TECHNICAL TRANSACTIONS | CZASOPISMO TECHNICZNE
CIVIL ENGINEERING | BUDOWNICTWO
1-B/2013

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BEHAVIOUR OF UNREINFORCED AND REINFORCED MASONRY WALLETTES MADE OF ACC BLOCKS SUBJECTED TO DIAGONAL COMPRESION

ZACHOWANIE SIĘ NIEZBROJONYCH I ZBROJONYCH MUROWANYCH ELEMENTÓW WYKONANYCH Z BLOCZKÓW Z AUTOKLAWIZOWANEGO BETONU KOMÓRKOWEGO PODDANYCH UKOŚNEMU ŚCISKANIU

Abstract

For more than 10 years, masonry units made of ACC turned out to be one of the most popular materials used in the construction of residential buildings in Poland and other European countries. The recommended technology for erecting such structures requires using thin bed joints and non-filled head joints. Unfortunately, masonry buildings erected using this technique are prone to damage and cracks. One of the methods to guarantee improvement of cracking resistance of this type of structure is using a special prefabricated reinforcement to be placed in bed joints. The main objective of the investigation presented here was to analyse the behaviour of unreinforced and reinforced, with prefabricated truss-type called MURFOR®, masonry walletts. All specimens were subjected to diagonal compression. Three groups of specimens were tested, each using a different mortar coating Testing was especially focused on the main technological problem of the proper adhesion between mortar and the reinforcement surface.

Keywords: ACC blocks, diagonal compression, reinforced masonry, shear strength

Streszczenie

Od ponad 10 lat elementy wykonane z autoklawizowanego betonu komórkowego są jednym z najbardziej popularnych materiałów stosowanych do wznoszenia budynków mieszkalnych zarówno w Polsce, jak i Europie. Zalecana technologia wykonywania konstrukcji z takich materiałów wymaga stosowania cienkich spoin wspornych i niewypełnionych spoin czołowych. Niestety konstrukcje wznoszone w tej technologii są podatne na zarysowania. Jedną z metod ograniczania zarysowań jest wprowadzenie do spoin wspornych prefabrykowanego zbrojenia. Głównym celem prezentowanych badań jest analiza zachowania się muru niezbrojonego oraz zbrojonego zbrojeniem typu MURFOR®. Badane elementy poddano ukośnemu ściskaniu. Przebadano trzy grupy elementów próbnych, różniące się sposobem otulenia zbrojenia zaprawą. W trakcie analizy zwrócono szczególną uwagę na główny technologiczny problem, dotyczący zapewnienia prawidłowej przyczepności zbrojenia do elementu murowego.

Słowa kluczowe: bloczki z betonu komórkowego, ukośne ściskanie, zbrojone mury, wytrzymałość na ścinanie

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1. Introduction

It is often found that the introduction of new products in the construction market results in both positive and negative consequences. An example here may be solid blocks made of AAC with grip holes on both sides of the head. Such masonry units are recommended for erecting masonry wall structures using non-filled vertical joints. It was found that masonry structures built using this type of masonry unit displayed different behaviour than popular masonry made of other materials, especially in the case of stiffening walls typically subjected to horizontal wind loads. As a result of stress caused by horizontal shear forces, diagonal cracks are often found to develop. This situation is unacceptable mainly from a serviceability point of view. Therefore, the solution for improving crack resistance for these types of walls is in wide demand. One of the methods used is to introduce a special reinforcement which will be placed in bed joints. The most popular method recommended worldwide is to use one of the custom-prefabricated steel reinforcement types specified in EN 845-3:2000 [1]. Occasionally, and especially during last ten years, we find attempts to use composite materials (such as FRP laminates) introduced in masonry due to their improvement properties [2]. Finally, the authors also tested availability and effectiveness of CRFP and GRFP used to improve crack resistance when AAC block masonry with thin joints is subjected to in-plane shearing loads [3].

Unfortunately, the basic part of European masonry standard EN 1996-1-1:2005 [4] (Eurocode 6) does not offer any design method or formulae for the determination of the load-bearing capacity of reinforced masonry walls made of any masonry units and types of mortar which are subjected to vertical and horizontal loads. Theoretical and experimental studies on the subject have been carried out worldwide for more than 20 years. Studies carried out in other countries focus mostly on the behaviour of masonry walls with different types of reinforcement placed in bed joints and subjected to shearing and shearing with precompression – see [5], [6] and [7]. In past years, similar studies were also carried out at the Silesian University of Technology. Masonry wallettes made of solid clay bricks and general purpose class M7 mortar were subjected to horizontal shearing [8] and vertical shearing with and without precompression [9]. This type of reinforced wall was also tested and analysed in terms of seismic and dynamic shear loads and influences, especially in countries with seismic activity [10]. Unfortunately, most research works refer to using different types of bed joint reinforcement – mainly in masonry walls made of solid clay bricks or other types of bricks and blocks excluding AAC blocks. As a result, a significant lack of knowledge in this field is still found, especially in terms of the technological requirements of bed joint reinforcement application. To fill this gap, comparative tests of unreinforced and reinforced masonry wall specimens were conducted with truss-type prefabricated bed joint reinforcement. The tests were performed at the Department of Structural Engineering of Silesian University of Technology in Gliwice.

