Attempt to Describe the Mechanism of Work of Masonry Joints

Iwona Galman 1, Radoslaw Jasinski 2

1 Department of Structural Engineering, Silesian University of Technology, Akademicka 5, 44-100 Gliwice, Poland
2 Department of Building Structures, Silesian University of Technology, Akademicka 5, 44-100 Gliwice, Poland

Iwona.Galman@polsl.pl

Abstract. The issue of joints in masonry walls is often neglected and belittled from the structural point of view. It may even be said that the problem of joints is better investigated in terms of their thermal and acoustic insulation. Although such an approach can be justified to some extent in traditional construction (ceramic walls joined with traditional masonry bond), in case of joints with steel connectors such negligence can even lead to catastrophe. The issue of load transfer and co-operation between the crossing walls is very important for durability and safety of the structure. There is little experimental research in this topic worldwide. Designers do not have sufficient procedures and recommendations to design joints safely. Because of this void in the state of the art as well as the need to investigate the issue of masonry joints the authors decided to work on this subject. Experimental campaign is currently on going at the Faculty of Civil Engineering of the Silesian University of Technology in which joints in masonry walls made of AAC blocks. This paper presents the results of three specimens with traditional masonry bond. The tests were performed in a specifically constructed T-shaped test stand, with a web and a flange of ~89 cm length. In addition to presentation of the differences in cracking and failure mechanisms as well as in load-bearing capacity, the authors attempted to describe the mechanism of each type of the joint.

1. Introduction

When controlling the ULS conditions of structural masonry walls spatial co-operation of the walls with other structural elements is assumed. In case of reinforced concrete floors, because of the transfer of vertical service loads and bending moments, this co-operation seems quite obvious. It is, however, different for the walls located at the level of the same storey. In the elements loaded mainly vertically the adjacent walls influence the stability determining the effective height \( h_{ef} \) of the calculated wall. In the stiffening walls co-operation between the walls (or its lack) determines the stiffness \( K \) of the stiffening system (cross-section with or without flange), and consequently the magnitude of the acting loads (horizontal forces \( N \) and bending moments \( M \)). Wall joints should be considered with respect to the direction of the load:

- Joints in walls loaded mainly vertically, most commonly formed under uneven loading of the walls;
- Joints in stiffening walls occurring in all co-operating parts of the walls (flanged walls) caused by horizontal loading.
Except for the safety reasons, the problems of the building physics – such as thermal or acoustic insulation of these parts of the building structure – are also of importance. Looking at the wall joints from the point of view of safety and building physics, they are essential to consider at the phase of design, service and maintenance of the whole building. It can be even said that the wall joints are better investigated from the point of view of their thermal and acoustic insulation than their structural behaviour. This tendency results undoubtedly from strict legal regulations, such as art. 20 of the directive 2010/31/EU on the energy performance of buildings [1]. Structural problem was neglected or belittled when traditional masonry bonds were applied and joints did not exhibit damages. Currently, with introduction of new types of masonry units and connectors damages become common. Design standards require that designers ensure appropriate co-operation between the crossing walls but do not provide any algorithms to control the ULS conditions, not only for full masonry bond but also when different types of connectors are used. Only constructional recommendations are given. Moreover, the procedures for empirical determination of the load-bearing capacity of the joints have not been standardized. Therefore, design of these parts of the buildings has purely intuitive character, unsupported by tests or detailed analyses. Research material related to the wall joints is not broad enough to verify normative statements, to investigate the behaviour of the joint and to formulate some practical construction rules with special regard to new construction technologies of masonry walls (this joints, light joints, unfilled head joints, etc.). The aim of this work is thus to put an order and fill the void in the current state of the art on wall joints. The paper presents code requirements, conclusions drawn from a little number of performed tests and, most importantly, the results of own tests with an attempt to engineering description of the mechanism of work of a wall joint.

