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SHEAR RESISTANCE OF LOW HEIGHT PRECAST CONCRETE LINTELS

A. AMBROZIAK [1], J. KONDRAT[2], M.WESOŁOWSKI [3]

The scope of the paper is to investigate analytically and determine experimentally the shear resistance of low height reinforced precast concrete lintels. The chosen procedures included in national and international standards applied for the design of structural concrete elements to an estimation of shear behaviour of reinforced concrete elements are described. The characteristic and designed shear strength of precast concrete lintels are determined and compared with experimentally obtained results. The shear resistance for precast concrete lintels was determined by laboratory tests according to a European standard. The assessment of the in-situ compressive strength of concrete in precast concrete lintel is specified. The designed compressive strength class is confirmed. The real reinforcement distribution is verified to assess the wide scatter of experimentally obtained failure forces. A short literature outlook of the papers concerning investigations on lintels and shear resistance of concrete is given also. The paper can provide scientists, engineers, and designers a theoretical and experimental basis in the field of precast concrete lintels shear resistance.

Keywords: precast concrete lintel; reinforced concrete beam; shear resistance; mechanical tests

[1] DSc., PhD., Eng., Prof. GUT, Gdansk University of Technology, Faculty of Civil and Environmental Engineering, St. Gabriela Narutowicza 11/12, 80-233 Gdansk, Poland, e-mail: ambrozan@pg.edu.pl, ORCID: 0000-0002-7735-7863

2 MSc., Eng., Gdansk University of Technology, Faculty of Civil and Environmental Engineering, St. Gabriela Narutowicza 11/12, 80-233 Gdansk, Poland, jarko@pg.edu.pl

3 PhD., Eng., Gdansk University of Technology, Faculty of Civil and Environmental Engineering, St. Gabriela Narutowicza 11/12, 80-233 Gdansk, Poland, e-mail: wesolow@pg.edu.pl, ORCID: 0000-0002-8273-2066

1. INTRODUCTION

Beams formed above the windows and doors openings to span openings are called lintels. There are many different materials and shapes are used to make of lintels, e.g.: timber, stone, brick, metal, concrete, ect.. Timber and stone lintels are generally rectangular in cross-section. Steel lintels are often designed as rolled structural steel sections. Concrete lintels are usually either rectangular, U-shaped, L-shaped, etc. Generally, two types of concrete lintels, reinforced or pre-stressed, are commonly used in civil engineering. Erecting masonry walls force the application of prefabricated lintels to maintain convenient and quick to cover openings. They are delivered to the construction site in specific, typification dimensions and lengths. A newly designed type of lintels and applications of new materials (e.g. hybrid materials) implies new investigations and laboratory tests. The practical design of reinforced masonry lintels in bending and shear is outlined in [1]. Memari et al. [2] described the experimental data of autoclaved aerated concrete (AAC) lintels strengthened with externally bonded glass FRP (fiber-reinforced polymer). Stavroulaki and Liarakos [3] investigated the effects of contact mechanisms in dynamic analysis between the lintels and the masonry. Mazur et al. [4] examined lintels made of AAC with an interacting wall of varying heights. Nowak [5] described results and analysis of experimental tests performed on brick lintels with different shapes. Neculai et al. [6] described an analysis of precast hybrid structural elements. Drobiec [7] investigated stress-strain analysis in the window zone area of full-scale wall models made of AAC and calcium silicate units. It can be seen that the subject of lintels is still taken into consideration in many engineering and scientific investigations. The different types of lintels are tested and new laboratory tests are performed to specify their behaviours and mechanical properties under different design loads.

The research aims to estimate the shear resistance of low height reinforced precast concrete lintels. The authors collect and described chosen standards procedures used to design structural concrete elements to an estimation of the shear strength of concrete members. To verification of design values of shear strength, the shear resistance laboratory tests were carried out. The assessment of the in-situ compressive strength of concrete is performed on cylindrical specimens taken from precast lintels after laboratory tests by a concrete core borehole diamond drill machine. A short literature survey concerns shear resistance of concrete is given also. The investigation was a part of an expert opinion on the bearing capacity and shear resistance of reinforced precast concrete lintels and the possibilities of carrying design loads and having a proper working life.

