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Shear Capacity of the Zone of Supporting of Precast Lintels Made of AAC

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Shear Capacity of the Zone of Supporting of Precast Lintels Made of AAC

Wojciech Mazur [1] , Radoslaw Jasinski [1], Lukasz Drobiec [1]

[1] Department of Building Structures, Silesian University of Technology, Akademicka 5, 44-100 Gliwice, Poland

Wojciech.Mazur@polsl.pl

Abstract. The paper presents the experimental results and analytical calculations of shear capacity of the ends of precast lintels made of Autoclaved Aerated Concrete (AAC). Three series of elements with that same span but with different dimensions of specimens' cross-section and types of reinforcement were taken into consideration. The lintels were tested in four-point bending test but the mode of damage indicate low shear capacity of lintels. During the tests forces, displacement and cracks propagation were recorded. The analytical calculations of shear capacity of the lintels were conducted according to three codes: PN-EN 1992-1-1:2008, PN-EN 12602, PN-84/B-03264. The analytical results were compared with test results. Significant influence of the method of anchoring of the longitudinal bars on the shear capacity of the beams has been shown. The analytical result of shear capacity gave danger values if the longitudinal bars were not properly anchored. Analytical calculations of longitudinal bars anchorage capacity in AAC precast lintels must be conducted mandatory.

1. Introduction

Lintels are beams that span door and window openings. They are usually made of structural shapes as reinforced concrete elements produced on site, precast or available in the form of precast systemic measures. Lintels are designed to act with the masonry. But at the assembly stage they are usually used as simply supported beams and can be loaded with concentrate forces. Thus, that phase requires the inspection of bending and shear resistance. The issues related to resistance of steel and reinforced concrete elements have been quite well explored. Precast lintels have to meet requirements specified in the standard [1]. They refer to specifications of additional products for masonry, and resistance is calculated and determined from tests in accordance with standards [2, 3, 4].

The reinforcement shape depends the production process, mechanical properties of aerated concrete and the required application of anti-corrosion protection. These aspects cause that resistance of aerated concrete elements is more complex to determine when compared to reinforced concrete elements. This paper describes the destruction mechanism for lintel beams and comparative calculations for the resistance of support zones defined on the basis of the standards [4], EC-2 [5] and [6]. The performed verifying calculations were aimed at drawing primary conclusions about the verification procedures for shear resistance.

2. Test models and testing technique

The experimental tests included three series of elements which differed in geometry, concrete strength, structure of longitudinal and transverse reinforcements, and steel grade. The elements from series A and C had cross-section of 176×240 mm and the total length of 2000 mm. The elements from series D had different cross-section equal to 200×249 mm.
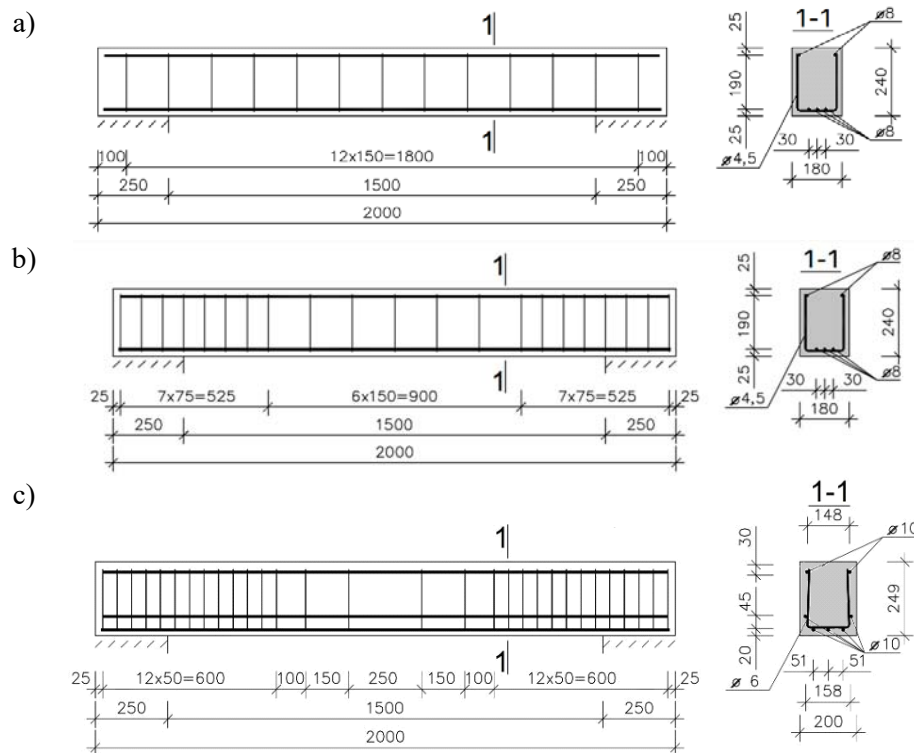

