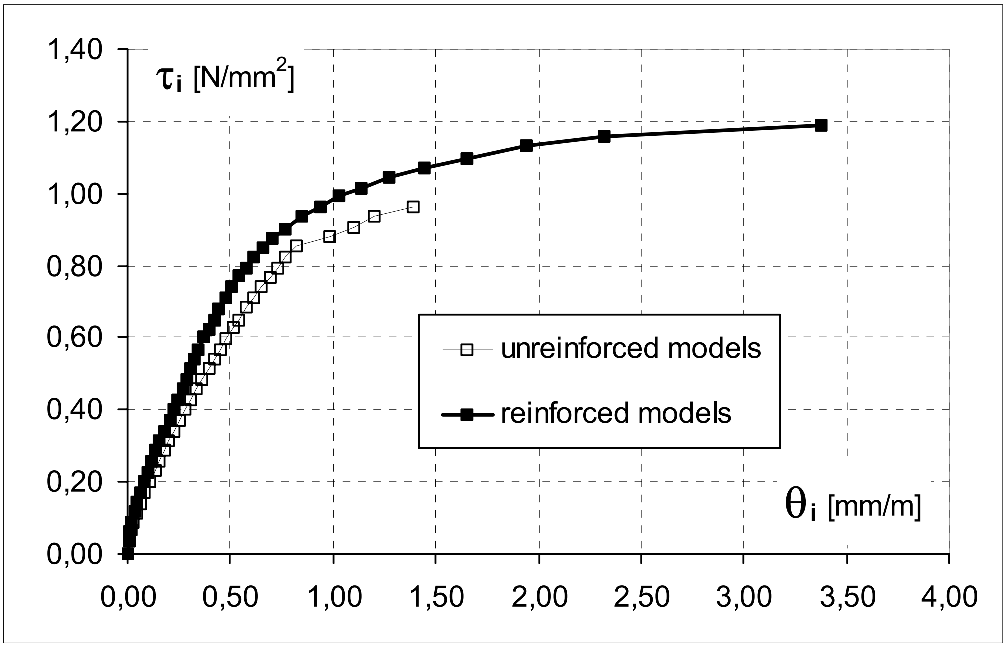


Fig. 9 Shear stress (τ_i) – non-dilatational strain angle (Θ_i) relationships for models subjected to vertical shearing with precompression (σ_c = 0,9 N/mm^2)

The analysis of values given in Table 3 and *(τ_i) – (Θ_i)* relationships from Fig. 8 and from Fig. 9 shows a significant positive influence of bed joint reinforcement used on tested parameters. Especially positive effect was observed in case of state of failure.
Generally, models with reinforcement demonstrated higher values of shear stresses ($\tau_{cr,i}$) that corresponded with the first crack appearance – ca. 20% for elements sheared without precompression (σ_c = 0) and about 10% for models simultaneously sheared and compressed (compressive stresses level σ_c = 0,9 N/mm^2). Ultimate values of shear stresses ($\tau_{u,i}$) for both types of tested specimens were quite similar as determined for the state of the first cracks appearance.

More significant differences can only be seen while comparing the non-dilatational strain angles. For the state of the first crack appearance, the difference between values determined for reinforced and unreinforced elements is considerably lower than values obtained in case of state of failure (especially for walls sheared with precompression level σ_c = 0.9 N/mm^2, where ($\Theta_{u,i}$) values for reinforced specimen exceeded twice the determined ones for unreinforced model).
Moreover, investigations carried out allowed to state, that in case of reinforced masonry elements the first crack width did not exceed 0,10 ÷ 0,15 mm. Whereas crack width recorded in tests of unreinforced models was much greater – over 0.3 mm.

4 Summary

Presented tests of masonry walls with truss type bed joint reinforcement should be analysed mainly from qualitative point of view. Among other things, it was proved that using reinforcement into bed joints gives positive influence on the behaviour of masonry walls sheared vertically. This influence is more significant on ultimate ($\Theta_{u,i}$) and cracking ($\Theta_{cr,i}$) values of non-dilatational strain angles than on ultimate ($\tau_{u,i}$) and cracking ($\tau_{cr,i}$) values of shear stresses.

According to results of presented investigations, the experimental (τ_l) - (Θ_l) relationships are not linear.
Up to first diagonal cracks appearance the values of steel strains (ε_i) were very small, but after cracking strains and connected with them stresses in longitudinal rods they were growing up. In opposite, skew rods (zigzag bars) did not have significant strains.
Moreover, the reducing influence (even up to 50%) of using reinforcement on width of first cracks was observed.

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