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Research paper

Shear capacity of the interface between concretes cast at different time in the light of experimental investigations and codes of practice

Michał Gołdyn[1]

Abstract: The paper presents selected issues related to the load carrying capacity of joints between concretes cast at different times. The most important factors affecting the shear resistance, such as: surface roughness (profile), shear reinforcement ratio, concrete strength as well as the aggregate composition are discussed, including results of previous experimental studies conducted on push-off specimens and composite reinforced concrete beams. The differences in behaviour and shear resistance of contacts between ordinary concretes, lightweight aggregate concretes and recycled aggregate concretes are presented. Principles of interface design in the light of codes of practise: AASHTO-LRFD, ACI 318-19, EN 1992-1-1 and prEN 1992-1-1 were described. The theoretical predictions were compared with 184 results of experimental tests on push-off specimens. It has been found that most of the procedures allow for a safe estimation of the load carrying capacity of interfaces – with and without shear reinforcement. However, the obtained results were mostly conservative (depending on the considered design procedure, ratio of the experimental to theoretical load carrying capacity lies in range 1.51÷2.68). This may indicate that the description of shear transfer mechanism between concretes cast at different times is still imperfect and need to be improved.

Keywords: shear capacity, shear-friction, push-off test, concrete interface, concretes cast at different time, overlay

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1. Introduction

Structural overlay is currently used in various types of composite structures: slabs supported on precast beams (Fig. 1a), floor slabs formed on Filigree elements (Fig. 1b)) or ceilings made of precast hollow-core slabs. It is also used in existing structures (Fig. 1c)), if the need for increasing load carrying capacity occurs. This may result from new functional needs (change in use of building), or the need to strengthen the structure as a result of design [46] and execution errors or technical wear (e.g. as a result of intensive abrasion or chemical attack). In such cases, additional reinforcement is placed in the concrete overlay to compensate for any load capacity deficits.

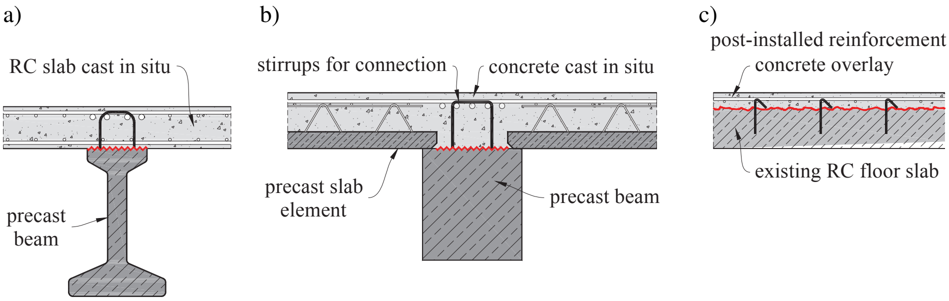

Fig. 1. Examples of composite structures (interface between concretes cast at different time marked with red): a) precast girder with RC slab, b) precast beam with composite floor slab, c) slab with RC structural overlay

The key issue then is to ensure proper bonding between the "old" and "new" concrete. For this reason, the paper attempts to characterize the factors affecting the load capacity of the interfaces between concretes cast at different time in the light of the experimental research conducted so far. Selected design procedures were also assessed, presenting the philosophy of calculations adopted in them.

2. Background

2.1. Shear-friction theory as first design approach

The issue of the shear transfer between concretes cast at different time has been the subject of experimental research since the 1960s. The shear-friction theory, presented for the first time by *Mast* [27] and described in detail in the work of *Birkeland* and *Birkeland* [4], was verified on the basis of numerous experimental push-off test, conducted, among others, by *Hofbeck* et al. [19], *Kriz* and *Raths* [25], *Mattock* and *Hawkins* [33]. It should be noted that shear-friction theory is a relatively simple engineering model. In this concept it is assumed that as a result of the mutual displacement of both contact surfaces and the interlocking of aggregate particles, the width of the crack tends to increase, which is

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counteracted by the reinforcement crossing the joint (the so-called saw-tooth model) – see Fig. 2. In this way, clamping stress σ, normal to the shear plane, arise what induce frictional forces at interface slip. In the ultimate state, yielding of the entire reinforcement crossing interface is assumed and the maximum shear stress is defined as follows

$$\tau_i = \mu \cdot \sigma = \mu \cdot \frac{A_s f_y}{A_v} = \mu \cdot \rho f_y \tag{2.1}$$

where: μ – effective coefficient of friction which includes also other effects (apart from friction itself), ρ – ratio of the reinforcement crossing the shear plane, f_y – yield strength of the reinforcing steel