Figure 1. Reinforcement in lintels: a) A series, b) C series, c) D series

All models were symmetrically supported on two supports (movable and fixed) in the axial spacing 1666 mm (according to the standard [4] $l-2 \times a_0 \times 2/3$, where a_0 is the support length on the masonry). Reinforcement in lintels composed of bars connected by welding, and then bent accordingly. All bars in the elements from series A and C were ribbed bars. Longitudinal reinforcement was composed of bars with a diameter of $\phi 8$ mm (two bars in compression zone, and three bars in tensile zone), and transverse reinforcement had bars with the diameter of $\phi 4.5$ mm. Stirrups in lintels from series A had the same span of 150 mm at the whole length of the element (figure 1a). For lintels from series C, stirrups in the support zone had the span reduced by half and equal to 75 mm (figure 1b). In the element from series D, longitudinal reinforcement was composed of smooth bars with a diameter of $\phi 10$ mm (two bars in compression zone, and three bars in tensile zone), and transverse reinforcement had bars, also smooth, with the diameter of $\phi 6$ mm. Spacing of the stirrups in the support zone was 50 mm, and was increasing up to 250 mm in the central part of the element (figure 1c). Reinforcement in lintels of all models was protected against corrosion by a mineral coat (lintels from series A and C) a synthetic coat from plastic (lintels from series D). Anchorage of bars from the longitudinal reinforcement beyond the support axis was 21.5 ϕ in the beams from series A and C, and 17 ϕ in the beams from series D. Thus, the anchorage length was shorter than recommended by a supplier or specified in EC-2 [5], according to which $l_{b, min} =31 \phi$ (series A and C) – 27 ϕ (series D).

The test stand was designed and performed for the tests. It was slightly modified depending on the beam size. The stand scheme and its view with the test model are shown in figure 2. The lintels were put coaxially with the steel frame on the supports. A hydraulic actuator was used to exert load on the

beam through the steel crossbeam. The load F was recorded using a dynamometer. The crossbeam distributed the load into a pair of concentrated forces which were applied at 1/4 of the distance between axes of the supports.
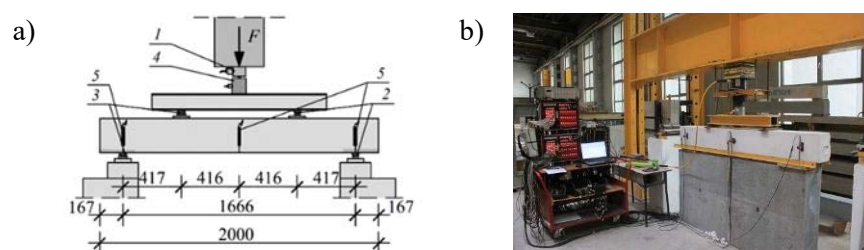
Figure 2. Test stand: a) diagram; *1* – hydraulic actuator, *2* – fixed support, *3* – movable support, *4* – dynamometer, *5* – inductive converter of displacement, b) view

The beams were monotonically loaded at an increase in the force every 5 kN to record cracks. During the tests, the force and deflection were measured and recorded using an automatic test stand (ATS). Cracks in the elements were also controlled.