2. State of the Art

2.1. Code regulations

Eurocode 6 [2] requires that the walls perpendicular or slanted with respect to one another should be connected in a way which ensures transfer of vertical and horizontal loads from one wall to another. This can be realized with: masonry bond (figure 1a), connectors (figure 1b) or extended reinforcement anchored in the other wall (figure 1c). The joint with steel connectors or reinforcement should be equivalent to the masonry bond from the structural point of view and at the same time cannot impair other properties of the wall, such as thermal or acoustic insulation. The number and spacing of connectors should be determined with static calculations.

![Figure 1. Wall joints: a) full joint, b) with the use of connectors, c) with the use of longitudinal reinforcement; 1 – connector, 2 – structural type reinforcement](image)

It is recommended that the crossing load-bearing walls are constructed simultaneously ensuring proper bonding of masonry units in their joint plane. In general recommendations EC6 requires that the structure and joints between its components ensure proper stability and stiffness during construction and service life. Nevertheless, no detailed recommendations are given. Less rigorous rules apply to non-structural walls, which carry only their self-weight and which if removed would not negatively affect the structure. Here, in addition to traditional bond and a joint with steel connectors,
the walls can be connected with plaster or with a properly shaped groove in the adjoining wall (“dovetail joint”). Contact joint does not provide support of the wall in the vertical plane but only limits freedom of horizontal displacements. In case of vertical joint between a non-bearing wall serving a fire-proof function and a load-bearing wall [3] standard recommends application of steel connectors. Co-operation between crossing walls in stiffening walls and vertically loaded walls is ensured only when the joint is able to transfer shear forces occurring in their contact plane. In addition to control of the ultimate limit states due to vertical loads it is also necessary to control code requirements for joints and ultimate limit state of shear in the joint. As it has already been mentioned in the introduction, shear stresses in the joint can be induced by non-uniform loading of wall parts (figure 2a).

![Figure 2. Longitudinal shear stresses in the joint: a) of stiffening walls acc. to [4], b) walls loaded vertically; 1 – stiffening wall, 2 – wall perpendicular to stiffening wall, 3 – real diagram of longitudinal shear stresses, 4 – idealized diagram of longitudinal shear stresses, 5 – idealized diagram of longitudinal shear stresses in the joint](image)

Such situation is very common because it appears in all walls loaded mainly vertically, with unequal values of \( \sigma_{1g} \) and \( \sigma_{2g} \), and even identical elastic properties of the joined walls. The values of static stresses in a wall joint depend on the effective flange width of the walls and the value of stresses normal to the wall. Nevertheless, so far no methods have been proposed, even approximated ones, for determination of stress trajectories and design standards do not require computational control of load-bearing capacity. In case of joints in stiffening walls – figure 2b (acc. to [4]), [2] standards recommend to control ULS conditions giving no calculation procedure. However, the guidelines of EC6 can be used on constant values of shear stresses on the top and bottom edge of the wall instead of parabolic diagrams of stresses determined according to the Żurawski equation. This assumption results in linear variation of shear stresses \( \tau(x) \) in the joint (figure 2a acc. to [4]). Total transverse force of crossing walls can be then determined with the formula:

\[
V_{Ed} = \int_0^h \tau(x) dx = \frac{\tau_g + \tau_d}{2} A_t,
\]

where:

- \( \tau_g \) – mean shear stress at the bottom of the wall, \( \tau_g = \frac{V_{Ed}}{l_{c,g}} \),
- \( \tau_d \) – mean shear stress at the top of the wall, \( \tau_d = \frac{V_{Ed}}{l_{c,d}} \),
- \( l_{c,g} \), \( l_{c,d} \) – length of compressive zone of the section (determined acc. to the procedure given in Sec. 2),
- \( A_t \) – area of the joint surface between the transverse wall and the stiffening wall.

This way the obtained transverse force in the joint \( V_{Ed} \) should be compared to the design resistance \( V_{Rd} \) determined according to p. 3.6.2 of EC6 [2]. Moreover, according to the National Annex of the before mentioned standard, because of the direction of action of the forces with respect to the bed joints plane, the code equation should be modified. The initial value of the shear strength of the wall \( f_{sko} \) should be replaced with the shear strength in the direction perpendicular to bed joints \( f_{svk} \). It might
be problematic to determine the characteristic value of the shear strength of the wall in vertical
direction \( (f_{\text{vk}}) \). This value is not specified except for the Polish NA to EC6 [2]. In case of the wall
joints made with connectors the calculated values of support reactions should be compared to the
design resistance of the joint (declared by the connectors producers). If resistance is exceeded the
number of connectors must be increased or double connectors should be used. It must be noted that the
desired value is the resistance of the joint with the connector, not the resistance of the connector itself.

