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COMPOSITE ACTION OF PRECAST HOLLOW CORE SLABS WITH STRUCTURAL TOPPING

ZESPOLENIE STRUNOBETONOWYCH PŁYT KANAŁOWYCH Z NADBETONEM

Abstract

There is no interface reinforcement in composite floors made of HC slabs covered with structural topping. In such structures, the preparation of the top surface of the precast element has a major influence on longitudinal shear strength. The recommendations of various codes concerning the bearing capacity of the non-reinforced joint in concrete composite structures are presented in this paper. The results of the authors' own computational analysis have been compared with the results of experimental tests carried out by different research institutions. Calculations according to Eurocode 2 underrate the value of the bearing capacity. A much stronger conformity of the results was achieved for the calculations carried out according to previously existing standards or according to information given in Model Code 2010.

Keywords: *composite action, composite structures, hollow core slab, longitudinal shear, structural topping*

Streszczenie

W stropach zespolonych, wykonywanych z płyt HC z nadbetonem konstrukcyjnym, nie stosuje się zbrojenia zszywającego w złączu. W tych konstrukcjach przygotowanie powierzchni górnej prefabrykatu ma zasadniczy wpływ na nośność styku na ścinanie. W artykule przedstawiono zlecenia różnych dokumentów normowych w zakresie nośności niezbrojonego złącza w betonowych konstrukcjach zespolonych. Wyniki własnych analiz obliczeniowych porównano z rezultatami badań doświadczalnych prowadzonych w różnych ośrodkach naukowych. Obliczenia według Eurokodu 2 dają zaniżone wartości nośności. Dużo lepszą zgodność obliczeń z wynikami badań uzyskuje się dla obliczeń prowadzonych według wcześniej obowiązujących przepisów lub informacji zawartych w Model Code 2010.

Słowa kluczowe: *konstrukcje zespolone, nadbeton konstrukcyjny, płyta kanałowa, ścinanie podłużne, zespolenie*

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1. Introduction

The casting of a structural layer of reinforced concrete topping on the surface of prestressed HC slabs is a structural procedure often applied in engineering practice. The positive influence of concrete topping on the ultimate and serviceability limit states of hollow core slabs might be significant if full monolithism is developed between the topping and the precast slabs. The presence of the concrete topping on the upper surface of slabs not only improves the load bearing capacity of the floor structure and allows for better cooperation between adjacent slabs (e.g. in the case of transmission of concentrated or linear loadings) but it also improves the diaphragm action of the floor. Topping reinforcement participates in the carrying of negative bending moments (either intentional or unintentional) in the supports zones. Since the concrete topping layer improves the dynamic characteristics of the structure, this type of structural solution is commonly used in countries where seismic loadings represent a threat to the safety of the precast structures.

Experimental research carried out in various scientific centres all over the world demonstrated that the concrete topping makes a major contribution in increasing the bending moment capacity of HC slabs (increase of approximately 25% [6, 11, 23]) and the shear capacity (in case of the bearing on the rigid supports, increase of approximately 35% [2, 22, 23]). The application of a concrete topping positively influences on the behaviour of the slabs in a serviceability limit state by increasing both the flexural stiffness of the floor and its cracking moment by approximately 15% [18]. This positive influence of structural topping application is apparent in the analysis of the fire resistance of HC slab floors [3] and in the improvement of the acoustic characteristics of these floors. Concrete topping is also one of the most preferable means of levelling the upper surface of floors constructed with precast elements which have different cambers due to prestressing.

It should be emphasised that the current HC slab production technologies (extrusion or slip-forming) do not allow for placing any transverse reinforcement in the cross-section of the slab, including the interface reinforcement. If the reinforcement crossing the interface between the topping and the precast element is necessary, it could be placed in the longitudinal joints between adjacent precast HC elements during the floor execution stage.

The concrete topping is most frequently implemented as a layer not thinner than 40 mm (usually 40–80 mm) with anti-shrinkage mesh reinforcement. Considering fire safety of HC slabs, it is advised that the thickness of the concrete topping in the mid-span should not exceed 50 mm, and the reinforcement of the topping layer in the support zone should not be greater than ø 6 mm every 150 mm in the longitudinal direction of the slab [15]. It is recommended that the concrete class of this layer should not be lower than C20/25.