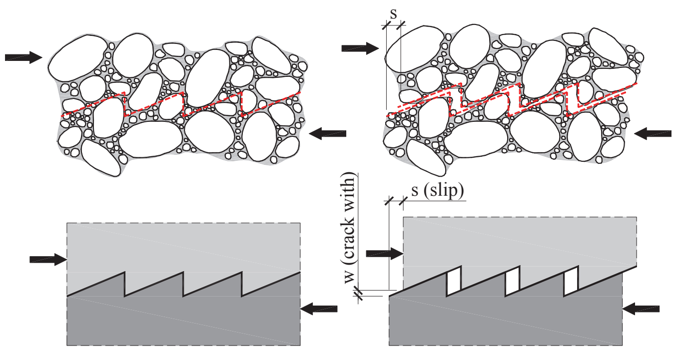

Fig. 2. Saw-tooth analogy proposed by *Birkeland* and *Birkeland* [4]

According to the authors of further works, the use of expression (2.1) may lead to a conservative estimation of the load capacity of the interfaces between concretes cast at different times as well as the load carrying capacity of monolithic elements subjected to shear-cutting. For this reason, numerous modifications of equation (2.1) have been proposed over the years, including consideration of the adhesive forces, aggregate composition or the possibility of concrete crushing. For the comparative purpose, the ultimate shear stress was presented in general form as

$$\tau_i = \underbrace{c}_{\text{adhesion}} + \underbrace{k_s \cdot \rho f_y}_{\substack{\text{friction} \\ \text{(clamping effect)}}} + \underbrace{k_f \cdot \sigma_n}_{\substack{\text{friction} \\ \text{(normal stress)}}} \tag{2.2}$$

where: c – coefficient reflecting adhesive forces, k_s – coefficient of friction (for contribution of reinforcement), k_f – coefficient of friction (for contribution of normal stress σ_n), the remaining designations are the same as in eq. (2.1).

The coefficients in the equation (2.2) are summarized in Table 1, where possible limitations are also indicated. Some of the proposals identified in the experimental studies have been included in the subsequent editions of the standards ACI 318 and AASHTO-LRFD (i.e. [1, 2] – see section 3).

Table 1. Comparison of design approaches based on shear-friction theory

Authors	Coefficients in eq. (2.2)			$\tau_{i,\max}$	Detailed application	
	c	k_s	k_f			
Birkeland and *Birkeland* (1966) [4]	–	1.7	–	≤ 5.52 MPa	monolithic concrete	
		1.4			artificially roughened construction joints	
		0.8÷1.0			ordinary construction joints and concrete to steel interfaces	
Mattock and *Hawkins* (1972) [33]	1.38 MPa	0.8	0.8	$\leq 0.3 f_c \leq 10.34$ MPa		
Mattock (1974) [28]	2.76 MPa			$\leq 0.3 f_c \leq 10.34$ MPa		
Hermansen and *Cowan* (1974) [18]	4.00 MPa	0.8	0.8			
Mattock, Li and *Wang* (1976) [32]	1.38 MPa	0.8	–	$\leq 0.2 f_c$ ≤ 5.52 MPa	ALWAC	
	1.72 MPa			$\leq 0.2 f_c \leq 6.90$ MPa	SLWAC	
Mattock (1988) [30]	$0.467 f_c^{0.545}$	0.8	0.8	$\leq 0.3 f_c$		
Mattock (2001) [31]	$0.1 f_c \leq 5.52$ MPa	0.8	0.8	$\leq 0.3 f_c \leq 16.55$ MPa	monolithic concrete	
	2.76 MPa			$\leq 0.3 f_c \leq 16.55$ MPa	concrete cast on hardened concrete with surface intention. roughened	NWC
	1.72 MPa			$\leq 0.2 f_c \leq 8.27$ MPa		SLWAC
	1.38 MPa					ALWAC
	–	0.6λ	–	$\leq 0.2 f_c \leq 5.52$ MPa	concrete cast on hardened concrete, surface not intentionally roughened	
		0.7λ			concrete cast on clean as-rolled steel	

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Table 1 [cont.]