2.2. Experimental research

Research material related to wall joints is very scarce. Variety of the used test stands and lack of
unified procedures makes it practically impossible to compare the obtained results. Therefore, the
results presented later in the paper should be treated demonstratively and no quantitative conclusions
should be made. Well documented tests of the walls in which simulations were made of a joint
between a stiffening wall and co-operating wall were presented in [5]. The tests covered symmetrical
H-shaped models made of ceramic masonry units, without slab – figure 3a or with a co-operating
reinforced concrete slab imitating a part of reinforced concrete floor slab – rys.3b. The walls were
joined with a full masonry bond. The load was transferred to the central part of the wall inducing shear
of the joint. In addition to the visible increase in the load-bearing capacity, the authors proved
differential mechanism of cracking and failure. In the models without the co-operating slab the walls
failed in the upper part close to the location of the load – figure 3a and along the whole height when
the RC slab was introduced – figure 3b. In the tests presented in [6], performed on the H-shaped
models of the walls, investigation was made of the influence of the type of wall joint. The tests
covered the elements without a bond and with a full masonry bond, made of hollow concrete blocks.
In this case the walls were loaded uniformly on the upper edge to obtain identical stress normal to the
bed joints plane. In the models without the bond (figure 3c) failure was caused by the loss of stability
of the shorter part of the wall. In the models with full bond the joined was sheared at the whole height – figure 3d. Except for the wall joints made with masonry units the tests were also performed of the
reinforced joints. Paganoni and D’Ayala [7] checked the effectiveness of steel anchors in wall joints. T
shape of the tested element was designed to simulate the realistic behaviour of the joint to the greatest
possible extent. Similar tests were performed by Maddaloni and his team [8, 9]. However, in this case
the effectiveness of innovative clamp anchors (carbon rods wrapped vertically and spirally with invar
steel mats) was evaluated. The performed tests showed high effectiveness of both types of joints.
Application of the innovative connector made of carbon fibre almost doubled the magnitude of the
load causing cracking in the joint with respect to the joint without connectors and without the masonry
bond. Unfortunately, the mere results of the tests of connectors and their effectiveness is difficult to
interpret because no comparison was made to the resistance of the joint with traditional masonry bond.
It is worth to mention Brazilian tests [10] in which a comparison of load-bearing capacity was made of
the walls connected with three different methods: with traditional masonry bond, with steel mesh cast
in the bed joint and with steel anchor. The tests showed that the joint with steel elements was able to
carry 60% of the load obtained in the wall with masonry bond.

Summarizing, the issue of wall joints and co-operation of the walls is obviously marginalized and
poorly investigated. The are no standards related to determination of internal forces acting in the joints
of crossing walls. A few existing test results do not make it possible to describe the mechanism of
work of the joints, let alone to formulate recommendations for design and construction. Therefore, the
authors defined the following general goals of their research:

- investigation of the mechanism of cracking and failure of the joints of the walls made of the
  masonry units most commonly used in the country, i.e. AAC blocks;
- comparison of the load-bearing capacity of the joints made with traditional masonry bond and
  with the use of steel connectors.
a) b) c) d)

Figure 3. Patterns of failure cracking of test specimens: a) vertically loaded slender walls without RC slab [5], b) vertically loaded slender walls with RC slab [5], c) flanged walls with full joints [6], d) flanged walls without full joints [6]