2. SHEAR RESISTANCE

In the early 1900's Ritter [8] and Mörsch [9] proposed classical truss analogy for shear design of reinforced concrete beams. In the 1960s Leonhardt and Walther [10] expanded truss analogy. Godycki-Ćwirko [11] described the state of knowledge about shear in reinforced concrete and given recommendations and design guidelines. At the latter time, many empirical formulas and theories are existed and developed for estimating the shear resistance of structural concrete elements. Hofer and Mc Cabe [12] studied the shear strength of reinforced concrete beams and discussed various building code shear prediction methods. Bentz et al. [13] summarized the results of over one hundred pure shear tests on reinforced concrete elements and presented a simplified analysis method to predict the shear strength. Paczkowski and Nowak [14] reviewed the available database and shear models for reinforced concrete beams without web reinforcement. Schnell and Thiele [15] suggested an introduced reduction of shear load capacity due to embedded ducting by reduction factor depend on the dimension of openings. Krejsa et al. [16] described the model uncertainty related to the shear resistance of reinforced concrete beams with stirrups. Słowik [17] discussed the influence of longitudinal reinforcement on shear capacity of reinforced concrete members without the shear reinforcement. Grandić et al. [18] reviewed theoretical background and shear design methods of reinforced concrete beams. Kovács [19] performed shear tests on reinforced concrete beams reinforced with polypropylene and steel fibres and compared experimental results to the shear model of fib Model Code 2010 [20]. Afrifa and Adom-Asamoah [21] described the shear behaviour of 26 reinforced concrete beams made of phyllite and granite aggregates and compared results with chosen shear design standards equations. Samora et al. [22] analysed the complementary mechanisms of the shear strength of lattice beams of reinforced concrete frames without transverse reinforcement Revision of the shear design equations for concrete members reinforced with FRP bars without stirrups is performed in [23]. Nadir et al. [24] proposed a compression stress field-based model to assess the shear strength of slender concrete beams without web reinforcement. Kim et al. [25] investigated wide beams where the steel plates with openings were used as shear reinforcement.

The literature concerning concrete shear resistance is very extensive. Challenges, recent developments and possibilities of shear strength are discussed in [26]. Nevertheless, it can be concluded, that new theoretical and experimental investigations are still developed to understanding complex shear behaviour. In the next part of this chapter, the chosen procedures included in national and international standards applied for the design of structural concrete elements to an estimation of shear behaviour of reinforced concrete elements are collected and described.

2.1. EN 1992-1-1 EUROCODE 2

The EN 1992-1-1 standard [27] defines the shear strength of concrete members V_{Rd} as the shear resistance of concrete $V_{\mathrm{Rd,c}}$ for sections without designed shear reinforcement and as the shear resistance of shear reinforcement $V_{\mathrm{Rd,s}}$ only for the section with designed shear reinforcement:

$$V_{\mathrm{Rd}} = V_{\mathrm{Rd,c}} \quad , \quad V_{\mathrm{Rd}} = V_{\mathrm{Rd,s}} \,. \tag{1}$$

The design value for the shear resistance of concrete $V_{\mathrm{Rd,c}}$ without axial force is determined as:

$$V_{\mathrm{Rd,c}} = \min\left[\left[C_{\mathrm{Rd,c}} \cdot k\left(100 \cdot \rho_l \cdot f_{\mathrm{ck}}\right)^{1/3}\right] \cdot b_w \cdot d; \quad v_{\min} \cdot b_w \cdot d\right], \tag{2}$$

where f_{ck} is characteristic compressive cylinder strength of concrete at 28 days in MPa, $k = 1 + \sqrt{200/d} \le 2$ (with d in mm), $\rho_l = A_{sl}/b \cdot d \le 0.02$ is reinforcement ratio for longitudinal reinforcement, $C_{\mathrm{Rd,c}} = 0.18/\gamma_{\mathrm{C}}$, $\gamma_{\mathrm{C}} = 1.5$ is partial safety factor for concrete (in the Polish National Annex for EN 1992-1-1 [27] standard is equal to 1.4) and

$$v_{\min} = 0.035 \cdot k^{3/2} \cdot f_{\mathrm{ck}}^{1/2} \,. \tag{3}$$

For members with designed vertical shear reinforcement, the shear resistance V_{Rd} is expressed as:

$$V_{\mathrm{Rd}} = V_{\mathrm{Rd,s}} = 0.9 \cdot \frac{A_{\mathrm{sw}}}{s} \cdot d \cdot f_{\mathrm{ywd}} \cdot \cot\theta \le V_{\mathrm{Rd,max}} = 0.9 \cdot \alpha_c b_w \cdot d \cdot \nu \cdot f_{\mathrm{cd}} / \left(\cot\theta + \tan\theta\right), \tag{4}$$

where: A_{sw} is the cross-sectional area of the shear reinforcement, s is the spacing of the stirrups, $f_{\mathrm{ywd}} = f_{\mathrm{ywk}}/\gamma_{\mathrm{S}}$ is the design yield strength of the shear reinforcement ($\gamma_{\mathrm{S}} = 1.15$, is partial safety factor for reinforcing steel), θ is angle between concrete compression struts and the main tension chord, $f_{\mathrm{cd}} = f_{\mathrm{ck}}/\gamma_{\mathrm{C}}$ is design value of concrete compressive strength, and factor ν is calculated as:

$$\nu = 0.6 \cdot \left[1 - \left(f_{\mathrm{ck}}/250\right)\right], \ (f_{\mathrm{ck}} \text{ in MPa}). \tag{5}$$

The recommended limits of the angle θ in EN 1992-1-1 standard [27] are stated as $1 \le \cot\theta \le 2.5$. Starting from the criterion of minimal shear reinforcement, the $\cot\theta$ can reach a maximum value of 3.00, see e.g. [28]. This limit value is adopted in the German standard DIN-1045 [29], while in the Polish National Annex for EN 1992-1-1 [27] standard it is limited to 2.00.