Authors	Coefficients in eq. (2.2)			$\tau_{i,\max}$	Detailed application	
	c	k_s	k_f			
Patnaik (2001) [37]	0.60 MPa	1.0	–	$\leq 0.2 f_c$ ≤ 5.52 MPa	smooth interfaces	
Kahn and *Mitchell* (2002) [24]	$0.05 f_c$	1.4	–	$\leq 0.2 f_c$	HSC	
Papanicolaou and *Triantafillou* (2002) [36]	$1.7(f_{ct})^{0.5}$	0.3	0.3		PAC	smooth interface
	$1.4(f_{ct})^{0.5}$	0.45	0.45			rough interface

ALWAC – all-lightweight aggregate concrete
SLWAC – sanded-lightweight aggregate concrete
NWC – normal weight concrete
HSC – high strength concrete
PAC – pumice aggregate concrete
f_c – concrete compressive strength
f_{ct} – concrete tensile strength

The coefficient λ is equal to 1.0 for NWC, 0.85 for SLWAC and 0.75 for ALWAC

2.2. Factors affecting shear capacity of the interface

Based on the results and conclusions from the previous experimental investigations on push-off elements, the effect of the most important factors influencing the load carrying capacity and behaviour of interfaces between concretes cast at different times were described in the following section.

2.2.1. Concrete strength

Most of the calculation methods and design procedures make the specific adhesion forces dependent on the roughness of substrate and tensile strength of concrete. As a rule, a linear relationship is assumed between the adhesion and tensile strength of concrete f_{ct} [35, 42, 43], although it was found in [41] that the contribution of adhesive forces is proportional to the third-degree root of the concrete compressive strength. In tests on specimens with shear reinforcement [44], the increase in concrete compressive strength from 34 to 55 MPa (by about 60%) resulted in an increase in load capacity by 20÷40% (rough surface) and 25÷70% (smooth surface), depending on the density of concrete. The studies of *Fang* et al. [8] demonstrated that strength of lightweight aggregate concrete had a moderate effect on the shear strength of the interface. An increase in strength of concrete from 32 to 49 MPa resulted in an increase in shear stress by 22.6, 6.5 and 10.7% for the shear reinforcement ratio ρ_s = 0.80, 1.20 and 1.80%, respectively. Therefore, the highest effect of concrete strength was visible in case of elements with the lowest reinforcement.

In addition to the forces associated with adhesion, the strength of concrete may be important from the point of view of the capacity of diagonal concrete struts, therefore e.g. in works [24] and [31] it is recommended to limit the shear stress to value of $0.2 \div 0.3 f_c$. In the paper of *Hsu* et al. [20], it was found that the shear-cutting strength of concrete is determined by the reduction of the concrete compressive strength due to cracking, depending on the reinforcement located in the vicinity of the interface, which limits the crack widths and thus reduces the concrete softening effect.

2.2.2. Shear reinforcement

There is a consistent view in the literature that increasing of transverse reinforcement leads to an increase in shear transfer capacity of the interfaces. *Harries* et al. [17] note that the reinforcement limits the slip between both contact surfaces and thus the development of the crack width. However, the maximum forces were recorded at a relatively low slip, when the stress in the reinforcement were far from the yield point. It has been stated that shear resistance is rather a function of reinforcing steel modulus of elasticity and proposed to include in the calculations of clamping force the product of $\mu \cdot \varepsilon_s = 0.002$ (which translates into strain of the reinforcement $\varepsilon_s \approx 1.4$‰in case of a rough joint interface). However, if the possibility of cracking has to be avoided (which, according to various researchers, occurs at a slip $s < 0.25$ mm [17], $s \sim 0.05$ mm [41], $s \sim 0.02$ mm [22]), then in the calculations only the adhesive forces should be considered (contribution of the reinforcement is ignored).

Also in the research by *Randl* and *Wicke* [41] it was found that in most cases the shear reinforcement did not yield at cracking of the interface and stress reached about 50% of the yield point (which was also confirmed by the earlier observations of *Mishima* et al. [34]). The authors have explained this situation by a combined action of two mechanisms, wherein the reinforcement is subjected: tension associated with the contact separation (clamping effect) and bending of the dowels (kinking effect). As a result of the slip, plastic hinges form in the vicinity of the shear plane (at a distance of about 1÷2 ∅ from the interface, where ∅ is the reinforcement diameter) and it is impossible to utilize full tensile capacity of shear reinforcement.