This article is an extended version of the paper presented at 7thInternational AMCM 2011 Conference, held in Cracow in 2011 [11]. It presents the analysis of the behaviour of unreinforced and reinforced masonry wallettes made of AAC blocks with thin joints and non-filled head (vertical) joints, based on the results of tests conducted. The experimental tests were carried out using speciments subjected to diagonally compressive loading, according to RILEM LUMB 6 [12] and American Standard ASTM E519-81 [13] regulations. These two standards are almost fully compatible. The problem of bed joint reinforcement,

correctly coated with mortar to ensure required adhesion, was identified and analysed. The article presents a comparison of two series of reinforced specimens which differed in terms of the number of mortar layers covering the steel bed joint reinforcement with the unreinforced elements taken as reference. During laboratory tests, shear stresses resulting in the development of micro-cracksand visible cracks (τ_{cr}), as well as ultimate shear stresses (τ_u) were determined and analysed. Additionally, the in-plane deformability of loaded walls (values of no-dilatational strain angle of θ and shear modulus) during cracking and at failure was presented. To identify the impact of the reinforcement application method (using of one or two layers of mortar coating for reinforcement) on the behaviour as well as the mechanical and deformability properties of masonry, the modes of failure of all tested specimens were observed, analysed and disscused.

2. Tested elements and test technique

2.1. Description of the test specimens

Laboratory tests were carried out using small masonry wallettes made of AAC solid blocks (Ytong Planblockstype W) and a typical thin joint mortar recommended by the block manufacturer. Specimens were prepared with non-filled head (perpend) joints with rectangular shape and overall dimensions of 900 × 805 × 240 mm (Fig. 1). AAC blocks used for preparation had the density of ρ_v = 600 kg/m^3 and the normalised compressive strength of f_b = 4.65 N/mm^2. The dimensions (length × height × width) of individual masonry units with rectangular prism shapes were 599 × 199 × 240 mm. Each masonry unit had grip holes on both sides of the head. The mean (tested) compressive strength cement mortar supplied by the manufacturer was f_m = 12.4 N/mm^2.

In two series of specimens, a prefabricated truss-type steel reinforcement (compatible with the requirements specified in EN 845-3 [1]) with the characteristic value of yield strength of f_{yd} = 350 N/mm^2 was placed in each bed joint. The reinforcement density ratio was 0.056% – which is slightly higher than the minimum value specified in Eurocode 6 [4] (ρ = 0.05%) for improving the material properties of masonry reinforced with this method.

In the first phase of testing, marked as (**Y-UR**) series, the specimens were unreinforced. The other two series, (**Y-R-1** and **Y-R-2**) included specimens with reinforced bed joints – see the test programme in Table 1. Reinforced elements differed in terms of the number of layers of mortar coatingthe reinforcement in each bed joint. In specimens of (**Y-R-1**) series, only one layer of mortar, located in every bed joint was applied. The reinforcement was placed into the fresh mortar. The outcome of using this method, especially if the workmanship quality is poor, is that some parts of longitudinal flat wires were not properly covered by mortar (the thickness of the mortar layer was 3 mm, but the thickness of the flat longitudinal steel wire was 1.25 mm). The reinforcement technology applied in masonry wallettes tested in the (**Y-R-2**) series was slightly changed. The reinforcement was placed between two layers of mortar (with the thickness of each layer of 2 mm). This allowed for the adequate coating of reinforcement and guaranteed a higher adhesion rate between the reinforcement and mortar; however, the total thickness of the bed joint exceeded 5 mm. Such value of the bed joint thickness is too high in relation to the maximum specified in Eurocode 6, where the thickness of thin joints is limited to 3 mm.

Table 1

Test programme

Series	Number of tested elements	Type of specimens	The number of layers of covering/ thickness of each mortar layer
Y-UR	5	unreinforced	1 mortar's layer/thickness of 3 mm
Y-R-1	5	reinforced	1 mortar's layer/thickness of 3 mm
Y-R-2	5	reinforced	2 mortar's layers /thickness of 2 mm each

Type of used prefabricated reinforcement, shape of tested specimens, their overall dimensions and localisation of measuring devices are shown in Fig. 1.

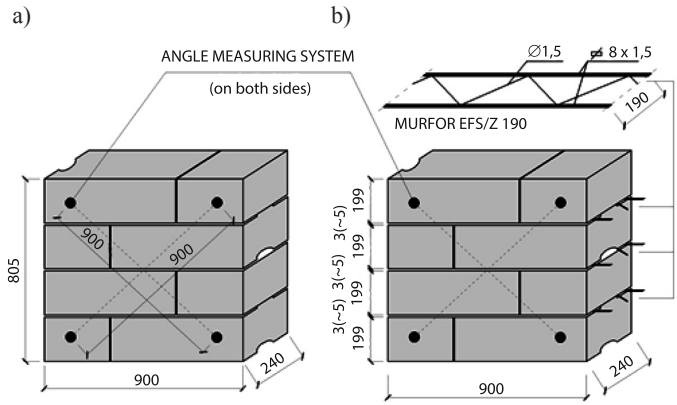

Fig. 1. Shape and overall dimensions of the test specimens: a) unreinforced specimens (**Y-UR** series), b) wallettes with truss type reinforcement in bed joints(**Y-R-1** and **Y-R-2** series)

2.2. Test stand and techniques

All specimens were subjected to a diagonally compressive load according to RILEM LUMB 6 [12].This international standard also provides the procedures for the explanation of results and their analysis. The compressive load was applied using a hydraulic press machine with the range up to 2000 kN through steel blocks positioned on two diagonally opposite corners of the specimen. The load was applied in one cycle – from zero up to the failure. The diagram of the test stand is shown in Fig. 2 and the view of the specimen prepared and ready to be tested is shown in Fig. 3.