The analysis of the bearing capacity of the interface between the concrete topping and the precast element under longitudinal shear in the aspect of design guidelines and international experimental research is considered in this paper. In actuality, this is a very complex problem, since on the behaviour of this interface influence not only the magnitude and type of both direct actions (for example operational load) and indirect actions (for example shrinkage and creep of concrete), but also number of the factors connected with the execution of the structure (for example the material characteristics of concrete, shaping of the joint interface, its moistening, contamination, possible cracking, presence of laitance etc.) Factors having an effect on the quality of HC slabs with topping composite action are graphically presented in Fig. 1 [6].
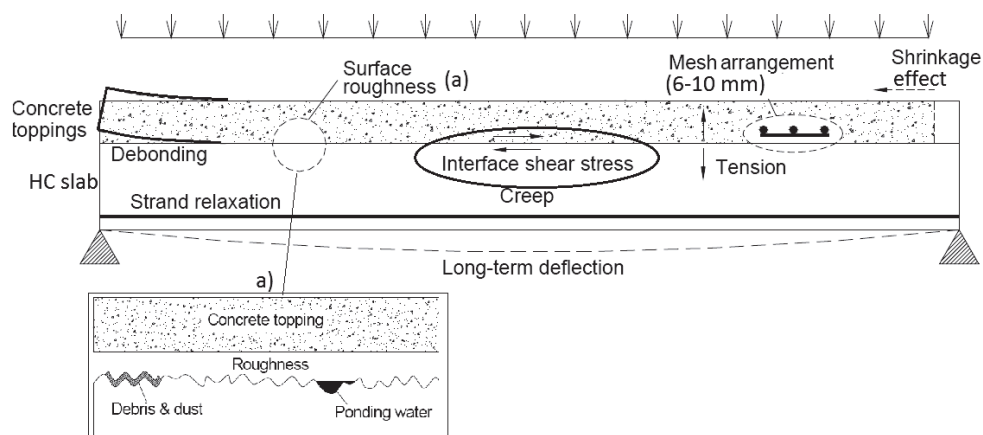
Fig. 1. Factors influencing the composite action of the HC slab with topping [6]

Effectiveness of the adhesive bond forces between the precast element and an in-situ concrete topping depends also from the preparation of the surface of the precast element and the degree of filling of the roughness of the old concrete with the grout of the new concrete mix.

The basic parameter which characterises the surface of the precast element is the average roughness R_a – this represents the medium deviation of the surface profile from the medium line (Fig. 2).
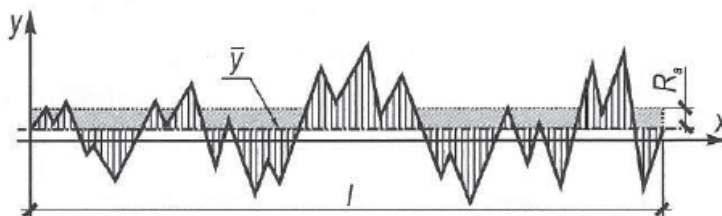

Fig. 2. Definition of the average roughness

In particular code regulations, the roughness of the precast element is defined in different ways. In [10] the classification due to the roughness measured by means of the engineering method (a sand patch method), easily applicable in the building site conditions, has been assumed. Four categories are distinguished:
– very smooth – for a non-measurable R_t;
– smooth – for $R_t < 1.5$ mm;
– rough – for $1,5$ mm $\leq R_t < 3.0$ mm;
– very rough – for $R_t \geq 3.0$ mm.

In EN 1992-1-1 [10], which will henceforth be referred to as EC2, and in many other codes (for example PN-B-03264 [20]), four different types of surface have been introduced.

These are defined by means of the type of the technological procedures applied at the stage of precast element execution:

− very smooth surface – obtained in steel moulds, plastic moulds or especially prepared wooden moulds;
− smooth surface – a slip-formed or extruded surface, or a free surface left without further treatment after vibration;
− rough surface – surfaces having at least 3 mm of unevenness at a spacing of about 40 mm, obtained by raking and aggregate exposure;
− indented.