3. Own experimental researches

3.1. Testing program

The tests were performed in the dedicated, specifically designed test stand, composed of a steel frame and vertical confining elements. The force causing shear in the joint was induced by a hydraulic press of 1000 kN range and measurements were recorded with a dynamometer of 250 kN range. The models were loaded in one cycle until failure by applying the force with 0.1 kN/s speed. Vertical load generating shear was transferred linearly along the whole height of the wall; thanks to that uniform shear stress was induced in the joint. Static scheme of the test models and the view of the test stand are shown in figure 4a, b. During the test continuous recordings were made of the loading and displacement of the loaded wall with respect to the non-loaded wall. Recordings were made with two independent systems. One side of the test model was monitored with the use of the optical displacement recorder ARAMIS. The other side was monitored with the use of three inductive displacement transducers of PJX-10 type with 10 mm range and 0.002 mm accuracy. Three series of three models of identical shape and size were made and tested. The models were monosymmetric and had a T shape with a web and a flange of ~89 cm length. A joint was formed between the loaded and non-loaded wall, which structure was differentiated. In the series of models denoted as P a traditional masonry bond was made between the web and the flange (figure 4a). The tests were performed on the models made of ABK masonry units and system mortar for thin joints, with unfilled head joints. Compressive strength of masonry, determined acc. to PN-EN 1052-1:2000 and presented in [11], was equal to \( f_c = 2.97 \) N/mm\(^2\), modulus of elasticity was equal to \( E_m = 2040 \) N/mm\(^2\), initial shear strength, determined acc. to PN-EN 1052-3:2004 and presented in [12] was equal to \( f_{vo} = 0.31 \) N/mm\(^2\), and shear modulus, determined acc. to ASTM E519-81 and presented in [13] was equal to \( G_{cr} = 329 \) N/mm\(^2\) and \( G_{1/3} = 475 \) N/mm\(^2\).

Figure 4. Scheme, view of the specimen and testing stand (description in the text)
3.2. Mechanism of cracking and failure

Behaviour of all models was similar. In the initial phase of loading no crashing was heard and no spalling was visible on the side surfaces of the elements. This phase lasted until first slanted cracks appeared in the direct vicinity of the walls joint – figure 5a. Increase of load caused visible development of existing cracks in the joint and their propagation towards the RC column transferring the load. In this phase the highest force was registered. Further increase caused visible increase in relative displacements and rotation of the walls. After failure the joint was disassembled – figure 5b. Almost vertical shearing of the joint was observed. No explicit damages were observed in other elements.

![Figure 5. Failure of a P series model a) first cracks in the reference model P_2 b) view of the joint at the moment of failure P_3](image)

3.3. Test results

Behaviour of the joint during loading was depicted in the diagrams of relationship between the load \( N \) and relative displacement of the joined walls \( u \) – figure 6. Until cracking of the joint, which happened under the load of around \( N_{cr} = 27.3 – 42.6 \text{ kN} \), displacements \( u \) were increasing almost proportionally to the increase in load. Hence, this phase was called an elastic phase. After cracking, in the post-elastic phase, joints were still able to transfer the load with simultaneous increase of displacements.

![Figure 6. Relationship between the total force and mean relative displacement of the joint – test and calculation results](image)

This phase terminated at the maximum forces of around \( N_u = 38.6 – 56.3 \text{ kN} \). Further attempts of loading in the failure phase caused a drop of forces registered by the force meter accompanied with an increase of the relative displacements. The force did not fall to zero and the joint was able to transfer residual load. In the final phase hardening was observed – the registered force increased. The final registered forces, conventionally called residual forces, preceded failure which was connected with complete spalling of the connected elements and relative rotation, and were equal to \( N_r = 10.2 – \)
18.8 kN. The values of forces and accompanying displacements are collectively presented in table 1, and linear approximation of the results is presented in figure 6. In particular phases of work stiffness of the joints were determined using the formulas (2) – (4), their values are collectively presented in table 2:

- Elastic joint stiffness:
  \[ K_t = \frac{N_{cr}}{u_{cr}}. \]  

- Post-elastic joint stiffness:
  \[ K_p = \frac{N_u - N_{cr}}{u_{u} - u_{cr}}. \]  

- Residual joint stiffness:
  \[ K_r = \frac{|N_r - N_u|}{u_r - u_{u}}. \]