Specimens for material tests of the element from the series A, C and D were collected after conducting tests on the lintel. The specimens were taken from undamaged parts of the support. The detailed material tests on the elements from series A and C are described in the paper [8]. Tests on compressive strength of aerated concrete were conducted according to [9]. They were performed on three blocks ($100 \times 100 \times 100$ mm). The determined strength was f_c=4.19 MPa, (v=32.2%) in the elements from the series A and C, and f_c=4.75 MPa, (v=11.4%) in the element form the series D. Bars in the longitudinal reinforcement were tested in accordance with the standard [10]. For the beams from the series A and C, the following values were obtained: $R_m/R_{p0,2}$=612/566=1.08=f_{tk}/f_{yk}=1.08, and for the element from the series D: $R_m/R_{p0,2}$=575.3/535.7=1.07<f_{tk}/f_{yk}=1.08, which classified steel to class B according to EC-2 [5]. Relatively short bars in the transverse reinforcement (< 300 mm) excluded strength tests. Thus, the parameters for steel in the longitudinal reinforcement (identical ribbing or no ribbing) were applied for further analyses.
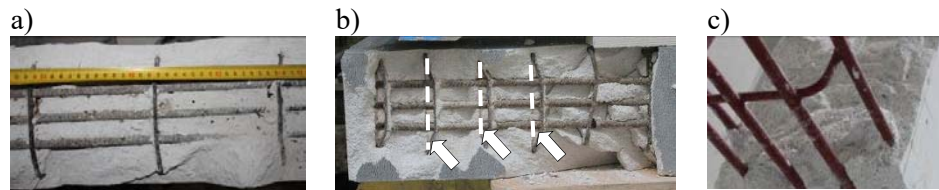
3. Test results

The cracking mechanism was the same in the beams from the series A and C. Flexural cracks were formed as first (figure 3a). They appeared in the span and reached ¾ height of the element. An increase in loading caused diagonal cracks in the shear zone figure 3b. They were running through almost the whole height of the beam. Further increase in loading developed diagonal cracks in the bottom area of the beam figure 3c. Prior to the destruction, they joined horizontal cracks which were rapidly formed in the longitudinal reinforcement – figure 3d.

Each time destruction was observed at one of the supports where, apart from diagonal cracks, horizontal cracks in the bottom area of the longitudinal reinforcement appeared. They were caused by anchorage failure in the bars. No anchorage failure was observed in the longitudinal bars in the beam from the series D. But the support failure occurred like in the beams from the series A and C. In the anchorage zones of bars in the beams from the series A and C, transverse bars (stirrups) connected with longitudinal bars were damaged – figure 4a, figure 4b (bow height marked with a dashed line in figure 4b). And in the beams from the series D, no failure was noticed in the uncovered reinforcement (figure 4c). Figures. 5-7 illustrate cracks in all the tested beams.
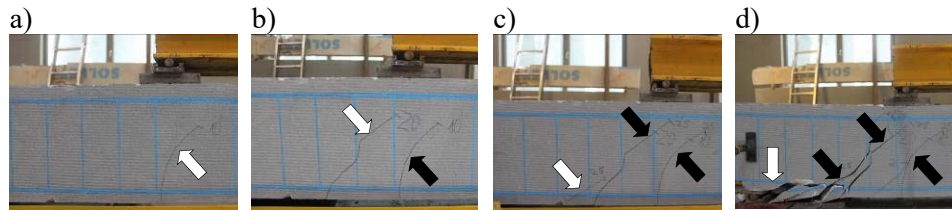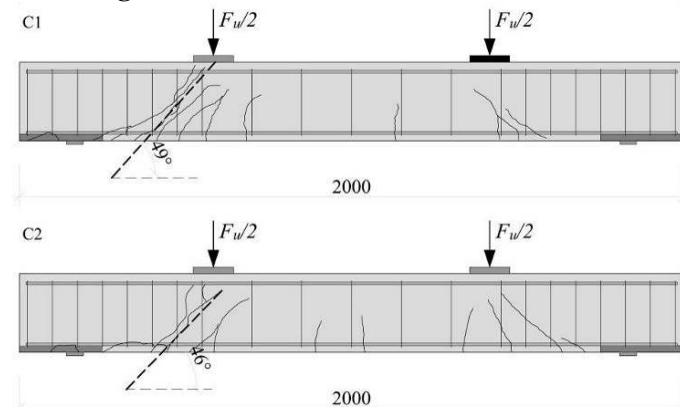
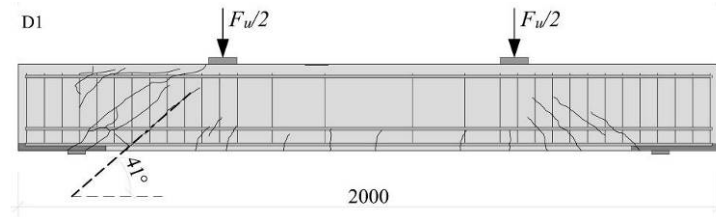**Figure 3.** Crack propagation in the lintel C2 during loading: a) 2, b) 4, c) 5, d) 6