When testing each specimen, the load force level and the displacements were measured with LDV gauges. The gauges were fixed to both surfaces of each wallette (using stiff resin-epoxy glue) along two diagonals of each model. The length of the base was 900 mm taken according to the guidelines of American Standard ASTM E519-81 [13], and should cover the greater part of the length of the specimens. Additionally, at failure, the crack pattern of each specimen was observed and recorded.

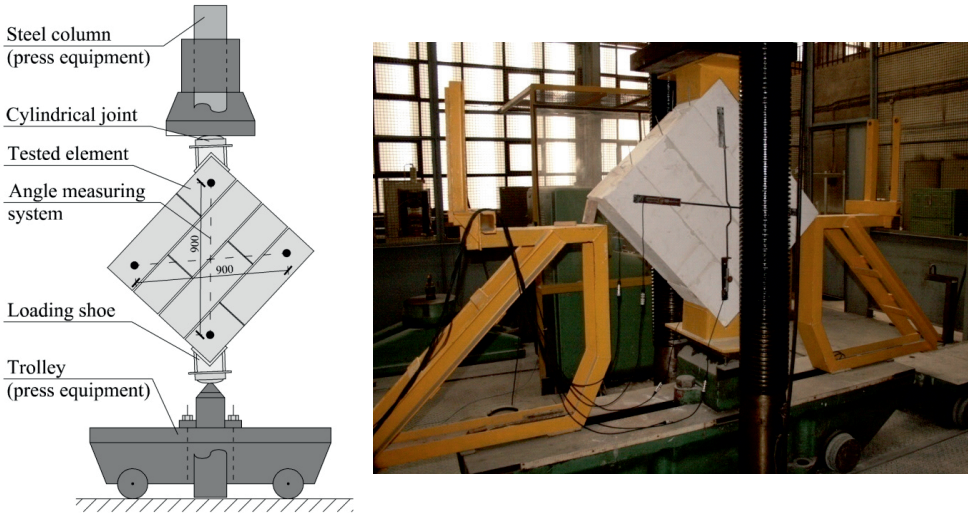

Fig. 2. Scheme of the test stand and view of the masonry wallettes ready to test

For each re-ordered force level F_i (at the i-th load level) the average value of the shear stresses $\tau_{v,i}$, defined as the quotient of load force F_i and the vertical cross-section area of the wall specimen A_h (along the diagonal),was calculated:

$$\tau_{v,i} = \frac{F_i}{A_h} = \frac{F_i}{t\sqrt{l^2 + h^2}}$$

where:

F_i – is the vertical load value at the i-th loading level,
t – 240 mm is the thickness of the wall specimens,
l – 922 mm is the length of the wallette,
h – 1009 mm is the height of the specimen.

To measure the in-plane deformability of each specimen during load application, inductive, measurement (LDV) system was mounted on both surfaces of the tested wallette (Fig. 2).

The values of the non-dilatational strain angle (shear strain) Θ_i were calculated on the basis of the horizontal and vertical length changes according to trigonometric (deformed base measurement) relationship, separated as shown in Fig. 3.

The non-dilatational strain angle at i-th level was determined by the following formula:

$$\Theta_i = 2\,\mathrm{arctg}\left(\frac{|\Delta x| + |\Delta y|}{x + y + |\Delta x| - |\Delta y|}\right)$$

where:

Δx – is the change in length of the horizontal measuring base (x – direction),
Δy – is the change in length of the vertical measuring base (y – direction),
x, y – is the primary length of the bases (900 mm), as appropriate.

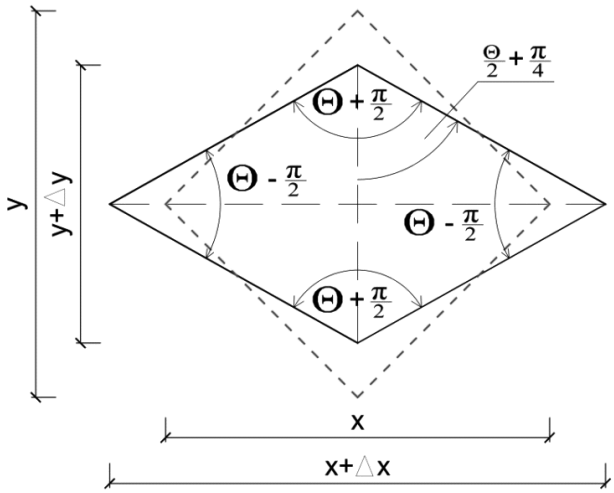

Fig. 3. Scheme for determination of non-dilatational strain angle (shear strain)

The shear modulus G_i (at the i-th load level) defined as the quotient of stresses $\tau_{v,i}$ and average value of corresponding to its' non-dilatational strain angle Θ_i was calculated from well-known formula:

$$G_i = \frac{\tau_{v,i}}{\Theta_i}$$

When a visible crack or cracks (with width greater than 0.1 mm) were observed, the values of the shear modulus G_{cr} was determined with the adequate level of F_{cr} shear stresses τ_{cr} to be loaded and their corresponding values of shear strains of $\Theta_{cr.}$.