2.2. ACI 318 BUILDING CODE

The shear strength at the section of concrete beams according to ACI 318 Building Code ([30], [31]) is calculated as the sum of the concrete's shear strength V_{c}^{ACI} and the shear reinforcement V_{s}^{ACI}:

$$V_{\mathrm{n}}^{ACI} = V_{\mathrm{c}}^{ACI} + V_{\mathrm{s}}^{ACI}. \tag{6}$$

The shear resistance provided by the shear reinforcement (vertical stirrups) is calculated as:

$$V_{\mathrm{s}}^{ACI} = A_{sw} f_{ywk} d/s. \tag{7}$$

According to ACI 318-14 Code [30], the shear strength provided by concrete for non-prestressed members without axial force can be calculated in the simplified method from the following equation:

$$V_{\mathrm{c}}^{ACI} = \lambda \cdot \left(\sqrt{f_{\mathrm{ck}}}/6\right) \cdot b_w \cdot d, \tag{8}$$

where λ is modification factor ($\lambda=1$ for normal weight concrete).

In a detailed method the ACI 318-14 Code [30] provides a more complex equation for determination of shear strength provided by concrete:

$$V_{\mathrm{c}}^{ACI} = \begin{pmatrix} \left(0.16 \cdot \lambda \cdot \sqrt{f_{\mathrm{ck}}} + 17\rho_l \dfrac{V_u d}{M_u}\right) \cdot b_w \cdot d \\ \left(0.16 \cdot \lambda \cdot \sqrt{f_{\mathrm{ck}}} + 17\rho_l\right) \cdot b_w \cdot d \end{pmatrix} \le 0.29 \cdot \lambda \cdot \sqrt{f_{\mathrm{ck}}} \cdot b_w \cdot d, \tag{9}$$

where V_u and M_u is factored shear force and factored moment at section (bending moment occurs simultaneously with shear force at the section considered).

The ACI 318-19 Code [31] introduced a new form of Eq. (9) which included the effect of member depth, commonly referred to as the "size effect" ($\lambda_s = \sqrt{2/\left(1+\left(d/250\right)\right)} \le 1$, d in mm) and the effects of the longitudinal reinforcement ratio on shear strength in the form:

$$V_{\mathrm{c}}^{ACI} = \left(0.67\lambda_s \lambda \left(\rho_w\right)^{1/3} \sqrt{f_{\mathrm{ck}}}\right) \cdot b_w \cdot d \le 0.4\lambda\sqrt{f_{\mathrm{ck}}} \cdot b_w \cdot d \tag{10}$$

In the ACI 318-19 Code [31] Eq. (9) is provided also as simpler option for calculation of the V_{c}^{ACI}.

2.3. MODEL CODE 2010

The design shear resistance V_{Rd}^{MC} according to fib Model Code 2010 [20] is determined as the sum of the shear resistance provided by the concrete and the shear reinforcement:

$$V_{\mathrm{Rd}}^{MC} = V_{\mathrm{Rd,c}}^{MC} + V_{\mathrm{Rd,s}}^{MC} \le V_{\mathrm{Rd,max}}^{MC} . \tag{11}$$

The design shear resistance provided by the concrete is specified as:

$$V_{\mathrm{Rd,c}}^{MC} = 0.9 \cdot k_v \cdot \left(\sqrt{f_{\mathrm{ck}}} / \gamma_{\mathrm{C}} \right) \cdot b_w \cdot d , \tag{12}$$

where k_v is strength reduction factor for concrete cracked in shear.

The design shear resistance attributed to the shear reinforcement is determined as:

$$V_{\mathrm{Rd,s}}^{MC} = 0.9 \cdot {}^{A_{sw}}\!/_{s} \cdot f_{ywd} \cdot d \cdot (\cot\theta + \cot\alpha) \cdot \sin\alpha , \tag{13}$$

where θ is angle defined by the different level of approximation and indicates the angle of principal compressive stress in the web, while α is the angle of the stirrups or bended bars from the beam axis.

The design shear resistance cannot exceed the crushing capacity of the concrete defined as:

$$V_{\mathrm{Rd,max}}^{MC} = 0.9 \cdot k_c \cdot \left(\sqrt{f_{\mathrm{ck}}} / \gamma_{\mathrm{C}} \right) \cdot b_w \cdot d \cdot \frac{\cot\theta + \cot\alpha}{1 + \cot^2\theta} . \tag{14}$$

where k_c is strength reduction factor for concrete cracked in compression and is determined according to the levels of approximation as: $k_c = k_\varepsilon \cdot \eta_{fc} = k_\varepsilon \cdot (30/f_{\mathrm{ck}})^{1/3}$, factor $\eta_{fc} \le 1$.