### Table 1. Test results

| Model | Cracking force | Force at failure | Residual force | Displacement at the moment of cracking | Displacement right before failure | Residual displacement |
|-------|----------------|------------------|----------------|----------------------------------------|----------------------------------|-----------------------|
|       | \( N_{cr} \) kN | \( N_{cr, mv} \) kN | \( N_u \) kN | \( N_{u, mv} \) kN | \( N_{m} \) kN | \( u_{cr} \) mm | \( u_{cr, mv} \) mm | \( u_{u} \) mm | \( u_{u, mv} \) mm | \( u_r \) mm | \( u_{r, mv} \) mm |
| P 1   | 27.3           | 56.3             | 20.7           | 0.07                                   | 0.31                             | 6.36                  |
| P 2   | 42.6           | 33.7             | 50.0           | 14.9                                   | 0.12                             | 0.10                  | 0.25                             | 0.24                  | 6.97                  | 6.32                  |
| P 3   | 31.2           | 38.6             | 13.8           | 0.12                                   | 0.16                             | 5.64                  |

### Table 2. Joint stiffness

| Model | Elastic joint stiffness | Post-elastic joint stiffness | Residual joint stiffness |
|-------|-------------------------|----------------------------|--------------------------|
|       | \( K_t \) MN/m          | \( K_{t, mv} \) MN/m       | \( K_p \) MN/m           | \( K_{p, mv} \) MN/m       | \( K_r \) MN/m         | \( K_{r, mv} \) MN/m   |
| P 1   | 413                      | 119                        | 6                        | 5                        |
| P 2   | 341                      | 341                        | 60                       | 114                      | 6                      | 5                      |
| P 3   | 268                      | 163                        |                          |                          |                        |                        |

3.4. Proposal for description of a wall joint behaviour

Based on the performed tests an attempt was made to generalize the obtained results. The defined phases of work of the joint allowed to create a diagram of \( N - u \) relationship for an unreinforced joint of the AAC masonry walls. Particular phases of work of the joint were approximated with straight lines. Elastic phase was defined in the range of 0 – \( N_{cr} \), post-elastic phase in the range of \( N_{cr} - N_u \), and failure phase in the range of \( N_u - N_r \) loads. The obtained relationships are schematically shown in figure 7, while the comparison of the test results and calculation results are collected in figure 6.

It might be problematic to use the obtained test results in practice because this requires to perform complicated tests. For the purpose of this work it was decided to relate the obtained results of the tests of joints to the results of standard tests performed according to the valid standards. In the works [14, 15] the results were presented of the tests of masonry walls made of AAC blocks diagonally compressed according to the ASTM E519-81 standard [13]. Based on these tests, in addition to shear modulus the values of cracking stresses \( \tau_{cr} \) and stresses at failure \( \tau_u \) were determined. The work [16] presents in turn the results of the tests of shear parameters according to the PN-EN 1052-3:2004 standard [12] as well as the parameters describing the behaviour of the wall in the failure phase when
the maximum shear stresses are reached. The results of the tests of material parameters were related to the results of the joints tests and are collectively presented in table 3.

Table 3. Comparison of mean results of joints tests and properties of walls according to ASTM E519-81 and PN-EN 1052-3:2004 standards

| Joint model test | Diagonal compression test |
|------------------|--------------------------|
| Cracking shear stress | N/mm² | A/Ncrcr = τcr,RL |
| Failure shear stress | N/mm² | A/Nuu = τu,RL |
| Residual shear stress | N/mm² | A/Nrr |
| Cracking shear stress | N/mm² | τcr,RL |
| Failure shear stress | N/mm² | τu,RL |
| Stiffness | MN/m | \( h \cdot \frac{AGK}{RL} \) |
| | | 312 |
| | | 0.130 |
| | | 0.186 |
| | | 0.057 |
| | | 0.192 |
| | | 0.196 |
| | | 117 |

Based on the obtained test results of the diagonally compressed walls empirical coefficients \( \alpha, \alpha_1 \), and \( \beta, \beta_1 \) were obtained which served to calibrate the linear function describing the behaviour of the joint in each phase of work:

- **Elastic joint stiffness and displacement:**
  \[ K_i = \alpha K_{RL} \Rightarrow u_{cr} = \frac{N_{cr}}{K_i}, \]  \( \alpha = 2.8, \alpha_1 = 0.6, \beta = 0.33, \beta_1 = 0.9 \) – empirical coefficients.