At failure, the ultimate load level taken as the ultimate value of compressive force F_u was recorded and the maximum shear stresses τ_u and shear strain of in-plane deformation $\Theta_{u,}$ together with shear modulus G_u were calculated.

3. Results and analysis

3.1. Mode of failure

The modes of failure of unreinforced specimens (**Y-UR**) were typical for the diagonally compressed elements. In all masonry wallettes, the crack was running diagonally across the whole specimen. This crack was oriented perpendicularly to the direction of principal tensile stresses. Fig. 4 shows an exemplary view of unreinforced elements with visible diagonal cracks.

In two groups of reinforced specimens in the (**Y-R-1**) and (**Y-R-2**) series, the modes of failure were completely different and significantly dependent on the number of mortar layers used to coat the bed joint reinforcement. In all specimens where reinforcement

consisted of only one layer of mortar (**Y-R-1**), the damage was caused by splitting. The splitting occurred between the masonry units and reinforcement, always in the location where the mortar coating did not suitably cover the whole surface of the reinforcement (flat steel longitudinal rods of the truss-type reinforcement). This type of failure shows that using only one layer of mortar in reinforced masonry was not sufficient to guarantee a full connection (adhesion) between the reinforcement and mortar. Additionally, based on the deformation observed, the resulting damage was dangerous for the stability of the whole structure.

In (**Y-R-2**) series, a completely different mechanism of failure for all specimens with two layers of mortar coating was observed. No splitting effect between the reinforcement and masonry units was noticed. The damage was caused by a roughly vertical crack, which was always running through the non-filled perpend joints. As in the case of cracks observed in unreinforced specimens (the failure which is typical for the elements subjected to diagonal compression), the crack was located along the axis which was perpendicular to the direction of the diagonal tensile stresses. The failure was pronounced when the crack width exceeded 1 mm. A vertical crack, with the width smaller than 0.1 mm was considered to be a secondary cracking state because no other type of damage was noticed at that time. The failure mode of all specimens of (**Y-R-2**) series can be considered safe, with no visible significant deformation and rapid action. The failure mode observed demonstrated a better behaviour of the bond between the reinforcement and masonry mortar when two layers of mortar were used. This situation was different than the modes of failure noticed in specimens in (**Y-R-1**) series, where the destabilisation of the elements inside the wallette occurred.

The pictures of damaged reinforced specimens are shown in Fig. 5. Fig. 5a and 5b present elements with one layer of mortar (**Y-R-1**), while Fig. 5c and 5d show modes of failure of wallettes with two layers of covering mortar (**Y-R-2**). Additionally, in Fig. 5b the splitting surface and the reinforcement without covering mortar are clearly visible.