The design shear resistance for Level of Approximation I can be calculated according to the following assumption: $\theta_{\min} = 30^\circ$, $k_\varepsilon = 0.55$ and

$$k_v = 180/(1000 + 1.25 \cdot 0.9 \cdot d) \; (d \text{ in mm}). \tag{15}$$

It can be shown that when $d \to 0$ (in Eq. (15)) the strength reduction factor $k_v \to 0.18$. The value 0.18 is corresponding to the characteristic value of the factor $C_{\mathrm{Rd,c}}$ in Eq. (2).

2.4. POLISH STANDARDS

The Polish standard PN-B-03264:2002 [32] specified the design value for the shear resistance of concrete without axial force as:

$$V_{\mathrm{Rd,c}} = \left[0.35 \cdot k \cdot f_{\mathrm{ctd}} \left(1.4 + 40\rho_l\right)\right] \cdot b_w \cdot d \leq 0.45 \cdot \nu \cdot f_{\mathrm{cd}} \cdot b_w \cdot d\,, \tag{16}$$

where f_{ctd} is design value of tensile strength of concrete, k is reduction factor $k = 1.6 - d \geq 1.0$ (d in meter), ν factor according to Eq. (5).

For members with designed vertical shear reinforcement, the shear resistance is calculated as:

$$V_{\mathrm{Rd,s}} = 0.9 \cdot \left(A_{\mathrm{sw}} \cdot f_{\mathrm{ywd}} / s\right) \cdot d \cdot \cot\theta \leq 0.9 \cdot \nu \cdot f_{\mathrm{cd}} \cdot b_w \cdot d \cdot \left(\cot\theta / 1 + \cot^2\theta\right), \tag{17}$$

The early version of PN-B-03264:1984 standard [33] specified the shear resistance of concrete as:

$$V_{\mathrm{Rd,c}} = 0.75 \cdot f_{\mathrm{ctd}} \cdot b_w \cdot d\,, \tag{18}$$

where $f_{\mathrm{ctd}} = \alpha_{\mathrm{ct}} \cdot f_{\mathrm{ctk}} / \gamma_{\mathrm{C}} = \alpha_{\mathrm{ct}} \cdot 0.7 \cdot f_{\mathrm{ctm}} / \gamma_{\mathrm{C}} = \alpha_{\mathrm{ct}} \cdot 0.21 \cdot f_{ck}^{2/3} / \gamma_{\mathrm{C}}$, α_{ct} is reduction factor, f_{ctk} is characteristic value of tensile strength of concrete, f_{ctm} is mean value of tensile strength of concrete.

The multiplier 0.75 in the Eq. (18) is connected with the assumption of minimum value of shear force ($0.5 \cdot f_{\mathrm{ctk}} \cdot b_w \cdot d = 0.5 \cdot \gamma_{\mathrm{C}} \cdot f_{\mathrm{ctd}} \cdot b_w \cdot d = 0.75 \cdot f_{\mathrm{ctd}} \cdot b_w \cdot d$).

2.5. BRAZILIAN STANDARD

The design shear resistance V_{Rd} according to ABNT NBR 6118 [34] is specified as:

$$V_{\mathrm{Rd}} = V_{\mathrm{Rd,c}} + V_{\mathrm{Rd,s}}\,. \tag{19}$$

The shear resistance of concrete is determined as:

$$V_{\mathrm{Rd,c}} = 0.60 \cdot f_{\mathrm{ctd}} \cdot b_w \cdot d\,, \tag{20}$$

and the shear resistance with vertical shear reinforcement is calculated as:

$$V_{\mathrm{Rd,s}} = 0.9 \cdot \left(A_{\mathrm{sw}} / s\right) \cdot d \cdot f_{\mathrm{ywd}}\,. \tag{21}$$

3. MATERIAL AND METHODS

The investigated reinforced precast concrete lintels of 120cm length have 12cm high (*h*) and 18cm width (*b*) and are designed to make of C20/25 concrete strength class, see Fig. 1. The minimum characteristic cylinder strength ($f_{ck,cyl}$) and cube strength ($f_{ck,cube}$) for C20/25 compressive strength according to EN 206 [35] class is 20MPa (N/mm^2) and 25MPa, respectively. The characteristic compressive strength is specified at 28 days. The designed longitudinal reinforced A_{s1} (bottom) is equal 0.85cm^2 (3#6) and A_{s2} (upper) is equal to 0.28cm^2 (1#6). The reinforcement ratio for longitudinal reinforcement ρ_l=0.51% ($A_{s1}/b \cdot d = 0.85/18 \cdot 9.2$). The nominal cover c_{nom}=2cm with permitted design deviation of cover equal to 5mm is specified. On the cross-section (see Fig. 1) the distances from the edges to the main longitudinal rebars are given. Characteristic yield strength of longitudinal reinforcement f_{yk} is equal to 500MPa (grade B500A steel) and the design yield strength of the shear reinforcement f_{ywd} is equal to 235MPa (grade S235JR steel). The designed spacing of the vertical triangle stirrups is *s*=7.5cm and 15cm and the dimension of #5mm. The precast concrete lintels were manufactured and delivered to the laboratory by a local precast factory plant. The laboratory test was performed in the Laboratory of Department of Concrete Structures at the Faculty of Civil and Environmental Engineering, Gdańsk University of Technology.