Figure 4. Anchorage of reinforcement after the tests: a) A series, b) C series, c) D series
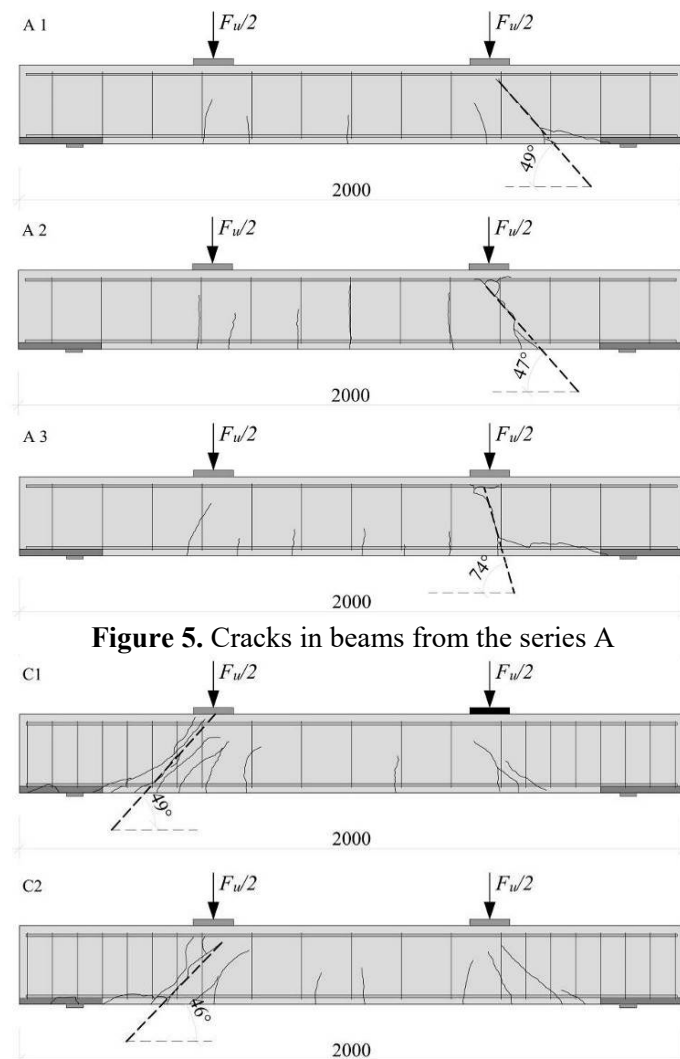
Figure 5. Cracks in beams from the series A

Figure 6. Cracks in beams from the series C

Figure 7. Cracks in beams from the series D

4. Analysis of test results

The pictures of cracks in the supports were used to determine inclination of concrete struts $\Theta_{test} = 41°\div74°$ (cracks developed before the loss at anchorage capacity). Also, the corresponding values were calculated $ctg(\Theta_{test}) = 0.29\div1.15$. Taking into account strength parameters of lintel materials, the values $ctg(\Theta_{cal})$ were calculated from the following relationship (1) according to [7]. For the analysis of the elements from the series C and D, the value $ctg(\Theta_{cal}) = 0.85$ was applied. It was lower than the value recommended by [5] from the range expressed as (2).