Research of *Ahmad* et al. [3] (ρ_s = 0÷1.6%) shows that shear reinforced interfaces are characterized by a higher deformability (the slip recorded at the maximum load was 0.23÷0.92 mm and about 0.1 mm for interfaces with and without reinforcement, respectively).

In the studies of *Júlio* et al. [23], in which the effect of the quantity and method of installation of the interface reinforcement was considered, it was found that the cross-section of shear reinforcement did not affect load at adhesion break, but contributed to a qualitative change in behaviour at post-peak phase (drop in load and load capacity at post-peak phase). In case of stronger reinforced interfaces (6 ∅ 6, ρ_s = 0.52%), after cracking, a gradual increase in load associated with slip, was recorded. The same could not be stated for the weakly reinforced elements (2 ∅ 6, ρ_s = 0.18%), when the force recorded in the post-peak phase stabilized at constant level, lower than load at cracking. The performed numerical analysis demonstrated that further increase in shear reinforcement ratio by changing the diameter of the bars from ∅ 6 to ∅ 8 or ∅ 12 might lead to a change in the contact behaviour and result in failure due to reinforcement rupture or debonding. Importantly, only a slight difference in the load carrying capacity of interfaces with reinforcement placed before casting of concrete and with post-installed rebars, reaching about 7÷8%, was found.

In the studies by *Costa* et al. [6] it was stated that increasing of the transverse reinforcement from 0.28 to 0.79% had only slight effect on shear capacity, but it resulted in a reduction in the load drop immediately after cracking from 35 to 16%.

Fang et al. [8], who considered interfaces between high-strength concrete base and lightweight aggregate concrete overlay, found only a slight effect of the shear reinforcement ratio on cracking load. The uniform contribution of all stirrups located along the contact length was recorded, however, at a slip s = 0.5 mm (denoting reaching of the ultimate load capacity according to [26]), the stress in the stirrups did not exceed the yield point, which was achieved only at slip of s = 0.7÷1.7 mm. The authors of the study stated that in the initial phase, the concrete contribution is dominant and the reinforcement becomes active only after interface cracking. For this reason, they recommended not to include simultaneous contribution of adhesion and shear reinforcement.

Authors' investigations [11] (ρ_s = 0÷0.56%) confirmed that cracking of the interface and thus breaking the adhesive forces occurs at a relatively low slip of 0.05÷0.10 mm, regardless of the shear reinforcement ratio. The cross-section of the reinforcement affected the forces achieved at cracking (the introduction of the reinforcement allowed to increase

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the load capacity up to 23%) and drop in load after cracking, ranging from about 50% (ρ_s = 0.56%) to even 85% (ρ_s = 0.14%), as well as the residual load capacity – see Fig. 3.

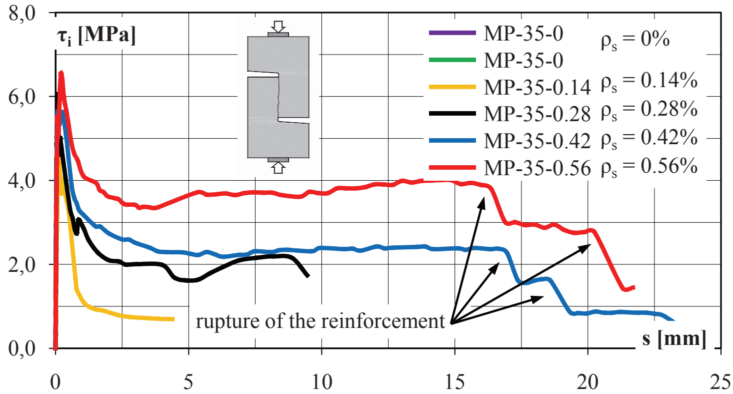

Fig. 3. Load-slip relation for push-off specimens with various shear reinforcement (extended research from [11])

The observations resulting from the previous experimental investigations are presented conceptually in Fig. 4. Initially, increasing the load V_{int} acting in the shear plane is accompanied with only a very low mutual surface displacements s. When the cracking load is reached, the crack occur and the specific adhesion bonds are broken. Further behaviour of the interface depends on the intensity of shear reinforcement that cross the shear plane. The following situations are possible (see Fig. 4):