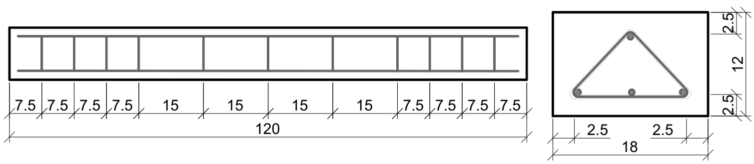

Fig. 1. View and lintel cross section - designed reinforcement distribution

3.1. LABORATORY TESTS

The shear resistance for precast concrete lintels was determined by laboratory tests according to EN 846-9 [36] standard by means of the computer-controlled Zwick testing machine. To carry out the laboratory tests the three lintels 120cm length (L120, see Fig. 1) were simply supported and subjected to vertical load. The number of lintels fulfil EN 845-2 [37] standard requirement for a minimum number of samples per test. The load diagram, steel supports (width *b*=100mm) applied in laboratory tests are shown in Fig. 2. All tests were performed at room temperature (about 20^{o}C). The force-controlled tests were carried out up to failure. The force held every 4kN in its waiting time when the evolution of crack was monitored. In Fig. 3 the lintels front view with visualization of cracks after the failure and the angles θ of the inclination of the dominant shear crack are also indicated. Two main forms of failures have occurred: flexural failure under force and shear failure, see Fig. 3. The diagram of compressive force versus displacement of movable crosshead up to rupture of the lintels is shown in Fig. 4. The values of failure forces obtained in the laboratory tests are shown in Table 1. The result of the mean value is presented as a sum of mean values and standard error of the mean of the specified range. The shear resistance according to EN 845-2 [37] standard is the mean shear load at which failure of a sample of lintels specimens occurs. Nevertheless, the shear resistance shall be greater than or equal to 50% of the declared value of load-bearing capacity. Comparison obtained results (see Table 1), it can be noted that a significant decrease in the failure force is observed between individual tests (e.g. L2 lintel). To explain such a big difference in the failure force values the real reinforcement distribution has to verify.

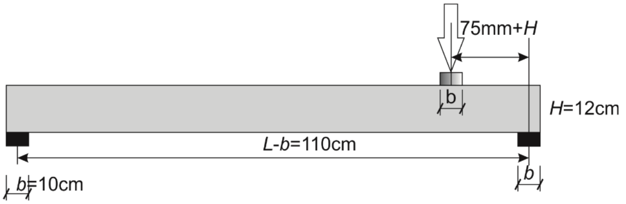

Fig. 2. Test arrangement for shear at end support

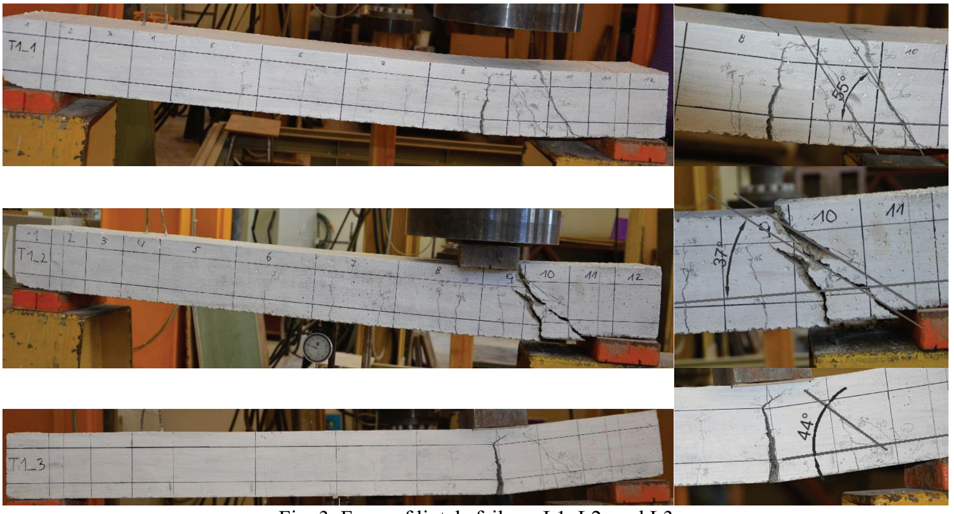

Fig. 3. Form of lintels failure: L1, L2, and L3

Table 1. Failure force of precast lintels

Lintel number	failure force [kN]	form of failure
L1	42.8	flexural failure occurs under force
L2	35.9	shear failure occurs
L3	42.2	flexural failure occurs under force
mean	**40.3 ± 2.2**	