$$ctg\Theta_{cal} = \sqrt{\frac{f_{ck}v}{\rho_w R_m} - 1} \qquad (1)$$

$$1,0 \leq ctg\Theta_{cal} \leq 2,0 \qquad (2)$$

Design shear resistance of the supports V_R determined on the basis of EC-2 [5] and the standard for calculations for prefabricated reinforced components of autoclaved aerated concrete PN-EN 12602 [4]. Both standards specify the resistance condition for concrete struts in the same way. The difference is in determining shear resistance of reinforcement in the form of stirrups. Shear resistance of the supports V_R was specified as the minimum resistance value of struts $V_{R2,cal}$ and tensile stirrups $V_{R3,cal}$ from the relationships (3a) and (3b) according to EC-2 [5].

$$V_{R,test}(ctg(\Theta_{test})) = \min(V_{R2,test}, V_{R3,test}) \qquad (3a)$$

$$V_{R,cal}(ctg\Theta_{cal}) = \min(V_{R2,cal}, V_{R3,cal}) \qquad (3b)$$

The test results are compared with the shear resistance of the reinforcement in the support V_{R3} according to the standard [4]. For comparison purposes, shear resistance for the section reinforced with stirrups Q_{sb} was calculated according to PN-84/B-03264 [6]. The calculated results are shown in table 1, and the comparison of calculated and test results is presented in table 2.

Table 1. Calculated values of transverse destructive forces

Series	Element	$ctg(\Theta_{test})$	$ctg\Theta_{cal}$	$V_{R2,cal}$ [kN]		$V_{R3,cal}$ [kN]		V_{R3} [kN]	Q_{sb} [kN]
				$ctg(\Theta_{test})$	$ctg\Theta_{cal}$	$ctg(\Theta_{test})$	$ctg\Theta_{cal}$		
A	1	0.87	1.56	39.9**	36.6	20.3**	36.6	14.8	32.7
	2	0.93	1.56	40.2**	36.6	21.8**	36.6	14.8	32.7
	3*	0.29*	1.56	21.3*	36.6	6.7*	36.6	14.8	32.7
C	1	0.87	0.85	39.9**	39.7	40.7**	39.7	20.8	53.7
	2	0.97	0.85	40.2**	39.7	45.2**	39.7	20.8	53.7
D	1	1.15	0.85	49.8	49.6	141.5	104.5	42.7	106.9

* - neglected value due to real inclination of struts that do not meet the standard requirements [5]
** - hypothetical value of the shear force applied in the further analysis while neglecting the anchorage failure of the longitudinal reinforcement

Table 2. Comparison of calculated and test results

Series	Element	$V_{u,test}$	$V_{R,test}(ctg(\Theta_{test}))$ acc. to (3a)	$V_{R,cal}$ $(ctg\Theta_{cal})$ acc. to (3b)	$\dfrac{V_{u,test}}{V_{R,test}}$	$\dfrac{V_{u,test}}{V_{R,cal}}$	$\dfrac{V_{u,test}}{V_{R,3}}$	$\dfrac{V_{u,test}}{Q_{sb}}$
			[kN]					
A	1	11.2	20.3	36.6	0.55	0.31	0.75	0.34
	2	13.1	21.8	36.6	0.60	0.36	0.88	0.40
	3	13.8	6.7*	36.6	2.06*	0.38	0.93	0.42
C	1	14.5	39.9	39.7	0.36	0.37	0.70	0.27
	2	13.5	40.2	39.7	0.34	0.34	0.65	0.25
D	1	44.1	49.8	49.6	0.89	0.89	1.03	0.41

Table 2 presents the values of destructive forces $V_{u,test}$ determined from the tests. They included self-weight of beams and steel equipment The lowest force was determined during the truss methods [4], [5], at Θ_{test} and Θ_{cal} (except for the element A3) from the resistance of steel ties – stirrups. Values of shear resistance of the lintels (at Θ_{test}), calculated according to the truss method, were higher by 11%÷66% than the values determined from the tests, except for the lintel A3. Shear resistance was increasing at $ctg\Theta_{cal}$. For calculations made according to[4], shear resistance of second order sections decreased, but it was still higher by 7%÷25% than destructive forces determined in the tests. And in the element D1, the calculated value was lower by 3% than the empirical value. The obtained results indicated the extent to which shear resistance was overestimated assuming the truss model of destruction with the proper anchorage of the tension reinforcement. The highest overestimation of shear resistance by 59%÷75% was obtained using the shear calculation method according to PN-84/B-03264 [6]. That method included the model with the section of destruction, and also assumed the proper anchorage of the longitudinal reinforcement. While designing the shear elements, $ctg\Theta$ within the range <1 – 2.5> can be applied in accordance with the standard [5]. The value $ctg\Theta > 2$ was not possible for the tested lintels due to the length of shear section from the support to the point of application. Table 3 presents the calculated values of resistance with reference to the values of the force determined from the tests at boundary values $ctg\Theta = 1$ and $ctg\Theta = 2$.