– non-reinforced interface – cracking is synonymous with the failure of the connection and exhaustion of the load capacity; only the adhesion forces determine the load carrying capacity and failure of the contact is violent;
– low reinforced interface – immediately after cracking, a significant drop in load and intense slip are observed, accompanied by an almost constant residual load capacity; residual load results to a small extent from friction (clamping effect) and, above all, from the dowel action of reinforcement; as a result of tension and bending, all rebars yield and then rupture almost simultaneously; as in case of non-reinforced contacts, the load capacity is governed by the adhesive forces, however, the failure itself is ductile;
– moderately reinforced interface – the drop in load immediately after cracking is observed, but it is not as high as in case of low reinforced contacts; residual load is constant or slightly increase at slip; with large mutual displacements of the surfaces, reinforcing bars gradually rupture and the load changes by leaps and bounds;
– heavy reinforced interface – depending on the intensity of the reinforcement, a drop in load after cracking is recorded or not; a gradual increase in load corresponding to interface slip is observed and maximum post-peak load may even exceed the load at cracking; failure may be a result of concrete crushing, debonding (post-installed reinforcement) or rupture of the reinforcement; in the post-peak phase, the aggregate interlock (friction) and dowel action mechanisms are decisive.

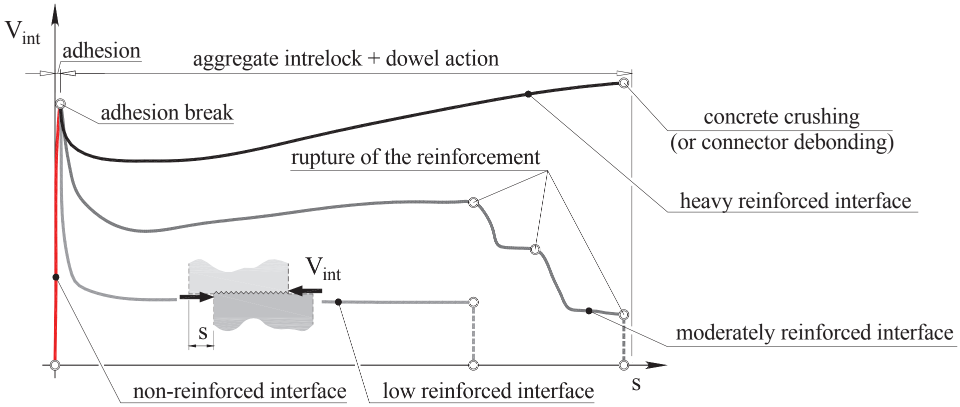

Fig. 4. Load-slip characteristics of concrete-to-concrete interfaces with different shear reinforcement

Activating the forces resulting from the aggregate interlock and dowel action of the reinforcement requires displacements of the contact surface (slip), which, however, may not be acceptable from the point of view of the serviceability (appearance and durability of the structure).

2.2.3. Aggregate composition

Particle size distribution of aggregates used in concrete influences the interlock effect and thus the so-called mechanical cohesion (Verhakungskohäsion) [41]. As the width of crack increases, the adhesion forces decrease, which, in the opinion of *Walraven* and *Reinhardt* [47], results from a decrease in the contribution of asperities in the interlocking mechanism. It turns out, however, that the mechanical properties of the aggregate also have a meaningful effect on the load carrying capacity and interface behaviour, which is especially visible in the case of lightweight aggregate concretes. The extensive research by *Mattock* et al. [32] demonstrated different load capacities of interfaces in ordinary and lightweight aggregate concretes (sanded-lightweight – SLWAC, all-lightweight – ALWAC), despite similar compressive strength. At the same shear reinforcement ratios, shear stress at failure were on average lower by 6% and 15÷34% (SLWAC and ALWAC specimens with initially uncracked interface) and 3% or 7% (SLWAC and ALWAC specimens with pre-cracked interface) in comparison to ordinary concrete elements. These observations were included in the multipliers modifying the coefficients of friction μ in the shear-friction theory, amounting to 0.75 and 0.85 for ALWAC and SLWAC respectively – see Table 1.

The research of *Jiang* et al. [22] demonstrated that the diameter of lightweight aggregate particles (the study considered aggregate with particle size above and below 16 mm) did not significantly affect the cracking loads or post-peak resistance of the interface. The authors of the study explained this observation with identical value of the friction coefficient according to the shear-friction theory (independent on particle size), but the real reason should rather be seen in the relatively low crushing resistance of lightweight aggregate, which may break. Authors' experiences with lightweight aggregate concretes [12] showed that the size of the particles does not significantly affect the shape of the shear plane.

