ACTA SCIENTIARUM POLONORUM
Acta Sci. Pol.
Architectura 17 (4) 2018, 83–92
acta_architectura.sggw.pl ISSN 1644-0633 eISSN 2544-1760 DOI: 10.22630/ASPA.2018.17.4.43

ORIGINAL PAPER

Received: 30.03.2018
Accepted: 12.08.2018

JOINTS IN MASONRY WALLS

Iwona Galman✉, Radosław Jasiński

Faculty of Civil Engineering, Silesian University of Technology, Gliwice

ABSTRACT

The paper presents the results of the authors' own tests of joints in the walls made of AAC blocks. The tests were carried out on the originally modified test stand. Cracking morphology and failure mechanism of different types of joints were analysed and the load–displacement relationships of these joints were compared. The obtained results were also compared to the results of the tests performed on the reference models with traditional masonry bonds. In other tested models joints were realised with the use of steel connectors. To join the walls without the masonry bond the most commonly used steel connectors were applied: punched steel flat profiles and steel bars. The differences in cracking mechanism and failure as well as in load-bearing capacity of each joint type were demonstrated. Three series of three elements each were tested altogether. Moreover, the code regulations for masonry wall joints were referred and the assortment of connectors for wall joints available on the market was presented.

Key words: masonry structures, stiffening walls, wall joints, connectors, bed joint reinforcement

INTRODUCTION

Control of ULS of structural masonry walls is always performed under the assumption of their spatial co-operation with other structural elements of the building. In case of reinforced concrete floor slabs this assumption is obvious due to the transfer of vertical service loads and bending moments. It is different for the walls at the level of the same storey. In the elements loaded mainly vertically the neighbouring walls influence the stability, determining the effective height of the calculated wall, h_{ef}. In stiffening walls, in turn, this co-operation (or its lack) determines the stiffness of the whole stiffening system, K (cross-sections with or without flange), and consequently the magnitude of the acting loads (horizontal forces N and bending moments M).

The problem of wall joints was neglected and belittled when traditional masonry bonds were used and the joints did not show any signs of damage. Currently, when new types of masonry units and connectors were put to use, damages become common. Design standards require that designers ensure proper co-operation between the crossing walls without giving usable algorithms to control ULS, not only for full bond but also when different types of connectors are used, limiting their recommendations only to constructional conditions. Moreover, empirical procedures for determination of the load-bearing capacity of wall joints have neither been standardised. Therefore, the design of such parts of buildings has commonly intuitive character, not backed up with any tests or detailed analyses. Research material on wall joints is not broad enough to verify normative statements, identify joints behaviour and formulate practical rules for construction taking into account new construction technologies of masonry walls (thin joints, light joints, unfilled head joints, etc.). Research material concerning wall joints is very scarce. Variety of the used test stands and lack of uni-

✉iwona.galman@pol.sl.pl

fied testing procedures makes the comparison of the obtained results practically impossible.

CODE REQUIREMENTS

Eurocode 6 requires that the walls perpendicular or slanting with respect to one another are connected in such a way that vertical and horizontal loads are transferred from one wall to another. This can be achieved by: bonding of the wall (Fig.1a), introduction of connectors (Fig.1b) or anchorage of reinforcement of each wall in the other one (Fig.1c).

It is also recommended that the crossing load-bearing walls are erected simultaneously which ensures proper bonding of the masonry units in their joint plane. General recommendations of EC6 require that the structure and the joints of its main components provide sufficient stability and stiffness both at the erection stage and during operation. Nevertheless, no specific recommendations are given. The same recommendations were also given by the previous Polish standards PN-B-03340:1999/Az1 and PN-B--03002:2007 as well as in ECV-6.

A joint constructed with the use of steel connectors or reinforcement should be equivalent to the masonry bond from the structural point of view and at the same time it cannot impair other parameters of the wall, e.g. thermal or acoustic insulation. The number and spacing of connectors should be determined with static calculations. The designers are expected to compare the calculated values of support reactions with the bearing capacity of the joint (declared by the connectors producer). When the capacity is exceeded, the number of connectors should be increased or double connectors should be used. It must be emphasised that the sought value is the bearing capacity of the joint with the connector, not the capacity of the connector itself.