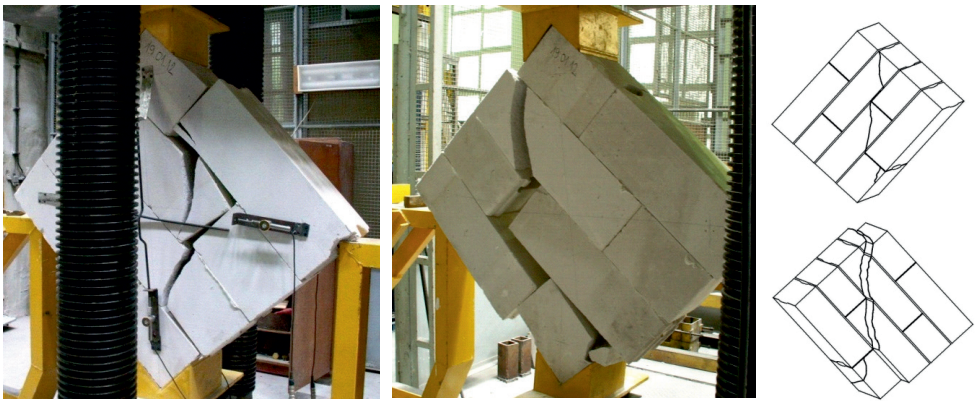

Fig. 4. View of typical modes of failure in case of unreinforced specimens

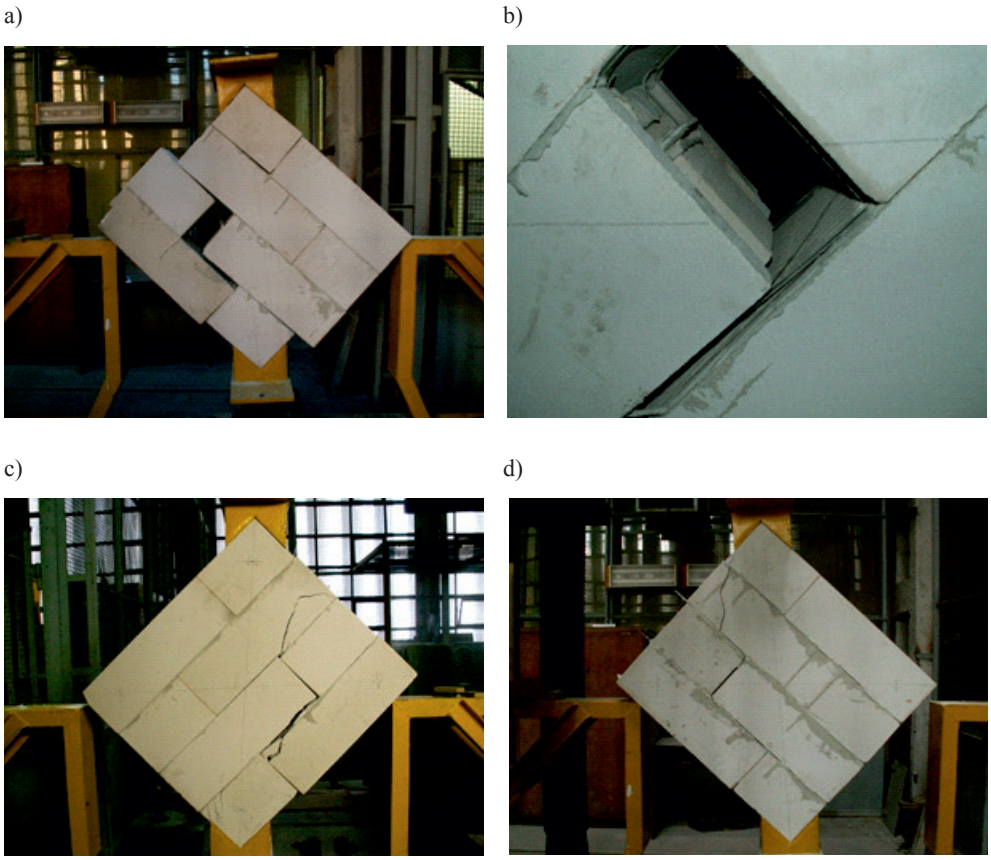

Fig. 5. Typical modes of failure of reinforced specimens: a) specimens made with one mortar layer (**Y-R-1**), b) splitting surface with visible reinforcement non-covered by mortar, c) and d) specimens made using two layers of mortar (**Y-R-2**)

3.2. State of cracking and ultimate stresses

During laboratory tests, at each measuring step (approximately every 30 seconds), the compressive force and displacements measured by inductive LDV gauges were recorded. Based on this, the characteristic values of forces and stresses (corresponding with the cracking moments and at the state of failure) were determined. Table 2 shows the values of cracking forces F_{cr1}, F_{cr2} and the maximum forces (the ultimate value) recorded at the state of failure F_u and corresponding to each force of cracking stresses τ_{cr1}, τ_{cr2} and the ultimate shear stresses τ_u observed. Table 3 shows the mean values of all characteristic forces ($F_{cr1,mv}$, $F_{cr2,mv}$, $F_{u,mv}$) and stresses ($\tau_{cr1,mv}$, $\tau_{cr2,mv}$ and $\tau_{u,mv}$) grouped for unreinforced specimens (**Y-UR**), the reinforced wallettes with one layer of mortar (**Y-R-1**) and the reinforced specimens with reinforcement placed between two mortar layers (**Y-R-2**). Additionally, the reinforced specimens (both **Y-R-1** and **Y-R-2**) were compared with the unreinforced specimens (**Y-UR**) in terms of the first cracking stresses and ultimate stresses.

Table 2

Values of diagonal compressive forces and calculated shear stresses at the state when first crack appear and at the state of failure

Element	F_{cr1} [kN]	τ_{cr1} [N/mm²]	F_{cr2} [kN]	τ_{cr2} [N/mm²]	F_u [kN]	τ_u [N/mm²]
Unreinforced specimens						
Y-UR-1	16.78	0.058	–	–	110.03	0.380
Y-UR-2	17.08	0.059	–	–	108.20	0.373
Y-UR-3	19.37	0.067	–	–	105.43	0.364
Y-UR-4	18.62	0.064	64.66	0.223	87.98	0.304
Y-UR-5	18.87	0.065	–	–	93.56	0.323
Reinforced specimens with one layer of covering mortar						
Y-R-1-1	17.69	0.061	–	–	102.50	0.354
Y-R-1-2	26.02	0.090	–	–	98.96	0.342
Y-R-1-3	18.19	0.063	60.57	0.209	97.04	0.337
Y-R-1-4	27.47	0.095	59.22	0.204	93.81	0.324
Y-R-1-5	19.29	0.067	–	–	108.46	0.375
Reinforced specimens with two layers of covering mortar						
Y-R-2-1	85.07	0.294	104.21	0.360	128.50	0.443
Y-R-2-2	51.89	0.179	120.36	0.415	135.69	0.468
Y-R-2-3	79.09	0.273	134.44	0.464	149.97	0.518
Y-R-2-4	80.55	0.278	110.10	0.380	132.46	0.457
Y-R-2-5	82.44	0.284	104.92	0.362	121.43	0.419

Table 3

Mean values of characteristics diagonal compressive forces and shear stresses for all series of tested elements

Element	$F_{cr1,mv}$ [kN]	$\tau_{cr1,mv}$ [N/mm^2]	$F_{cr\text{-}vis,mv}$ [kN]	$\tau_{cr\text{-}vis,mv}$ [N/mm^2]	$F_{u,mv}$ [kN]	$\tau_{u,mv}$ [N/mm^2]
Unreinforced**Y-UR**	18.14	0.063	–	–	101.04	0.359
Reinforced with 1 layer of mortar **Y-R-1**	21.73	0.075	59.90*	0.207*	100.15	0.346
Reinforced with 2 layers of mortar **Y-R-2**	75.81	0.262	114.81	0.396	133.61	0.461
Y-R-1/Y-UR	–	**1.19**	–			**0.96**
Y-R-2/Y-UR		**4.16**				**1.28**
Y-R-2/Y-R-1		**3.49**				**1.33**

* uncerntaily average values – obtained on the basis of only two results.