Table 3. Calculated values of transverse destructive forces

Series	Element	$V_{u,test}$	$V_{R,2ctg\Theta}$	$V_{R,3ctg\Theta}$	$V_{R,2ctg\Theta}$	$V_{R,3ctg\Theta}$	$\dfrac{V_{u,test}}{V_{R,2}\cot\Theta}$	$\dfrac{V_{u,test}}{V_{R,3}\cot\Theta}$	$\dfrac{V_{u,test}}{V_{R,2}\cot\Theta}$	$\dfrac{V_{u,test}}{V_{R,3}\cot\Theta}$
		[kN]	$ctg\Theta = 1$		$ctg\Theta = 2$		$ctg\Theta = 1$		$ctg\Theta = 2$	
A	1	11.2	40.3	32.2	23.4	46.8	0.28	0.35	0.48	0.24
	2	13.1	40.3	32.2	23.4	46.8	0.32	0.41	0.56	0.28
	3	13.8	40.3	32.2	23.4	46.8	0.34	0.43	0.59	0.30
			average:				**0.31**	**0.40**	**0.54**	**0.27**
C	1	14.5	40.3	32.2	46.8	93.6	0.36	0.45	0.31	0.15
	2	13.5	40.3	32.2	46.8	93.6	0.34	0.42	0.29	0.14
			average:				**0.35**	**0.44**	**0.30**	**0.15**
D	1	44.1	50.2	40.2	123.0	245.9	0.88	1.10	0.36	0.18

Resistance of AAC beams in the elements from the series A and C at the strut inclination $ctg\Theta = 1$ was higher by 65%-69% than the test value. Resistance of concrete strut V_{R2} was critical. A longer section subjected to shearing and the applied value $ctg\Theta = 2$ reversed the tendency. The reinforcement resistance V_{R3} was critical for the resistance, which was then overestimated by 73% – 85%. For the beam with the largest longitudinal reinforcement, without the anchorage failure of the longitudinal reinforcement, we achieved the highest compatibility between the test results and the calculations at the highest inclination of the struts $ctg\Theta = 1$. Because of the struts, resistance was lower by 12% than the test value. At the highest inclination of the struts and $ctg\Theta = 2$, overestimation of the resistance was the highest. The above results indicate that design algorithms for the elements from the series A and C could lead to significant and dangerous overestimation of shear resistance. It is the consequence of not fully used transverse reinforcement as the truss in the support zone has not been completely

formed. Therefore, shear resistance in such elements should be always verified after controlling anchorage of the longitudinal reinforcement even if all construction conditions have been met. As protective coating on the reinforcement is required, the longitudinal reinforcement is properly anchored in the AAC beams if the transverse reinforcement (stirrups) is applied.

According to the standard [4], we verified the design anchorage of the longitudinal reinforcement, neglecting the reinforcement adhesion to aerated concrete, but taking into account the transverse reinforcement in the form of stirrups. On the basis of destruction image of lintels, for each series we applied a number of anchorage stirrups n_t. Then, anchorage capacity of transverse bars F_{RA} was determined, and the corresponding transverse force $V_{uRA,cal}$ was calculated. The calculated results were compared with the highest values of transverse force $V_{u,test}$ in the beams from each series. The calculated results are presented in table 4.