Execution mistakes at the stage of the production, i.e. inappropriate methods of obtaining the rough surface of the precast element, might cause the formation of micro-damage to this surface and contribute to a reduction of the bearing capacity of the joint. Contaminations of the joint surface such as air bubbles and water puddles resulting from the improper moistening of the precast element surface or the segregation of the concrete topping grout may lead to a significant decrease in adhesion [6]. Sawdust, hydraulic oil, dirt and construction debris must be cleaned off prior to casting the topping [17]. In practice, the methods of casting and compacting the topping layer of the concrete and further maintenance are of major importance, together with the atmospheric conditions during each of these procedures. Research [5] has shown that the proper compacting of the topping concrete layer (using a vibrator) can increase the joint strength by up to 100% higher than in the case of poor compaction (for example, with use of a steel rod). The influence of all those execution aspects of concrete topping casting, on the later behaviour of the whole structure, is very often underrated by the designers and contractors.

2. Analytical model of composite action

In composite structures consisting of two different concretes which were cast at different times, with no existing transverse reinforcement crossing the interface, the proper work of the structure is possible only as a result of the occurrence of the bonding forces between the two elements. These forces arise from physicochemical phenomena, i.e. adhesive bond and friction at the contact surface.

Adhesion is characterised by the properties of combined materials. At the time of the setting of topping concrete, chemical reaction between the components of the fresh cement grout and the anhydrated cement particles in the precast element occur [13]. Penetration of the new concrete mix into local unevenness of the precast element surface results in the occurrence of the mechanical bond in the form of aggregate interlock mechanism [24].

The occurrence of external stresses perpendicular to the joint interface and pressing adjacent surfaces, σ_N, is connected with the occurrence of the friction force in the interface surface, the magnitude of which depends on the friction coefficient μ.

In the classical model of contact surface behaviour used to determine of the interface shear strength $\tau_{Rd,j}$ under the stresses σ_N, the Coulomb hypothesis is adjusted as a basis for the majority of standardised methods. This hypothesis assumes that the bearing capacity of the joint depends on so-called coherence (being the function of adhesion factor c and tensile strength of the weaker concrete f_{ctd}) and friction, characterised by the coefficient μ:

$$\tau_{Rd.j} = c \cdot f_{ctd} + \mu\sigma_n \le 0.5 \cdot \nu \cdot f_{cd} \tag{1}$$

where:

f_{cd} – is the design compress strength of the concrete,

f_{ctd} – is the design tensile strength of the concrete.

The expression given in the standards describing tangent shear stresses at the interface of the composite elements is determined according to the basic rules of the strength of the materials proportional to the shear force. These stresses can be calculated from the expression:

$$\tau_{Ed} = \frac{\beta \cdot V_{Ed}}{z \cdot b} \tag{2}$$

where:

β – is the ratio of the longitudinal force in the topping area and the total longitudinal force in the compression zone, calculated for the section considered;

z – is the lever arm of the composite cross-section (usually assumed as $z = 0.8d$, where d is the effective height of the cross-section);

b – the width of the interface;

V_{Ed} – design shear force.

When calculating the bending strength of the joint in composite structures, the interface itself is usually treated as being only under shear action – this is apparent in the construction of the code formulas, including only tangent stresses from external loadings However, if we assume that the interface is parallel to the longitudinal axis of the element and analyse the trajectory of the principal stresses, it can be proved that the interface is also under the action of normal stresses. Magnitude and notation of those stresses depend not only on the position of the analysed cross-section at the length of the element, but also on the position of the interface along the height of the cross-section.

European standard EN 1168+A3:2011 [7], concerning precast hollow core floor slabs, assumes the possibility of the occurrence of two models of failure of the composite cross-section as a result of exceeding tensile strength for the following shear actions:

− type a) – shear failure of the concrete webs as a result of principal tensile stresses;

− type b) – shearing off of the topping layer as a result of interface shear capacity excess.

In the case of structural failure as a result of type b) longitudinal shear, code [7] recommends the application of the procedure given in EC2 [8]. The longitudinal shear resistance of the composite elements, constructed from precast element and an in-situ topping layer without transverse reinforcement, consist of bond between composite elements concretes and friction between elements and presence of interface reinforcement (if any). Interface surface characteristics are described by the following parameters: c – adhesive coefficient and μ – friction coefficient.

In the currently withdrawn polish standard PN-B:03264:2002 [20], similar to many others codes and recommendations previously applied in European countries (e.g. *fib* Bulletin 6 [9], published in 1995), the method of the determination of the interface longitudinal shear capacity was analogical to EC2, except from slightly higher values of coefficient c. *fib* recommendations [9] additionally indicated the necessity of including in calculations of

longitudinal tangent stresses at the interface τ_{Ed} –the additional component of those stresses, connected with placing reinforcement in filled cores of the support zone of HC slab.