OVERVIEW OF EXISTING RESEARCH

The test results presented in this paper should be regarded as demonstrative and no quantitative conclusions should be drawn because there are no unified standard procedures to perform these tests.

Quite well-documented tests of walls in which the joint between the stiffening wall and the co-operating wall was simulated has been presented by Capuzzo, Correa and Ramalho (2008). Symmetrical H-shaped models made of ceramic masonry units were tested, without a slab or with a co-operating reinforced concrete slab simulating a part of an RC floor. The walls were connected with a full masonry bond and the loads were transferred to the middle part of the wall, inducing shear in the joint. In addition to the increase in load-bearing capacity, the authors have shown different mechanisms of cracking and failure. In the models without the co-operating slabs the joints failed in the upper part close to the locations where the loads were applied. When an RC slab was cast in the upper part the joints failed along the whole height. The tests Castro, Alvarenga and Silva (2016) were focused on the influence of the type of joint. The H-shaped models were made of concrete masonry units with hollow cores. The elements without the bond and with a full bond were tested. In this case the walls were loaded uniformly at the upper edge to achieve identical magnitude of stresses normal to the bed joints plane. In the models without the bond failure was caused by the loss of stability of the shorter

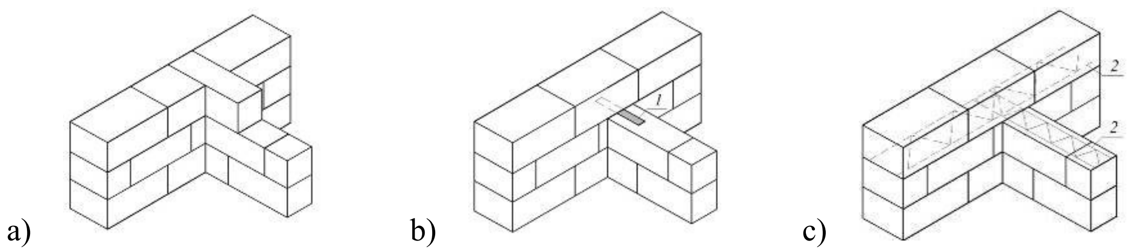


Fig. 1. Wall joints: (a) full joint, (b) with the use of connectors, (c) with the use of longitudinal reinforcement: 1 – connector, 2 – structural type reinforcement

part of the wall. In the models with full bond the joint failed due to shear along the whole height.

The tests of the reinforced joints were also performed. Paganoni and D'Ayala (2014) checked the effectiveness of steel anchors in joining of walls. T shape test elements were designed to model the actual behaviour of the joint in a real structure as closely as possible. Similar tests were performed by Maddaloni, Balsamo, Di Ludovico and Prota (2016a, b). However, in this case the efficiency of innovative clamp anchors was investigated. These clamps were made of carbon fibre rods wrapped longitudinally and spirally with a stainless steel mat. The performed tests have shown high efficiency of both types of joining methods. Application of innovative carbon-fibre connector almost doubled the value of load causing cracking in the joint with respect to the cracking load in the joint without the connector and without the masonry bond. Unfortunately, the results of the tests of sole connectors, and consequently their effectiveness, is hard to interpret because of no reference to the load-bearing capacity of the corner with traditional masonry bond.

It is also worth mentioning Brazilian tests (Corrêa, Moreira and Ramalho, 2009) in which the load-bearing capacities of joints realised with three different methods were compared. These methods were: masonry bond, steel mesh immersed in the bed joint and steel anchor. The tests have shown that the joint with steel elements was able to bear 60% of the load transferred in the wall with masonry bond.

AVAILABLE ASSORTMENT OF CONNECTORS

Depending on the requirements and purpose of use, connectors can be either stiff or flexible. It is also possible to realise total restraint of the walls. It is then required that the connectors can transfer a pair of forces of a specific magnitude. All kinds of connectors – anchors, reinforcement, etc. – must comply with the requirements of PN-EN 845-1 or PN-EN 845-3 standards, in terms of load-bearing capacity, dimensions tolerances and anti-corrosion protection. Table 1 collectively presents the assortment of elements used in Poland to join the walls.