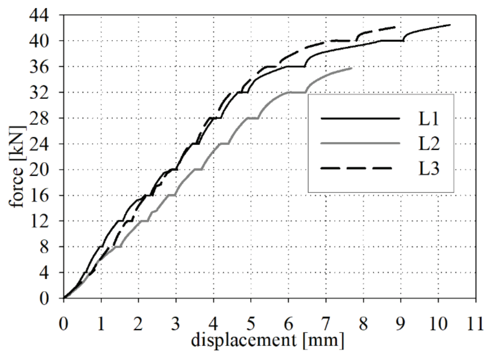

Fig. 4. Vertical force versus displacement up to rupture of lintels

To determine the mechanical properties of concrete the cylindrical specimens were taken from the non-cracking part of precast lintels after laboratory tests by a concrete core borehole diamond drill machine. The cylindrical samples were prepared from the exploratory boreholes with diameter D equal to approximately 50 mm and length L equal to approximately 50 mm (length to core diameter ratio L/D=1). The dimensions of the concrete cores were taken according to standard EN 12504-1 [38], where the preferred length/diameter ratios are 1.0 if the strength results are to be compared to the cube strength of $15 \times 15 \times 15$ cm concrete specimens. On the other hand, the ASTM C31 standard [39] states that the cylinder length shall be twice the diameter (this requirement is often impossible to fulfil) and diameter shall be at least 3 times the nominal maximum size of the coarse aggregate for a concrete structure.

The uniaxial experimental tests used the Advantest 9 C300KN mechanical testing machine. The experiments were performed to the failure of the concrete cylinder specimens and used a constant rate of loading with a range of 0.6 MPa/s according to EN 12390-3 [40]. Uniaxial tensile test results of compressive strength and dry density are presented in Table 2. The mean dry density value is equal to 2275 ± 8.5 kg/m^3. According to the EN 206 standard [35], the concrete can be categorized as normal concrete when the dry density range from 2000 to 2600 kg/m^3. The ACI 318-19 standard [31] indicates normal-weight concrete with a density between 2160 and 2560 kg/m^3.

The dimensions of the core specimens made require that the effect of core diameter on core specimen strength should be taken into account. The EN 12504-1 standard [38] state that 100 mm diameter cores ($f_{c,cycl\ 100}$) were approximately 7% stronger than 50 mm diameter cores ($f_{c,cycl\ 50}$). Brunarski and Dohojda [41] highlight that for the cored specimens with a diameter of 50 mm taken from C20/25 or higher-class concretes, the conversion factor to the diameter of 100 mm of 1.1 is justified. When the cylinder strength $f_{c,cycl\ 100}$ is known the cube strength $f_{c,cube}$ can be specified as $f_{c,cube} = f_{c,cycl\ 100}$.

Table 2. Compressive strength and dry density

	$f_{is,cycl\ 50}$ [MPa]	$f_{is,cube}$=$f_{is,cycl\ 100}$= 1.07 $f_{is,cycl\ 50}$ [MPa]	dry density [kg/m^3]
1	29.0	31.0	2297
2	30.2	32.3	2280
3	32.4	34.7	2258
4	29.2	31.2	2267
mean	**30.2 ± 0.8**	**32.3 ± 0.8**	**2275.5 ± 8.5**

The estimated characteristic in-situ compressive strength in equivalent strength of a 150 cube $f_{ck,is,cube}$ according to EN 13791 [42] is the lower value of:

$$f_{ck,is,cube} = \min\left\{f_{m(n),is} - k, \quad f_{is,lowest} + 4\right\} = \min\left\{32.3 - 7, \quad 31 + 4\right\} = 25.3\text{MPa}, \tag{22}$$

where $f_{m(n),is}$ is the mean in-situ compressive strength of *n* (in present investigation *n*=4) test results, $f_{is,lowest}$ is the lowest in-situ compressive strength test results, *k* is the margin depends on the number of tests results (*k*=7 for tests results equal to 4).

The minimum characteristic in-situ cube compressive strength is in the range: $26\text{MPa} > f_{ck,is,cube} = 25.3\text{MPa} \geq 21\text{MPa}$ that C20/25 compressive strength class according to EN 13791 [42] can be confirmed. The in-situ mechanical properties of concrete sometimes are highly different form designed state - especially for old concrete elements, see e.g. [43].

Following, the spacing of stirrup and concrete cover of longitudinal reinforcement was verified. The precast concrete lintels were demolished and the measure of cover and reinforcement distribution was identified. The comparison of designed and real reinforcement distribution is drawn in Fig. 5. The visualization of cracks after the lintels' failure and the dominant cracks (bolded lines) are also indicated. It can be shown, that the real assembly of reinforcement is variable and is not strictly as a designed state (compare Fig. 5 and Fig. 1). The deviation of bottom covers for the L2 specimen reaches 17mm (and it is 3 times more than the design deviation of cover). Additionally, it is identified a lack of reduction of stirrup spacing to 7.5cm close to supports. The real mean stirrup spacing close to supports $\overline{s_{is}}$ is equal to about 12.1cm and it is 1.6 times more than design. The wide scatter in stirrup spacing and concrete covers resulted in very low-quality control during the manufacturing process of precast concrete lintels. A lack of appropriate covers decreases protection from environmental conditions through what highly reduces the durability of reinforced concrete elements. The shear cracks distribution (see Fig. 5 and 3) confirms the necessity of reduction of stirrup spacing to 7.5cm close to supports.