The values of coefficients c and μ suggested by [7–9] and [20] are compiled in the Table 1.

Table 1

Values of c and μ coefficients

Roughness	Adhesive coefficient c		Coefficient of friction μ	
	Eurocode 2 EN 1168	*fib* Bulletin 6 PN-B 03264	Eurocode 2 EN 1168	*fib* Bulletin 6 PN-B 03264
Very smooth	0.025÷0.1	0.02	0.5	0.5
Smooth	0.2	0.35	0.6	0.6
Rough	0.4	0.45	0.7	0.7
Indented	0.5	0.50	0.9	0.9

Chapter 7 (section 7.3.3.6) of Model Code 2010 [10], which is dedicated to structural designing, presents exactly the same calculation algorithm as EC2 for the interface between two concretes without crossing reinforcement. However, section 6.4.3 of this same document, which discusses issues of modelling and the designing of the bond between the two concretes, gives different, more accurate information resulting from experimental studies. The ultimate capacity of the interface under shear forces is defined there as the sum of the adhesion effect with the interlock mechanism, friction under shear forces and dowel action of the transverse reinforcement crossing the interface:

$$\tau_u = \tau_a + \mu\left(\rho \cdot \kappa_1 \cdot f_y + \sigma_n\right) + \kappa_2 \cdot \rho \cdot \sqrt{f_y \cdot f_{cc}} \leq \beta_c \cdot \nu \cdot f_{cc} \qquad (3)$$

where τ_a is the strength due to the adhesion and mechanical interlock.

For the joint without reinforcement or with a small amount of reinforcement ($\rho < 0.05\%$), it can be assumed that: $\tau_u = \tau_a$.

The medium values of adhesive bond stress τ_a and friction coefficient μ, given in MC2010, were assumed on the basis of the research by Randl [21] – see Table 2. Those values were determined for the adequately prepared joint for concrete C50/60 and C20/25.

Table 2

Values of τ_a and μ coefficients, according to MC2010 [10]

Roughness	Adhesive bond stress τ_a	Coefficient of friction μ
Smooth	(0.5–1.5)[*]	0.5–0.7
Rough	~1.5–2.5	0.7–1.0
Very rough	~2.5–3.5	1.0–1.4

[*] values not given in [10], but suggested by [21].

In the further part of this article, information abbreviated as MC2010 will concern the model given in clause 6.4.3.

Since in [10], following [21], only medium values of shear strength for individual surface roughness were given; for the purposes of the calculation of the composite element limit strength, it seems essential to determine the design values for the suggested adhesive bond strength. This can be calculated according to the following expression:

$$\tau_{Rd.d} = \frac{1}{\gamma_c} \tau_{a.m} \cdot \frac{f_{ctk0.05}}{f_{ctm}} \tag{4}$$

and for each rage of adhesive bond stress, the following results were obtained:
– for smooth surface – 0.23–0.70 MPa;
– for rough surface – 0.70–1.15 MPa;
– for rough surface – 1.15–1.60 MPa.

In American standard ACI 318M-11 [1], two methods for determining of the interface capacity of the composite are presented. According to method A, the values of the horizontal shear force V_u and shear horizontal strength V_{nh} should be compared. The expression allowing the determination of the value V_{nh} is the function of the width of joint b_v and the effective height of the cross-section, for a non-reinforced clear surface joint, roughened and free from cement wash. If the surface was not intentionally roughened, but minimal crossing reinforcement was applied, then $V_{nh} = \varnothing b_v d$ [kN]. The interface capacity condition could be written as follows:

$$\varphi V_{nh} \geq V_u$$

where φ is the safety coefficient for strength calculations, equal to 0.85.

According to method B, horizontal shear force can be calculated as the difference between horizontal compressing and tensile forces in individual cross-section segments. In this case, the bearing capacity of the joint is calculated at the surface of joint $b_v \cdot l_v$, where l_v is the length of the shear cross-section. Method B assumes a limitation of the maximum interfacial shear stress, to a value not exceeding 0.55 MPa (design value).