Table 1. Assortment of connectors used for wall joints

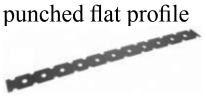
Connector	Notes
punched flat profile 	Element used in joints of walls made of the units of the same height. Replaces masonry bond. Can be used with traditional or adhesive mortars.
Winding flat profile 	Element replacing masonry bond in the walls with thin bed joints.
Punched L-shape profile 	Element for joints of masonry walls with reinforced concrete structure as well as for joints in walls made of the units of different heights.
Two-arm L-shaped profile 	Element for joints of masonry walls with reinforced concrete structure or existing masonry walls, as well as for joints in walls made of the units of different heights.
Punched flat movement profile 	Equivalent of the punched flat profile used in the locations where expansion joint must be introduced between the connected elements.

Table 1 – cont.

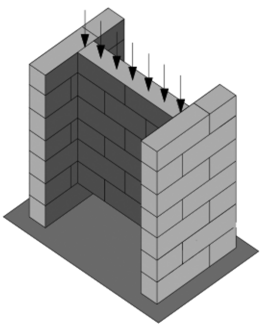
Connector	Notes
Punched L-shape movement profile 	Equivalent of the punched L-shape profile used in the locations where expansion joint must be introduced between the connected elements.
Two-arm L-shaped movement profile 	Equivalent of the two-arm punched L-shape profile used in the locations where expansion joint must be introduced between the connected elements.
truss 	Prefabricated reinforcing beams composed of two parallel flat profiles connected with the sine-shaped rod.
Steel bars 	Bar of the appropriate length assembled in the previously drilled hole.

PRELIMINARY RESEARCH

The pilot tests of masonry wall joints performed by the authors have been presented by Galman and Jasiński (2017). These were the first such experiments in Poland and one of the few in Europe. In these tests the behaviour of three types of wall joints were compared: joints with a traditional masonry bond, with steel L-shaped profiles and with steel two-arm punched flat profiles. Static scheme of the test models and the view of the test stand are shown in Figure 2.

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. Not only was the load-bearing capacity of the joints with steel connectors much lower but also the process of cracking and failure of these joints was completely different. Failure was not preceded with cracking but a sudden increase of displacements accompanied with instant drop of load. Traditional masonry bond had almost 5 times higher load-bearing capacity than the models

a)



b)

Fig. 2. Pilot test model: (a) static scheme; (b) test stand view (Galman & Jasiński, 2017)

with steel L-shaped profiles. The capacity of the joint with flat profile was almost two times higher.

The obtained test results encourage further analyses aiming at detailed identification of the joint behaviour and application of new methods in the construction of joints, with the use of different connectors, greater number of connectors in a given bed joint and optimisation of the flat profiles shape. The performed pilot tests revealed also imperfections of the shape of the test elements and testing technique. Asymmetric failure images of two identical joints did not allow to identify the behaviour of a single joint. Although point loads were applied close to the joint plane, cracks also appeared in the lower part of the web wall, which signified bending of this part of the model and undoubtedly complicated further analyses. Another worrisome phenomenon observed during the tests was variation in deformation of steel connectors depending on the location of the joint with respect to the loaded edge of the web wall which signified uneven work of the joint. That is why in the main phase of the tests the shape of the test elements and the method of load application have been decided to be changed.

MAIN TEST PHASE

Test model and testing technique

It has been decided that in the main phase of the tests the shape of the test elements and the test stand would be changed to avoid the before mentioned imperfections.

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 1,000 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^{-1} 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 3. 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.

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 by Drobiec and Jasiński (2015a), 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 by Drobiec and Jasiński (2015b) was equal to f_{vo} = = 0.31 N·mm^{-2}, and shear modulus, determined acc.