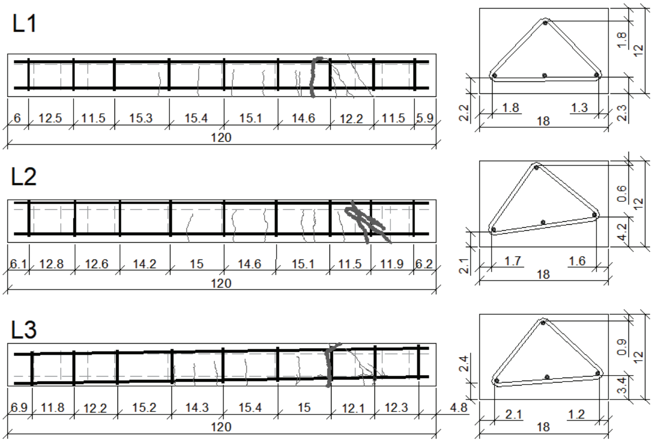

Fig. 5. View and cross section of real reinforcement distributions and crack after failure

3.2. Design shear resistance

Based on the described in the previous chapter chosen standard procedures used in the design process of concrete elements the design and characteristic values of shear resistance contributed to concrete $V_{\mathrm{Rd,c}}$, vertical stirrup $V_{\mathrm{Rd,s}}$, and shear strength V_{Rd} are calculated and collected in Table 3. In the calculations, the value of in-situ compressive strength of concrete $f_{\mathrm{is,ck}}$=25.8MPa ($f_{\mathrm{is,cube}}$=32.3MPa multiplied by 0.8) and the real mean stirrup spacing close to support $\overline{s_{is}}$ =12.1cm are taken into account. The value of the $\cot\overline{\theta} = 1$ are accepted according to the mean dominant shear crack apparent during laboratory tests ($\overline{\theta} = 45°$, see Fig. 3). It can be observed, that during the shear resistance tests over the vertical load value of 32kN (see Fig. 4) the force-displacement curves are highly nonlinear. Besides the nonlinear character, a large increment of displacement occurs to the failure of lintels. For the L1 and L3 lintels, the maximal crack width under the load of 32kN did not exceed 0.4mm. The value of 0.4mm is the limitation of the maximum design crack width for X0, XC1 exposure class specified by EN 1992-1-1 [27]. Before the comparison is made the impose a vertical load of 32kN should be recalculated to the shear force that occurred in the cross-section. It can be determined, that the maximal shear force calculated from the free supported beam scheme (see Fig. 2) under 32kN vertical load occurred in cross-section of the lintel is equal about V=25.76kN. This value can be treated as the safe limit of the shear force which can be carried by investigated precast concrete low height lintels – experimentally specified shear strength. The characteristic shear strength value calculated for the ACI 318-14 [11] code (21.06kN) and for the PN-B-03264:1984 [33] standard (22.80kN) are nearly the same, see Table 3. The lowest values of the characteristic shear strength is determined for the EN 1992-1-1 [27] standard (14.10kN) and ACI 318-19 [12] (16.75kN). The highest values for the characteristic shear strength are calculated for the ABNT NBR 6118 [34] standard (24.56kN) and the PN-B-03264:2002 [32] standard (25.75kN) and are closest to value of the shear force in cross-section (V=25.76kN) of the low high lintel. These computed shear strengths can be treated as a proper approximation of laboratory established values of shear resistance for investigated low height precast concrete lintels.

Table 3. Shear strength of precast lintels

standard		concrete contribution $V_{Rd,c}$ [kN]	shear reinforcement contribution $V_{Rd,s}$ [kN]	shear strength V_{Rd} [kN]
EN 1992-1-1 [27]	γ_C=1.5, γ_S=1.15 ($\cot\theta=1.0$)	9.40 (Eq. 2) *14.10**)	5.50 (Eq. 4) *6.33**)	9.40 *14.10**)
	γ_C=1.4, γ_S=1.15 ($\cot\theta=1.0$)	10.07 (Eq. 2) *14.10**)	5.50 (Eq. 4) *6.33**)	10.07 *14.10**)
ACI 318-14 [11]	γ_C=1.0, γ_S=1.0 ($\cot\theta=1.0$)	14.03 (Eq. 8)	7.03 (Eq. 7)	21.06
ACI 318-19 [12]	γ_C=1.0, γ_S=1.0 ($\cot\theta=1.0$)	9.72 (Eq. 10)	7.03 (Eq. 7)	16.75
fib Model Code 2010 [20], LoA I	γ_C=1.5, γ_S=1.15 ($\cot\theta=1.0$)	8.24 (Eq. 12) *12.36**)	5.50 (Eq. 13) *6.33**)	13.74 *18.69**)
PN-B-03264:2002 [32]	γ_C=1.5, γ_S=1.15 ($\cot\theta=1.0$)	17.16 (Eq. 16) *25.75**)	5.50 (Eq. 17) *6.33**)	17.16 *25.75**)
PN-B-03264:1984 [33]	γ_C=1.4, γ_S=1.15 ($\cot\theta=1.0$)	16.28 (Eq. 18) *22.80**)	5.50 (Eq. 17) *6.33**)	16.28 *22.80**)
ABNT NBR 6118 [34]	γ_C=1.5, γ_S=1.15 ($\cot\theta=1.0$)	12.16 (Eq. 20) *18.23**)	5.50 (Eq. 21) *6.33**)	17.66 *24.56**)