The analysis of both the cracking and maximum (ultimate) stresses at failure leads to very interesting conclusions. Development of the first cracking was recognised as a typical micro-crack, barely visible on the surface of the element. Their occurrence in masonry of non-filled perpend joints caused during the first load phase, the closing of internal spaces between the masonry units and tightening of the masonry wallettes structure. This phenomenon was noticed during the first cracking observed on the basis of the recorded data and recognised as disturbance on the displacements diagram. The next cracking state (the second crack), clearly observed in reinforced specimens with a proper bonding between the reinforcement and masonry units (using two layers of mortar **Y-R-2**), was found when visible vertical cracks appeared on the surface of the specimens. The typical width of the cracks was smaller than 0.1 mm. The development of the second crack was also noticed on the displacement diagrams as the changing of the inclination of the graph line.

In both reinforced elements, the values of cracking stresses (first and second) were higher than those noticed in the unreinforced specimens. In the unreinforced masonry wallettes, the first cracks were found for stresses in the range of τ_{cr} = 0.058÷0.067 N/mm^2. Slightly higher values (τ_{cr} = 0.061÷0.095 N/mm^2) were recorded for reinforced specimens with 1 layer of mortar. When using 2 layers of mortar coating, the increase of stresses at the cracking moment was significant because the stresses amounted to τ_{cr} = 0.179÷0.294 N/mm^2. For both types of the reinforced masonry wallettes, the delay in development of cracks was noticed, but in the first group of reinforced elements (**Y-R-1**), the increase analysed for the average values amounted to only 19%, and in the reinforced elements with 2 layers of mortar coating (**Y-R-2)**, it was much more significant and was as high as 316%. The comparison of both reinforced types of specimens shows that the introduction of two layers of reinforcement mortar coating corresponded with 249% increase in micro-cracking stresses.

Analysis of a possibility of a second crack occurrence shows that only specimens with two layers of mortar developed visible cracks with no ensuing danger of rapid damage to masonry wallettes. The value of stresses, at the time when the second crack was observed, was very high and was determined to be equal to 0.86% of the ultimate stresses. In unreinforced elements and the first series of reinforced elements (**Y-R-1**), a visible and safe second crack was practically not observed. The development of cracks in the unreinforced element occurred together with a simultaneous rapid damage of this element. In reinforced specimens with one layer of mortar coating, the crack development was the result of a splitting effect observed and recognised as the failure state of this element.

A similar tendency was found in the analysis of the ultimate stresses. Improvement of the maximum stresses was recorded only for two layers of mortar application. Unfortunately, when using only one layer of mortar, the failure occurred almost at the same moment as the failure of unreinforced specimens (the difference of the ultimate stresses amounted only up to 1% and should be neglected). This phenomenon was associated with a splitting effect observed in reinforced specimens (**Y-R-1**), which occurs instead of a wide vertical crack observed in unreinforced specimens (**Y-UR**).

In the second group of reinforced specimens (with two layers of mortar coating in **Y-R-2** series), failure occurred later than in unreinforced specimens. The maximum (ultimate) stresses were higher than 35% when compared to the maximum stresses noticed for unreinforced elements. In this case, reinforcement was placed between the two layers of mortar, which allowed for a better bonding between the reinforcement and ACC blocks and thus the lack of occurrence of the splitting effect.

3.3. In-plane deformations

An analysis of deformation characteristics for the tested elements was conducted on the basis of in-plane deformability parameters obtained from the displacement measured using inductive gauges sets fixed along the diagonals of both surfaces of the specimens. The recorded data allowed for the determining of values of the non-dilatational strain angles occurring in the in-plane stiffness of masonry wallettes subjected to diagonal compression. Additionally, the values and changes of shear modulus were also analysed.

In Table 4, the values of non-dilatational strain angles (shear strains), determined at the time of the appearance of first and second cracks Θ_{cr1}, Θ_{cr2} and at the state of failure Θ_u for all tested specimens are presented. The table also covers the average values of shear strains $\Theta_{cr,mv}$, $\Theta_{u,mv}$ and shear modulus for both cracking moment G_{cr} and the average value $G_{cr,mv}$.

A comparison of calculated values, presented in Table 4, indicates that deformation of both types of reinforced specimens should be discussed separately, because the behaviour of specimens of series (**Y-R-1**) was significantly different than that of the wallettes of (**Y-R-2**) series. Additionally, the deformation had to be analysed together with the values of stresses observed for characteristic moments, as shown in Table 2.