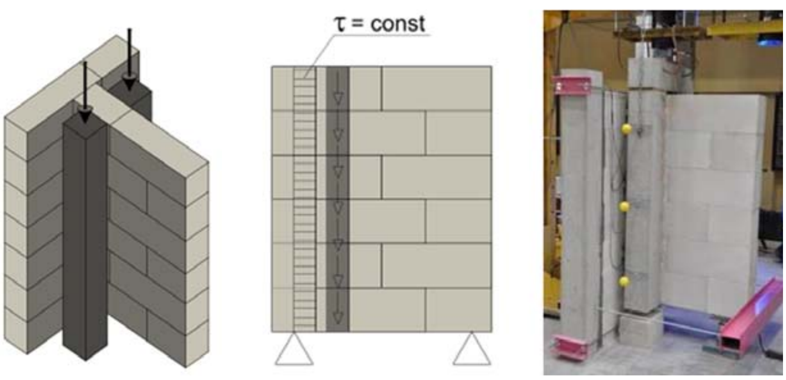


Fig. 3. Scheme and view of the modified testing set-up

to ASTM E519-81 and presented by Drobiec and Jasiński (2015c) was equal to $G = 329\ \mathrm{N \cdot mm^{-2}}$.

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 (Fig. 4a). These were the reference elements which mechanical parameters and behaviour during loading and failure was compared with the results of other tests. In the next two series of walls (geometry acc. to Fig. 4b) the joint between the walls was realised with the use of steel connectors, with no bond of masonry units. In the elements of B series the joint was formed by immersing horizontal single punched flat profiles in bed joints (Fig. 4c). In the elements of F series ø10 steel bars of 36 cm length were applied, anchored in each layer of masonry units (Fig. 4d). Bars were placed into the previously drilled holes across the whole thickness of the flange wall and anchored by 18 cm in the web wall. The bars were additionally stabilised with PUR foam.

The names of the elements with the shapes of connectors are collectively presented in Table 2.

Mechanism of cracking and failure

Character and morphology of cracking depended on the method of joint construction. First visible cracking in the reference wall appeared at the level of around 70% of the force at failure (Fig. 5a), The so formed cracks systematically increased in width (Fig. 5b – cracks are marked in red). Destructive crack ran through the vertical joint and crossed the concrete block (Fig. 5c, 5d).

a b c d

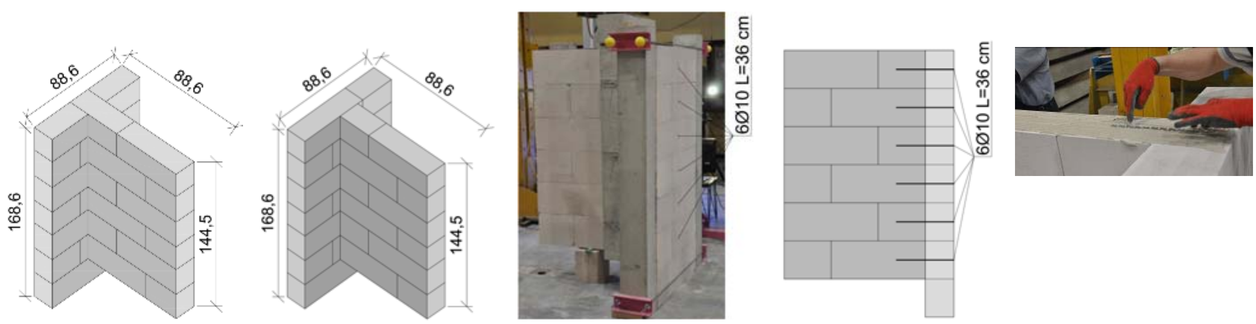
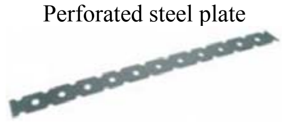


Fig. 4. Geometry: (a) of P type reference masonry wall; (b) masonry walls with steel connectors (wall B and F); (c) method of anchorage with steel punched flat profile; (d) method of connection with steel bar

Table 2. Testing program

Series	Type of joint	No of walls in series
P	Traditional masonry bond	3
B	Perforated steel plate 	3
F	Steel bar φ10 	3

a b c d

Fig. 5. Failure of a P series model: (a) first cracks in the reference model P_2 (locations of maximum deformations are marked on the map in yellow); (b) view of the joint at the moment of failure P_3; (c) view of the joint at the moment of failure P_1; (d) view of failed joint after disassembly of the loaded wall P_2