- **Post-elastic joint stiffness:**
  \[ K_p = \beta K_i \Rightarrow u = u_{cr} + \frac{N_u - N_{cr}}{K_p}, \]  \( N_u = \beta u_{u,RL} A, \)  \( u_{cr} = \alpha_1 \tau_{cr,RL} A, \)

where: \( A = 0.26 \text{ m}^2 \) – cross-section area of the joint, \( \alpha = 2.8, \alpha_1 = 0.6, \beta = 0.33, \beta_1 = 0.9 \) – empirical coefficients.

It is more difficult to describe the behaviour of the joint in failure phase where dry slip of the disconnected walls occurs. Identical behaviour of the wall was obtained performing the test according to the PN-EN 1052-3:2004 standard. In this type of tests relative displacements were measured between the two masonry elements joined with the mortar and fracture energy of the joint was determined to be \( G_f^{II} = 2.37 \cdot 10^4 \text{ MN/m} \) [16]. According to the continuum fracture mechanics fracture energy allows to describe the behaviour of brittle materials in failure phase. The idea of determination of the fracture energy performed in the work [16] is shown in figure 8.
Figure 8. Diagram of $H - u$ relationship and identification of fracture energy $G_f^{II}$ [16] in standard tests according to PN-EN 1052-3:2004 [12].

Assuming that the fracture energy in the unit area of the joint $G_f^{III}$ is equal to the $G_f^{II}$ obtained in the tests, the residual displacement $u_r$ was defined by the relationship:

\[ G_f^{II} = G_f^{III} = \frac{1}{2} \left( \frac{N_u - N_r}{A} \right) u_r = u_u + \frac{2G_f^{II} A}{(N_u - N_f)}. \]  

(9)

where:

$N_r$ – residua force equal to:

\[ N_r = \gamma \tau_{u, RL} A, \]  

(10)

where: $\gamma = 0.3$ – empirical coefficient.

Displacements were calculated using the obtained quasi-empirical relationships; the results are collectively presented in table 4 and figure 6.

Table 4. Comparison of test results and computation results

| Joint model test | Calculation results |
|------------------|---------------------|
| Cracking force   | Failure force       |
| $N_{cr,mv}$ kN   | $N_{u,mv}$ kN       |
| Failure force    | Residual force      |
| $N_{u,mv}$ kN    | $N_{r,mv}$ kN       |
| Cracking force   | Failure force       |
| $N_{cr}$ kN      | $N_u$ kN            |
| Residual force   | $N_r$ kN            |
| $N_{r,mv}$ kN    | $N_u$ kN            |
| $N_{u,mv}$ kN    | $N_{r,mv}$ kN       |

| Cracking displacement | Failure displacement | Residual displacement |
|-----------------------|----------------------|-----------------------|
| $u_{cr,mv}$ mm        | $u_{u,mv}$ mm        | $u_{r,mv}$ mm         |
| Failure displacement  |                      |                       |
| $u_{cr}$ mm           | $u_u$ mm             | $u_r$ mm              |
| Residual displacement |                      |                       |
| $u_{cr}$ mm           | $u_u$ mm             | $u_r$ mm              |

4. Conclusions

The presented tests are a part of the research currently performed at the Laboratory of Civil Engineering of the Silesian University of Technology in the topic of the joints of walls made of AAC blocks. Hereafter are presented only three models with traditional masonry bond. The process of
damage and development of cracking in the wall with masonry bond was progressing in stages and was relatively smooth. Before failure visible cracking developed within the joint. Particular stages of work were defined based on which empirical proposal was made to determine forces and displacements in wall joints with the use of the results of less complicated standard tests. Satisfactory compliance between the test results and calculation results were obtained. Future works will cover expansion of the tests results range to increase the credibility of the obtained empirical coefficients and to create the model taking into account realistic distribution of shear stresses in the joint. In parallel, the authors are performing tests focused on the influence of different types of connectors on the load-bearing capacity of joints.