*) characteristic values calculated for γ_C=1.0, γ_S=1.0

The design process of new reinforced concrete elements specified by standards has to comply with the principles and requirements for the safety and serviceability of structural elements and structures. Each national standards incorporate specific theoretical background used to describe phenomena proceeded in reinforced concrete structural elements. Therefore, the calculated design values of the shear strength are widely scattered, see Table 3. Nevertheless, the specified design shear strength possesses necessitate reserve of safety to fulfils the ultimate limit state and serviceability limit state.

4. DISCUSSION AND CONCLUSIONS

The main objective of the present investigation was to assess the shear resistance of low high precast concrete lintels. Based on the performed experimental and analytical investigation the following conclusions may be drawn:

- The mean dry density of concrete cores is equal to 2275 ± 8.5 kg/m^3. The in situ concrete specimens were classified as normal concrete according to the EN 206 standard [35] and also ACI 318-19 standard [31].
- The C20/25 compressive strength class, according to EN 13791 [42], can be confirmed for in-situ concrete. The value of characteristic in-situ compressive strength of concrete is specified as equal to $f_{is,ck}$=25.8MPa.
- The low high reinforced precast concrete lintels had a high scatter of stirrup spacing and concrete covers. The wide scatter in stirrup spacing and concrete covers resulted in very low-quality control during the manufacturing process. The poor quality may be explained also by production technology. The manufacturer should be verified a Factory Production Control procedure (details of extent, nature, and frequency of testing and controls).
- The EN 845-2 [37] standard requirement for a minimum number of samples per test should be verified and increased to a minimum of 6.
- The characteristic shear strength calculated on the basis of the ABNT NBR 6118 [34] (24.56kN), and the PN-B-03264:2002 [32] (25.75kN) standards are closest to value of the shear force in cross-section (V=25.76kN) of the investigated experimentally low high reinforced precast concrete lintels.

The paper can be treated as a possible base for new investigations. Given shear resistance of low high reinforced precast concrete lintel undergoing shear resistance tests, it is possible to plan new detailed investigations of these elements. The obtained results encourage the authors to continue the research, on the basis of extended shear resistance tests for reinforced precast concrete lintels to a detailed description of the behaviour of low high concrete elements under shear cyclic and rheological loads. The authors are hopeful that the described investigation sparks interest a wide group of engineers and scientists to take into consideration the subject of low height precast concrete lintels.

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ODPORNOŚĆ NA ŚCINANIE PREFABRYKOWANYCH NADPROŻY O NISKIEJ WYSOKOŚCI

Keywords: prefabrykowane nadproża betonowe, belka żelbetowa, nośność na ścinanie, testy wytrzymałościowe

STRESZCZENIE

Celem artykułu jest weryfikacja i określenie oporności na ścinanie prefabrykowanych nadproży o niskiej wysokości. We wstępie dokonano przeglądu literatury z zakresu badań elementów stosowanych do przykrycia otworów drzwiowych i okiennych w budownictwie. Zwrócono uwagę, iż tematyka badawcza związana z nadprożami jest ciągle aktualna i podejmowana przez środowisko naukowe i inżynierskie. W kolejnym rozdziale zebrano i opisano wybrane normowe procedury określania odporności na ścinanie elementów żelbetowych. Opis normowych procedur poprzedzono przeglądem literatury z zakresu ścinania w elementach żelbetowych. Określono charakterystyczne i obliczeniowe nośności na ścinanie prefabrykowanych nadproży betonowych według opisywanych procedur normowych i porównano je z wynikami uzyskanymi eksperymentalnie. Wskazano, iż możliwe jest oszacowanie odporności na ścinanie niskich belek nadprożowych na podstawie wybranych procedur normowych.

Badania eksperymentalne belek nadprożowych prowadzone były na podstawie europejskiej normy opisującej metody określania odporności na ścinanie nadproży. Wyznaczono eksperymentalnie średnią siłę niszczącą badanych belek nadprożowych. Wykonano odwierty rdzeniowe, na podstawie których określono rzeczywistą wytrzymałość betonu na ściskanie. Na tej podstawie potwierdzono klasę wytrzymałości betonu na ściskanie. Przeprowadzono weryfikację rzeczywistego rozkładu zbrojenia w belkach nadprożowych dokonując odkrywek zbrojenia w badanych elementach.

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