In the models where the joint was realised with steel elements (B and F series) no development of cracking or damage was observed. Failure was sudden and caused by shear of the joint and visible vertical displacement (by approx. 17 mm) of the web wall (Fig. 6a), which settled on wooden protection. Failure of the models of B series with punched flat profiles was caused by plastification, bending of steel flat profiles in the joint (Fig. 6b). Thanks to the holes in the flat profile there was no slip of the connector in the mortar of bed joints: the mortar penetrating the holes was not sheared but acted as a dowel eliminating movement. The walls of F series, like the walls of B series, failed due to displacements of the loaded wall edge with relative to the unloaded edge. However, in this case there was no plastification of the steel element. Steel bar was pressed into the concrete block under the shear force (Fig. 6c).

a b c

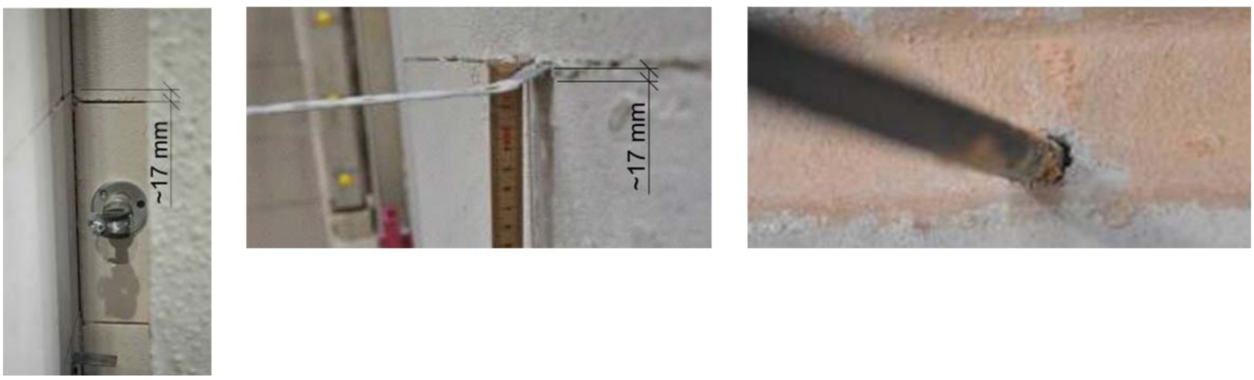


Fig. 6. Failure of B and F series models: (a) view of a damaged model with dimensioned displacements between the bed joints in perpendicular walls (B_1); (b) typical cambers in the steel flat profile within the joint (B_3); (c) view of a bar joining the walls, pulled put after the test (F_2)

Test results

First visible cracks in the reference models (P) appeared under the force of 27–43 kN. Failure occurred under the load of 39–56 kN. Almost three times smaller loads could be taken by the models with steel connectors in the form of punched steel flat profiles (12–24 kN) (B). Comparing the values of the force at failure in the reference models and in the models with a joint with steel bar, almost two times higher load-bearing capacity of the masonry bond was observed.

Collection of results in the form of forces and displacements recorded at the moment of cracking and failure is presented in Table 3. Stiffness of the joint is also given for each model, defined as a quotient of the load transferred to the joints and corresponding displacement.

In addition to the forces causing cracking and failure, an important parameter characterising each joint is its stiffness. By knowing the stiffness it is possible to determine relative displacements of the joined walls under the known load as well as the value of the load for the known relative displacements. The models with traditional masonry bond were characterised with the greatest elastic stiffness. The stiffness of the models with punched flat profiles was almost two times smaller. Finally, the stiffness of the models with steel bars was around 50% smaller. Figure 7 shows the relationship between the force and the mean relative displacement of the loaded and non-loaded wall of all the tested elements.

The force–displacement relationship obtained in the tests of the walls with traditional masonry bonds depicts multi-stage behaviour of the joint. Until cracking displacements developed proportionally to the load, so the behaviour of the joint was elastic. After cracking a subtle bend of the diagram was observed but still displacements increased proportionally to the increasing load (except for the test model P_3). At the moment right before failure there was a sudden increase of displacements at a slight increase of vertical load.