Table 4. Comparison of values of shear resistance determined from the reinforcement anchorage and the performed tests

Series	Element	n_t	F_{RA} [kN]	$V_{uRA,cal}$ [kN]	$V_{u,test}$ [kN]	$\dfrac{V_{uRA,cal}}{V_{u,test}}$
A	3	3	16.5	7.7	13.8	0.56
C	1	6	33.5	15.7	14.5	1.08
D	1	4	97.3	44.2	44.1	1.00

The smallest difference between the design transverse force caused by anchorage and the real transverse force was observed in case of the lintels D1 and C1. In the beam from the series D1, the transverse force caused by anchorage corresponded to destructive force determined from the tests. In the beam from C1 series, the force $V_{uRA,cal}$ was higher by 8% than the non-destructive force determined from the tests. The best estimation of the beam resistance was made for the lintel A3, where the transverse force caused by the anchorage was lower by 44% than the destructive force $V_{u,test}$. The obtained results suggest that the verification of shear resistance of AAC beams, calculated as simply supported elements that did not act with the masonry unit (and without load applied to the bottom and top surface of the beam), is not satisfactory and can lead to dangerous results. Similarly, taking the anchorage of longitudinal reinforcement which meets the standard requirements as satisfactory condition for applying the truss method of calculation seems to be dangerous. Calculating shear resistance for such types of the elements (without hooks or loops) should be always preceded by verification of anchorage capacity taking into account all transverse bars at the supports.

5. Conclusions

All the tested lintels were destroyed at the supports. Diagonal cracks in the elements from the series A and C were developed before the anchorage failure of the longitudinal reinforcement. If the anchorage conditions for the longitudinal reinforcement are met, resistance can be verified by the truss model. Verifying calculations were made using three methods and indicated the significant overestimation of resistance when compared to the results determined from the tests. The most similar results were obtained using the standard [4] for the AAC elements. According to the standard recommendations, at strut inclination tgΘ =1 in the beams from the series A and C, resistance was clearly overestimated, and the most similar calculations for shear resistance were obtained for the beam from the series D. Due to the type of beam destruction caused by bond failure, the additional verifying calculations were made to determine the transverse force from the anchorage capacity. In that case, the calculated values of transverse force were either lower (A series) by 44% or slightly higher (C series) by 8% than the empirical destructive forces, or even the same (D series).

To sum it up, the truss models used for verifying the resistance of ACC beams can produce dangerous results in contrast to the properly designed reinforced concrete elements subjected to

bending, where such models usually provide good results. It is caused not only by low tensile strength of aerated concrete, but also reduced bond strength caused by protective coats (mineral or plastic). Even stirrups welded to the longitudinal reinforcement caused the anchorage failure of the longitudinal reinforcement, which in that case did not act as the anchorage transverse reinforcement. For the tested beams, we obtained the satisfactory results for the resistance by determining the transverse force from the reinforcement anchorage. For such a complex mechanism of beam destruction at the supports, which included primary diagonal cracking and secondary destruction of anchorage, verifying the anchorage resistance and simultaneously neglecting bond strength (regardless of the length of the bar anchorage) is the required and satisfactory condition for satisfactory estimation of the beam resistance.

Acknowledgements

References

[1] PN-EN 845-2:2013-10: Specification for ancillary components for masonry. Part 2: Lintels.

[2] PN-EN 846-9:2002 Methods of test for ancillary components for masonry. Part 9: Determination of flexural resistance and shear resistance of lintels.

[3] PN-EN 1356:1999: Performance test for prefabricated reinforced components of autoclaved aerated concrete or lightweight aggregate concrete with open structure under transverse load.

[4] PN-EN 12602+A1:2013-11: Prefabricated reinforced components of autoclaved aerated concrete.

[5] PN-EN 1992-1-1:2008/NA:2010 Eurocode 2: Design of concrete structures. Part 1-1: General rules and rules for buildings.

[6] PN-84/B-03264: Plain, reinforced and prestressed concrete structures - Analysis and structural design.

[7] M. Knauff, K. Klempka, ,"Algorithms and desing charts for shear design according to Eurocode 2", *Inżynieria i Budownictwo* Nr 2/98, pp. 78–80, 1998.

[8] W. Mazur, Ł. Drobiec, R. Jasiński, ,,Research of Light Concrete Precast Lintels", *Procedia Engineering,* Vol. 161/2016, pp. 611-617, 2016.

[9] PN-EN 679:2008: Determination of the compressive strength of autoclaved aerated concrete.

[10] PN-EN ISO 6892:2010 Metallic materials -- Tensile testing -- Part 1: Method of test at room temperature.