When applying approach [1] to method B, higher horizontal shear strength results are obtained compared to method A due to the averaging of horizontal shear stress. Another difference between these two methods is the use of the depth parameter d in method A – this is a simplification of the lever arm between the tensile and compressive cross-section forces, which can be stated more formally as $(d – a/2)$, where a is the depth of the compressive stress block.

3. Research on the interface shear

The experimental tests carried out in Poland by A. Ajdukiewicz et al. [2] were developed in order to clarify the actual behaviour of slabs with an in-situ concrete topping layer without interface reinforcement. The interaction of the topping was very satisfactory, up to level of

over 95% load at failure. The results of the experiment showed that in the case of a well-developed interface surface, shear resistance at the interface is much higher in comparison to the idealised shear resistance calculated according to [7] or [8]. This proves that rules given in these codes may be omitted in practical situations, because the value of the surface factors c and μ may be considered to be too conservative.

M. Gohnert [12] studied the influence of technological aspects on the longitudinal shear strength of the interface. As tests results for beams with topping under flexure showed, a poor correlation exists between the horizontal shear strength and the compressive strength of the concrete. It is therefore not advisable to specify the horizontal shear strength as a function of the concrete compressive strength. However, the regression analysis does indicate an upward trend in shear capacity as the concrete strength is increased. Although the influence of the concrete strength may be less apparent than other factors, the compressive strength does influence the shear strength; therefore, at least a minimum compressive strength should be specified.

Specifying a roughness only by stating the instrument used to create the undulations is not sufficient; an actual measurement of roughness should be specified. A brush or rake can produce a vast range of roughness values depending on the stiffness of the instrument, the amount of pressure applied and the viscosity or age of the mix. A significant increase in the shear capacity as a function of the roughness is shown in [12].

The latest American research focused on the interfacial shear strength between HC slabs and the topping layer with the units fabricated using different typical producer practices [16]. Twenty-four push-off specimens with varying base block surface conditions fabricated using dry-mix (extruder procedure) and wet-mix (slip-former procedure) were tested. The precast slabs were fabricated with different surface conditions to determine the influence of surface roughness on interface shear strength. Elements with machine finished, longitudinally raked, longitudinally/transversely broomed or sandblasted and top surfaces were prepared. In some specimens, a non-shrink sand-cement grout, with a thickness of approximately 1.5 mm was applied to simulate the work of slabs with the presence of laitance.

According to Mones and Brena [17], the interfacial shear stress limit of 0.55 MPa specified in [1] for intentionally roughened surfaces is conservative for all surface conditions tested including machine-finished specimens. Test result show that higher shear strengths of unreinforced composite interfaces can be obtained by roughening the hardened surface. For dry-mix HC slabs, a strong positive linear correlation was observed between surface roughness and interfacial shear strength and horizontal slip capacity. The interfacial shear strength of wet-mix HC units was related to both surface roughness and the presence of laitance. Roughened interfaces developed a higher strength and horizontal slip capacity than machine-finished interfaces. Sandblasting removed the laitance layer from the wet-mix HC slabs and improved interfacial shear strength by providing a higher quality cohesive bond and, to a lesser extent, by increasing the surface roughness. Grout generally did not impair the interfacial shear strength of dry-mix and wet-mix HC slabs surfaces. In fact, grouted dry-mix specimens had significantly higher interfacial shear strength and horizontal slip capacity than companion specimens that were not grouted. The results for grouted wet-mix specimens are not so clear, but it has been shown that, in general, the grout layer does not impair the join work.

Based on the results of the experimental push-off type tests, carried out by Moenes [16], the authors of this paper calculated the values of the shear stresses τ_{Ed}. Taking into account the

knowledge concerning the concrete topping strength, it was possible to determine adhesion coefficient *c*. The results of these calculations are presented in Table 3. Because for each type of surface only two computational results were available, for the purpose of further analysis, the smaller from obtained results was applied. The minimal value of the coefficient *c* was compared with the values of adhesion coefficients recommended by codes [8] and [20], and the minimal value of shear stress was compared with the recommendations of [1] and [10] – see Table 4.