In the models with flat profiles elastic phase of work was unnoticeable. In almost whole range displacements developed non-proportionally. The force–displacement relationship of the models with steel bar was linear in the initial phase. When the level of force was reached causing pressing of the bar into a soft concrete block, a sudden increase of displacements was observed at inconsiderable further increase of loading force.

Table 3. Test results

Test model	Cracking force N_{cr} [kN]	Force at failure N_u [kN]	Displacement at the moment of cracking u_{cr} [mm]	Displacement right before failure u_u [mm]	Joint stiffness $K_t = N_{cr}/u_{cr}$ [MN·m^{-1}]
P_1	27	56	0.06	0.32	450
P_2	43	50	0.12	0.25	358
P_3	31	39	0.08	0.18	388
B_1	24	24	0.12	0.12	200
B_2	16	16	0.05	0.05	320
B_3	12	12	0.09	0.09	133
F_1	25	25	0.13	0.13	192
F_2	28	28	0.07	0.07	400
F_3	26	26	0.08	0.08	325

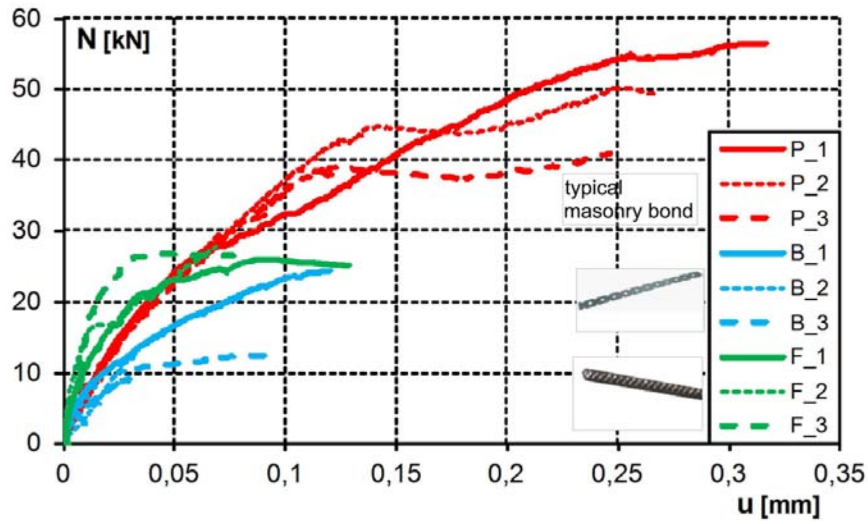


Fig. 7. Relationship between the total force and mean relative displacement of the joint

CONCLUSIONS AND FUTURE PROGRAM

The presented tests are a part of the research performed currently at the Laboratory of Civil Engineering of the Silesian University of Technology in the topic of joints of walls made of AAC blocks. Hereafter are presented only three types of joints: traditional masonry bond, joint with the use of punched flat profiles and steel bars.

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. Not only was the load-bearing capacity of the joints with steel connectors much lower but also the process of cracking and failure of these joints was completely different. No cracking appeared prior to failure but displacements increased suddenly accompanied with instant drop of load.

Further research is currently in progress where it is planned to test the joints with the most popular connectors available on the market as well as with different types of meshes, mats and other elements which would allow for easy and effective connection of the walls. Moreover, it is planned to design a new steel plate which would have an optimum shape to transfer the loads between the joined walls.

Acknowledgements

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

REFERENCES

Capuzzo, N. V., Correa, M.R.S. & Ramalho M.A. (2008). Distribution of vertical loads between interconnected masonry walls with and without a top slab. *14th International Brick & Block Masonry Conference – IBMac.* Sydney [CD-ROM].

Castro, L. O., Alvarenga, R.C.S. & Silva, R. M. (2016). Experimental evaluation of the interaction between strength concrete block walls under vertical loads. *Revista IBRACON de Estruturas e Materiais*, *9*(5) [CD-ROM].

Corrêa, M.R.S., Moreira, E.M.S. & Ramalho, M.A. (2009). Experimental small-scale analysis of the connections between structural clay block work masonry walls submitted to vertical loads. *11th Canadian Masonry Symposium.* Toronto [CD-ROM].