Acknowledgements
The authors would like to thank the company Solbet Sp. z o.o. for their valuable tips and supply of materials (masonry units, mortar and steel connectors) used to build the models and perform the tests.

References
[1] Directive of the European Parliament and of the Council 2010/31/EU of 19 May 2010 on the energy performance of buildings (Official Journal of the European Union L 153 of 18/06/2010, page 13) (In Polish).
[2] PN-EN 1996-1-1+A1:2013-05P, Eurocode 6: Design of Masonry Structures - Part 1-1: General rules for reinforced and unreinforced masonry structures. (In Polish)
[3] PN-EN 1996-1-2:2010/NA:2010P: Eurocode 6: Design of Masonry Structures - Part 1-2: General rules. Design due to fire conditions. (In Polish)
[4] L. Drobiec, R. Jasiński, A. Pickarczyk, „Masonry Structures According to Eurocode 6 and related Standards”, Volume 2, Wydawnictwo Naukowe PWN, Warszawa 2014. (In Polish)
[5] N.V. Capuzzo, M. R. S. Correa, M. A. Ramalho, „Distribution of vertical loads between interconnected masonry walls with and without a top slab”, 14th International Brick & Block Masonery Conference – IBMac 2008, Sydney (CD-ROM).
[6] L.O. Castro, R.C.S.S. Alvarenga, R.M. Silva, „Experimental evaluation of the interaction between strength concrete block walls under vertical loads” Revista IBRACON de Estruturas e Materiais Vol.9, No.5, 2016 (CD-ROM).
[7] S. Paganoni, D. D’Ayala, „Testing and design procedure for corner connections of masonry heritage buildings strengthened by metallic grouted anchors”, Engineering Structures Vol. 70/2014, s. 278 – 293.
[8] G. Maddaloni, A. Balsamo, M. Di Ludovico, A. Prota A, „Out of Plane Experimental Behavior of T-Shaped Full Scale Masonry Orthogonal Walls Strengthened with Innovative Composite Systems”, Fourth International Conference on Sustainable Construction Materials and Technologies, Las Vegas 2016 (CD-ROM).
[9] G. Maddaloni, M. Di Ludovico, A. Balsamo, A. Prota, „Out-of-plane experimental behaviour of T-shaped full scale masonry wall strengthened with composite connections”, Composites Part B 93, 2016, pp. 328 – 343.
[10] M.R.S. Corrêa, E.M.S. Moreira, M.A. Ramalho, „Experimental small-scale analysis of the connections between structural clay block work masonry walls submitted to vertical loads”, 11th Canadian Masonry Symposium, Toronto 2009 (CD-ROM).
[11] PN-EN 1052-1:2000 Methods of tests for masonry. Part 1: Determination of Compression Strength. (In Polish)
[12] PN-EN 1052-3:2004 Methods of tests for masonry. Part 3: Determination of Initial Shear Strength. (In Polish)
[13] ASTM E519-81 Standard Test Method for Diagonal Tension (Shear) of Masonry Assemblages.
[14] R. Jasiński, Ł. Drobiec, „Study of Autoclaved Aerated Concrete Masonry Walls with Horizontal Reinforcement under Compression and Shear”. Procedia Engineering, Vol. 161, 2016, pp. 918–924. DOI: 10.1016/j.proeng.2016.08.758
[15] R. Jasiński, Ł. Drobiec, „Comparison Research of Bed Joints Construction and Bed Joints Reinforcement on Shear Parameters of AAC Masonry Walls”. *Journal of Civil Engineering and Architecture*, Vol. 10, 12/2016, pp. 1329–1343, DOI: 10.17265/1934-7359/2016.12.004, ISSN 1934-7359 (Print); 1934-7367 (Online)

[16] R. Jasiński, „Research and modeling of masonry shear walls”. *PhD DsC Thesis*. Silesian University of Technology, Gliwice, Poland 2017. (in Polish)