Using only one layer of mortar coating for the reinforced specimens (**Y-R-1**) resulted in the element's behaviour being almost the same as unreinforced specimens. The failure of both masonry wallettes (**Y-UR** and **Y-R-1**) was caused by the same shear stresses and the same deformability parameter. The non-dilatational strain angle calculated for reinforced specimens was Θ_u=1.161 mm/m, where the average value of strain angle for unreinforced

Table 4

Values of deformation parameters and shear modulus

Element	Θ_{cr1} [mm/m]	G_{cr1} [N/mm^2]	Θ_{cr2} [mm/m]	G_{cr2} [N/mm^2]	Θ_u [mm/m]	G_u [N/mm^2]
Unreinforced specimens						
Y-UR-1	0.169	343	–	–	1.074	345
Y-UR-2	0.333**	177**	–	–	2.942**	127
Y-UR-3	0.169	394	–	–	1.280	284
Y-UR-4	0.193	332	0.751	297	1.142	251
Y-UR-5	0.136	480	–	–	0.749	422
Reinforced specimens with one layer of covering mortar						
Y-R-1-1	0.062	981	–	–	1,069	331
Y-R-1-2	0.126	715	–	–	0,926	369
Y-R-1-3	0.089	782	0.257	833	0.797	565
Y-R-1-4	0.072	1312**	0.253	807	0.690**	469
Y-R-1-5	0.094	705	–	–	1.851	179
Reinforced specimens with two layers of covering mortar						
Y-R-2-1	0.828	354	1.048	343	2.416	184
Y-R-2-2	0.308	581	0.980	424	2.286	205
Y-R-2-3	0.527	518	1.081	429	2.330	222
Y-R-2-4	0.914	304	1.374	277	2.448	139
Y-R-2-5	0.774	368	1.063	341	1.436	292

** uncertain value not taken into account in mean values determination.

specimens was only 10% lower (Θ_u=1.061 mm/m). Development of the second cracking state was not noticed in either element; however, in two cases of reinforced specimens (most likely due to the fact that the reinforcement coating was a little better) a vertical crack was observed, but the deformation observed at the moment of cracking was not very high. Only in the case of the first cracking recorded, the strain deformation of the reinforced element was different than the one observed for the unreinforced element. The first micro-cracks occurred almost at the same load level as for the unreinforced specimens, but the corresponding deformation was smaller.

Table 5

Values of deformation parameter and shear modulus

Element	$\Theta_{cr1,mv}$ [mm/m]	$G_{cr1,mv}$ [N/mm^2]	$\Theta_{cr2,mv}$ [mm/m]	$G_{cr2,mv}$ [N/mm^2]	$\Theta_{u,mv}$ [mm/m]	$G_{u,mv}$ [N/mm^2]
Unreinforced **Y-UR**	0.200	387	–	–	1.061	288
Reinforced with 1 layer of mortar **Y-R-1**	0.089	796	0.255*	820*	1.161	383
Reinforced with 2 layers of mortar **Y-R-2**	0.670	425	1.109	363	2.183	208

* uncertain average values – obtained on the basis of only two results.

Analysis of the in-plane deformation of reinforced specimens, where reinforcement was fully covered by the mortar (two layers of mortar were used in **Y-R-2**), indicated a proper behaviour of these elements, especially in relation to masonry wallettes with the reinforcement placed only in one mortar layer (**Y-R-1** series). The micro-cracking in these elements occurred much later than in unreinforced specimens (the difference was about 330%). This is associated with higher deformation of specimens and relatively lower increase of shear modulus calculated at the development of the first crack. The increase of non-dilatational strain angles was up to 235% with a 10% increase in the values of shear modulus. This phenomenon was the result of a better adhesion between the reinforcement surface and masonry units. The reinforcement deforms together with the specimens when applying load and results in the masonry wallette being much more flexible and ductile. The increase of ductility was associated with the obvious development of the second crack and corresponded with intense in-plane deformation. The calculated average value of the non-dilatational strain angle for **Y-R-2** series of reinforced specimens (Θ_{cr2} = 1.109 mm/m) was comparable to the value determined for unreinforced elements (**Y-UR**) at failure (Θ_u = 1.061 mm/m). However, the second crack did not cause damage of reinforced elements (series **Y-R-2**). The in-plane deformation of reinforced masonry wallettes observed at failure was significant (when compared to the unreinforced specimens) and was as high as Θ_u = 2.138 mm/m. This resulted in a positive impact on the deformation of reinforced specimen by correct coating of the reinforcement by mortar. First of all, for reinforced