Drobiec, Ł. & Jasiński, R. (2015a). Wpływ rodzaju zaprawy na parametry mechaniczne z ABK poddanych ściskaniu. *Materiały Budowlane*, *4*, 3–7. doi: 10.15199/33.2015.04.02

Drobiec, Ł. & Jasiński, R. (2015b). Wpływ rodzaju zaprawy na parametry mechaniczne murów z betonu komórkowego poddanych ścinaniu. *Materiały Budowlane*, *5*, 106–109. doi: 10.15199/33.2015.05.44

Drobiec, Ł. & Jasiński, R. (2015c). Wpływ rodzaju zaprawy na parametry mechaniczne murów poddanych ścinaniu – odkształcalność postaciowa. *Materiały Budowlane*, *7*, 116–119. doi: 10.15199/33.2015.07.32

Galman, I. & Jasiński, R. (2017). Badanie połączeń ścan murowych. *Materiały Budowlane*, *10*, 94–96. doi: 10.15199/33.2017.10.32

Maddaloni, G., Balsamo, A., Di Ludovico, M. & Prota A. (2016a). 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 [CD-ROM].

Maddaloni, G., Di Ludovico, M., Balsamo, A. & Prota A. (2016b). *Out-of-plane experimental behaviour of T-shaped full scale masonry wall strengthened with composite connections. Composites Part B*, *93*, 328–343.

Paganoni, S. & D'Ayala, D. (2014). Testing and design procedure for corner connections of masonry heritage buildings strengthened by metallic grouted anchors. *Engineering Structures*, *70*, 278–293.

ENV-1996-1-1:2002. Eurocode 6: Design of Masonry Structures. Part 1-1: Common Rules for Reinforced and Unreinforced Masonry Structures. CEN / TC 250.

PN-B-03340:1999/Az1. Konstrukcje murowe zbrojone. Projektowanie i obliczanie.

PN-B-03002:2007. Konstrukcje murowe niezbrojone. Projektowanie i obliczanie.

PN-EN 845-1:2008. Specyfikacja wyrobów dodatkowych do murów. Część 1: Kotwy, listwy kotwiące, wieszaki i wsporniki.

PN-EN 845-3:2008 Specyfikacja wyrobów dodatkowych do murów- Część 3: Stalowe zbrojenie do spoin wspornych. (In Polish).

PN-EN 1052-1:2000. Metody badań murów – Określenie wytrzymałości na ściskanie.

PN-EN 1996-1-1+A1: 2013-05P. Eurokod 6: Projektowanie konstrukcji murowych. Część 1-1: Reguły ogólne dla niezbrojonych i zbrojonych konstrukcji murowych.

POŁĄCZENIA ŚCIAN MUROWYCH

STRESZCZENIE

W artykule zostały przedstawione wyniki badań własnych połączeń ścian wykonanych z autoklawizowanego betonu komórkowego. Badania przeprowadzono na autorskim, zmodyfikowanym stanowisku badawczym. Analizowano morfologię zarysowań i mechanizm zniszczenia, porównano zależności obciążenie–przemieszczenie różnych typów połączeń. Uzyskane rezultaty odniesiono do rezultatów otrzymanych w modelach referencyjnych, które stanowiły modele z klasycznym wiązaniem murarskim. W pozostałych modelach badawczych połączenie zostało ukształtowane z użyciem stalowych łączników. Do połączeń ścian bez wiązania murarskiego wykorzystano najpowszechniej stosowany w Polsce łącznik w postaci blaszek otworowanych oraz stalowy pręt. Wykazano zróżnicowany mechanizm zarysowania i zniszczenia oraz wyraźne różnice w nośności każdego typu połączenia. Łącznie przebadano trzy serię murów po trzy elementy badawcze. Ponadto zaprezentowano ustalenia normowe w zakresie, jakim jest łączenie ścian murowanych, a także przedstawiono podstawowy asortyment dostępny na rynku służący do łączenia ścian.

Słowa kluczowe: konstrukcje murowe, ściany usztywniające, połączenia ścian, łączniki, zbrojenie spoin wspornych