Table 3

Characteristic interface strength parameters, determined on the basis of researches [16]

Specimen	Surface condition	Max. force F_{test} [kN]	Shear stress τ_{Ed} [MPa]	Topping tensile strength f_{ctm} [MPa]	Adhesion coefficient c_{test} [−]	Mean value of adhesion coefficient $c_{test.m}$ [−]
DRY-MFX-1	Machine finished	206.8	**1.45**	2.9	**0.50**	0.46
DRY-MFX-2		152.1	**1.06**	2.5	**0.42**	
DRY-SBX-1	Sandblasted	161.9	**1.13**	2.9	**0.39**	0.44
DRY-SBX-1		215.3	**1.50**	3.1	**0.48**	
DRY-LRX-1	longitudinally raked	223.3	**1.56**	2.9	**0.54**	0.50
DRY-LRX-2		205.1	**1.43**	3.1	**0.46**	
DRY-TBX-1	Transversely broomed	287.8	**2.01**	3.1	**0.65**	0.65
DRY-TBX-2		319.4	**2.23**	3.4	**0.66**	
DRY-MFG-1	Machine finished, grouted	275.8	**1.93**	2.9	**0.66**	0.76
DRY-MFG-2		377.2	**2.63**	3.1	**0.85**	
DRY-LRG-1	Longitudinally raked, grouted	276.7	**1.93**	3.1	**0.62**	0.61
DRY-LRG-2		266.0	**1.86**	3.1	**0.60**	
WET-MFX-1	Machine finished	198.4	**1.38**	3.1	**0.45**	0.37
WET-MFX-2		127.7	**0.89**	3.1	**0.29**	
WET-SBX-1	Sandblasted	267.8	**1.87**	3.0	**0.62**	0.57
WET-SBX-2		225.1	**1.57**	3.1	**0.51**	
WET-LBX-1	longitudinally broomed	222.0	**1.55**	3.1	**0.50**	0.42
WET-LBX-2		144.1	**1.01**	2.9	**0.35**	
WET-TBX-1	Transversely broomed	257.5	**1.80**	2.9	**0.62**	0.57
WET-TBX-2		247.8	**1.73**	3.3	**0.52**	
WET-MFG-1	Machine finished, grouted	157.5	**1.10**	2.9	**0.38**	0.39
WET-MFG-2		165.5	**1.15**	2.9	**0.40**	
WET-LBG-1	Longitudinally broomed, grouted	247.3	**1.73**	2.7	**0.64**	0.61
WET-LBG-2		218.4	**1.52**	2.6	**0.59**	

As conducted analyzes demonstrate, the interface strength parameters determined for the test results presented in master thesis [16] are in each case significantly higher than their equivalents given in codes or standards. The adhesion coefficient for the smooth surface given in [8], as well as the shear stress given in [1], are at least twice as small as those obtained in the tests. Fairly strong correlations (while maintaining of all safety reserves) are obtained for the parameters c and μ taken from previously obligatory codes (e.g. in [20] or [21]), as well as using the information from Chapter 6 of Model Code 2010 [10].

Ta b l e 4

Comparison of the research results with calculation results

Surface condition	Min. value of adhesion coefficient obtained in tests $c_{test.m}$ [−]	Adhesion coefficient given in [8] c [−]	Adhesion coefficient given in [20] c [−]	Min. value of shear stress obtained in tests τ_{Ed} [MPa]	Shear stress given in [1] τ_{Rd} [MPa]	Design shear stress for [10] $\tau_{Rd.d}$ [MPa]
DRY-MFX	**0.42**	0.2	0.35	**1.06**		0.23–0.70
DRY-SBX	**0.39**	0.2	0.35	**1.13**		0.70–1.15
DRY-LRX	**0.46**	0.4	0.45	**1.43**		~1.15
DRY-TBX	**0.65**	0.4	0.45	**2.01**		~1.15
DRY-MFG	**0.66**	0.2	0.35	**1.93**		0.23–0.70
DRY-LRG	**0.60**	0.4	0.45	**1.86**		~1.15
WET-MFX	**0.29**	0.2	0.35	**0.89**	0.55	0.23–0.70
WET-SBX	**0.51**	0.2	0.35	**1.57**		0.70–1.15
WET-LBX	**0.35**	0.4	0.45	**1.01**		~1.15
WET-TBX	**0.52**	0.4	0.45	**1.73**		~1.15
WET-MFG	**0.38**	0.2	0.35	**1.10**		0.23–0.70
WET-LBG	**0.59**	0.4	0.45	**1.52**		~1.15

4. Calculation example – comparison of design standards

To present the differences arousing from application of the different codes, concerning longitudinal interface shear capacity of the hollow core slab with concrete topping, computational example has been conducted. For the calculations, a prestressed HC slab with a height of 500 mm and made of C50/60 concrete was assumed. The concrete topping with a height of 60 mm and made of C20/25 class concrete was layered on the upper surface of the slab. It was also assumed that the slab is simply supported and uniformly loaded with the self-weight $g_d = 6$ kN/m², an additional static load $\Delta g_d = 1$ kN/m² and a service load of a variable value.