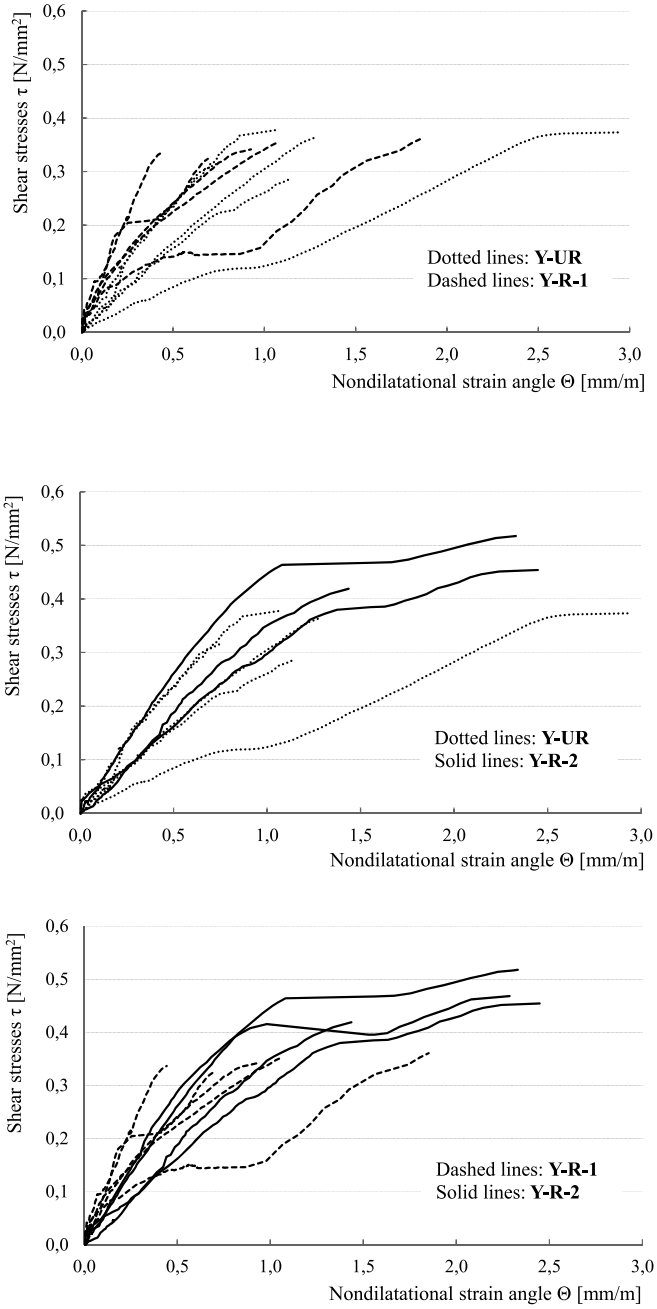

Fig. 6. Comparison of shear stress – non-dilatational strain angle ($\tau - \Theta$) relationships for: a) unreinforced specimens (**Y-UR**) and reinforced specimens with one layer of mortar (**Y-R-1**), b) unreinforced specimens (**Y-UR**) and reinforced specimens with two mortar's layers (**Y-R-2**) c) both reinforced specimens (**Y-R-1** and **Y-R-2**)

masonry wallettes, the no-crack phase was significantly extended (almost four times); secondly, the possibility of visible cracking development with no damage to the element was observed and thirdly, the deformation of the element at failure was two times higher than that of the unreinforced specimens. These phenomena are very desirable in the event of the occurrence of vertical forces (shearing).

In order to show noticed above remarks and observations, some diagrams with shear stress – shear strain (τ - Θ) relationships of all specimens were presented. Figures 4a and 4b show the behaviour of deformation of unreinforced specimens (**Y-UR**) in comparison with reinforced specimens with one layer of mortar (**Y-R-1**) and with two layers of mortar (**Y-R-2**). In the Fig. 4c the comparison of both reinforced specimens was presented.

4. Conclusions

The results of the carried out investigations of unreinforced and reinforced, using truss type bed joint reinforcement, masonry wallettes subjected to diagonally compressive loading were presented and discussed. Based on them, the significant influence of proper bed joint reinforcement covering by mortar on behaviour, mode of failure and positive modification of mechanical properties such reinforced masonry made of AAC blocks was recorded. The technological problem of bed joint reinforcement correct covering by mortar to ensure required adhesion was examined. Simultaneously, a very important influence of workmanship quality on behaviour and material properties of such masonry was stated.

In spite of testing only one type of masonry unit and mortar destined for thin joints, the obtained results permitted the formulation of some general conclusions for bed joint reinforced masonry walls subjected mainly to in-plane, especially shear loading:

1. A significant enhancement of material properties (i.e. shear strength) is observed only in situations where reinforcement is fully covered by mortar. Practically, this is not possible to guarantee using only one mortar layer. According to the presented investigation, in cases of masonry walls with bed joint reinforcement, two mortar layers (with the total thickness ca. 2×2 mm) should be used.
2. Using two layers of mortar has a very positive influence on crack resistance and mode of failure, shear (diagonal tensile) strength, and state of in-plane deformations of sheared (diagonally compressed) masonry. Splitting effect at the state of failure was not observed, but a shear strength enhancement of over 20% was recorded.
3. Reinforced masonry wallettes with two mortar layers were characterised by over three times higher shear stresses recorded for the state of first crack appearance and quite similar values of corresponded to them shear modulus in comparison to unreinforced members.
4. In case of masonry walls made using thin joints, it is very important to ensure a good quality of workmanship.
5. It is necessary to introduce Eurocode 6 regulations using joints with medium thickness, i.e. between 5 to 8 mm. Using bed joint reinforcement in situations where the maximum permitted thickness for thin joints should not exceed 3 mm is practically not possible.

Research works presented in this paper were carried out at the Department of Structural Engineering with partially financial support of BEKAERT S.A., Zwevegem, Belgium and own university grant of Politechnika Śląska in Gliwice.

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