In order to determine the coefficient of compressive stress distribution carried by the topping concrete and slab (coefficient β), detailed analyses of the diagram of stresses along the height of the cross-section were carried out. Stresses due to self-weight, prestressing, additional loads and service loads at the appropriate levels were calculated. Additionally, stresses from the shrinkage difference for two different materials (topping and slab concrete) were determined. The distribution of stresses in the composite cross-section is presented at the schematic diagram below (Fig. 3).
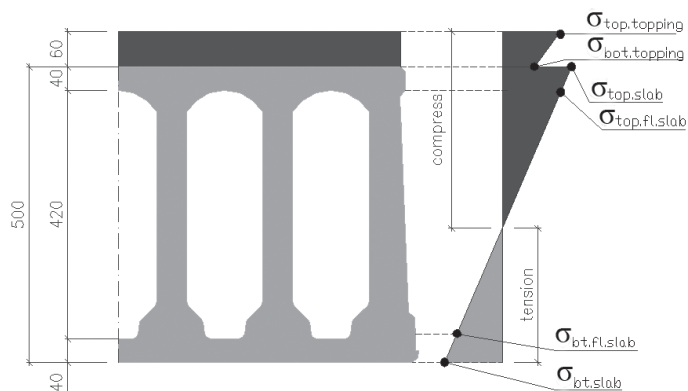

Fig. 3. Concrete stress distribution on the height of the composite cross-section

Initial calculations concerned the determination of the longitudinal shear strength, making allowance for those types and characteristics of surface preparation, which are used for hollow core slabs – i.e. smooth and rough surface according [8] or [20]; rough or very rough surface according to [10] and rough surface according [1]. The results of this analysis at the adopted level of service loading $q_d = 5$ kN/m^2 are presented in Table 5. Table 6 presents the values of longitudinal shear stresses that would possibly appear in the joint of the analysed floor with a span length variable in the range from 14 m to 20 m.

T a b l e 5

Comparision of interfacial shear strength calculated for different codes

v_{Rd} [kPa]							
EC 2 EN 1168		*fib* Bul. 6 PN-B 03264		MC2010		ACI 318M-11	
smooth $cf_{ctd} + \mu\sigma_n$	290 + 6 = **296**	smooth $cf_{ctd} + \mu\sigma_n$	508 + 6 = **514**	smooth rough	**230-700** **700-1150**	clean, free of laitance and intentationally roughened	**550**
rough $cf_{ctd} + \mu\sigma_n$	580 + 8 = **588**	rough $cf_{ctd} + \mu\sigma_n$	652 + 8 = **660**	rough very rough	**700-1150** **1150-1600**		

As the calculation results demonstrate, the smallest interface shear strength values were definitely obtained when applying design procedures recommended by EC2 (and therefore recommended by code EN 1168, which in this range has a direct reference to EC2). Limitation of the shear stresses, given in ACI-318M [1], is convergent with recommendations of previously obligatory standards for smooth surfaces but when concerning the surfaces regarded in Europe as rough, this limitation should be treated with greater caution. It should be noted that the PCI Design Handbook [19] states that experience and testing indicate that normal finishing methods used for precast concrete surfaces (in Europe considered as a smooth surfaces) may be treated as intentionally roughened. Given in Chapter 6 of the latest version of Model Code [10] interface shear strength magnitudes, result in the highest values, even when taking into consideration only the lower limits of the recommended ranges.

Table 6

Interfacial shear stress for different span length of the slab

	$L = 14$ m	$L = 16$ m	$L = 18$ m	$L = 20$ m
v_{Ed}	81 kPa	99 kPa	117 kPa	135 kPa

It should also be pointed out that the contribution of friction forces in the interface shear strength is very small for floor structures. The magnitude of the component connected with friction $\mu\sigma_n$ depends on the level of service loading. However, even for extreme levels of loading, the component itself, when analysing composite floors, remains at a negligible level of 1–4% for smooth surfaces and 1–2% for rough surfaces. In Table 7, the values obtained for the analysed floor with a 16 m span and service loading in the range of 3.5 kN/m² to 10.0 kN/m² are presented.

Table 7

Interfacial shear strength obtained for different live loads

		EC2	*fib* Bul. 6	EC2	*fib* Bul. 6	EC2	*fib* Bul. 6	EC2	*fib* Bul. 6
Live load q [kN/m²]		3.5		5.0		7.5		10.0	
smooth	cf_{ctd} [kPa]	290	508	290	508	290	508	290	508
	$\mu\sigma_n$ [kPa]	5.2		6.5		8.8		11.0	
	$\mu\sigma_n/cf_{ctd} + \mu\sigma_n$	**2%**	**1%**	**2%**	**1%**	**3%**	**2%**	**4%**	**2%**
rough	cf_{ctd} [kPa]	580	652	580	652	580	652	580	652
	$\mu\sigma_n$ [kPa]	6.0		7.6		10.2		12.9	
	$\mu\sigma_n/cf_{ctd} + \mu\sigma_n$	**1%**	**1%**	**1%**	**1%**	**2%**	**2%**	**2%**	**2%**

For the emphasizing deserves also the fact, that obtained design bearing capacity values of non-reinforced interface of HC slab and concrete topping – even in case of application of the most strict recommendations given in standards or codes – provides much higher values than possible stresses obtained for typical uniformly loaded floor structures. The problem with the bearing capacity of the interface between the precast slab and the concrete topping might possibly occur only in situations in which high values of concentrated or linear loadings would generate high magnitudes of transverse forces.

5. Conclusions

Each year in Europe, more than 20 million square meters of prestressed precast hollow core slabs are being produced. Very often, directly on the upper surface of the precast element, the layer of structural concrete topping is cast in order to improve the performance of the floor structure. Commonly used technologies of HC floor element production disable the possibility of the application any kind of transverse reinforcement. For that reason, the interface between the slab and the concrete topping in most cases remains non-reinforced. In these situations, the quality of the upper surface of the precast HC element (both its formation during the production process and the preparation of the surface prior to casting the topping) has a significant influence upon the interface capacity.

In this paper, different recommendations of various standards and codes have been presented (EC2, EN 1168, *fib* Bulletin 6, PN-B-03264, Model Code 2010, ACI-318M) concerning the scope of the bearing capacity of concrete composite structures without interface reinforcement. Comparisons of these guidelines were conducted on the basis of computational examples, where for individual standards, different results of interface shear capacity were obtained. For a better recognition of the problem, the results of the experimental research carried out by other scientists (especially by Mones [16]), giving considerations for different ways of forming the upper surface of the HC slabs, were analysed. It was demonstrated that calculations according to the current obligatory European codes ([7, 8]) provided the lowest possible values of joint bearing capacity, these were far smaller than the values obtained in the experiments. Similar conclusions were drawn in other, earlier pieces of research (e.g. [2]). The analysis of the existing databases, covering the results of experimental research of the composite concrete beams, revealed that the values of ultimate loads obtained on the experimental path are 3–11 times bigger than the values calculated according to the procedures presented in [8]. For the monolithic elements, the ratio of the experimental to calculated failure loads remained within the range of 1 to 3 [14]. The introduction of the decreased values of coefficients c and μ (characterising the composite joint) during the changes to the European standards and codes taking place at the beginning of the century was not justified. The values of those coefficient in previously regulations (e.g. [9] and [20]) give the interface strength values closer to the real interface capacity. The recently published Model Code 2010 [10] introduced changes to both the classification and the characteristics of individual types of interface, as well as in the procedure of calculation of the longitudinal shear capacity given in Chapter 6. It appears that the mean values of the adhesive bond stresses suggested

in [10] allow for a good representation of the actual performance of those types of joints. In this paper, the procedure of transition from medium values of adhesive bond stresses to design values of those stresses, that could be applied in the structural design, has been presented.

Analyses carried out by the authors demonstrated that in the case of ceiling or roof slabs, the participation of the friction effect in joint longitudinal shear capacity is negligible and can be omitted in calculations.

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