The composition of the aggregate affects the density of concrete, therefore this parameter was the subject of detailed investigations, among others in studies by *Shaw* and *Sneed* [44] and *Sneed* et al. [45], who considered reinforced interfaces (ρ_s = 0.9÷2.2%) in monolithic and precast lightweight aggregate concrete structures cast from concrete with contemporary artificial aggregates made of sintered clay, slate or shale. In case of elements of the first series [45], the destructive forces for specimens with concrete of similar strengths were close, despite the different densities, which is in contradiction with the observations [32]. However, the results could have been influenced by the relatively high intensity of the transverse reinforcement. In the tests of the second series, an increase in maximum shear stress by 15÷20% was found, related to the increase in concrete densityfrom 1700 to 2350 kg/m^3, but only in case of monolithic elements and connections between concrete cast at different times with a rough surface of the interface. In case of joints with a smooth surface, no clear effect of concrete density was found.

A clear effect of concrete density related to the aggregate composition was observed in the studies of *Costa* et al. [6]. As a result of increasing the surface roughness (increasing the average maximum peak height R_{pm} from about 1 to 7 mm), a significant increase in the ultimate shear stress, reaching by 50÷120% (depending on the level of stress normal to the shear plane σ_n = 0÷6 MPa), was noted only in case of ordinary concrete. In elements made of lightweight aggregate concretes, the change in limit stresses amounted to about 3÷5% (ρ = 1500 and 1700 kg/m^3) and about 15÷20% when ρ = 1900 kg/m^3, which may indicate a limited contribution of the aggregate interlock effect.

Resistance of the aggregate can also play a decisive role in concrete with normal density but high compressive strength. In the studies by *Kahn* and *Mitchell* [24] on the push-off specimens with reinforced joints, made of concrete with a strength of over 120 MPa, aggregate crushing was observed, which resulted in a reduction of the load capacity corresponding to aggregate interlock.

Replacing natural aggregate with recycled aggregate (RAC – recycled aggregate concrete) may also lead to a reduction in the load carrying capacity of the interface between concretes cast at different times. As a result of replacing 50 or 100% of natural with recycled aggregate [39], the cracking loads were reduced by 12 and 20%, and the destructive forces by 7 and 18%, which confirms the conclusions of the previous studies [38]. Similar conclusions can be drawn from the research by *Xiao* et al. [48], who also considered concretes with aggregate replacement ratios of 50 and 100%. Replacing natural aggregate led to a decrease in destructive forces in monolithic elements by about 20%. In the elements with cold joint, however, the opposite tendency was observed – the replacement of all natural aggregate by recycled aggregate was accompanied by an increase in the destructive force of about 15% on average. This change could, however, result from a higher cement content in the concrete mix, what translated into an increase in adhesion forces. A research demonstrated that replacing natural with recycled aggregate has a negative effect on aggregate interlock, especially in monolithic elements. In the paper [48] it was found, that properties of the recycled aggregate are largely dependent on the old cement mortar that coat the aggregate particles. Due to initial damage to the old mortar (microcracks), cracks in RAC form within the contact zones of the old mortar and aggregate (see Fig. 5), which translates into a smaller effective particle size and a reduction of the aggregate interlock effect.

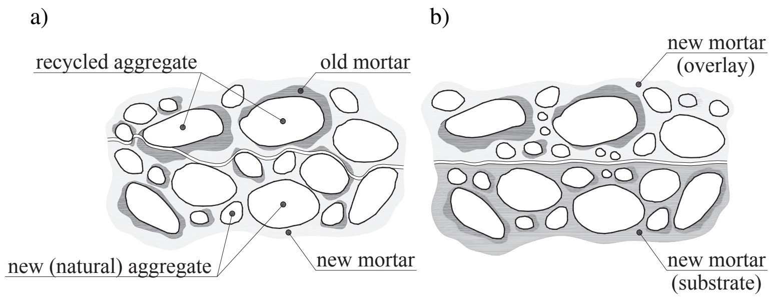

Fig. 5. Formation of cracks in concrete with recycled aggregate: a) monolithic element, b) element consisted in parts cast at different times (cold joint)

2.2.4. Profile of the surface

The effect of the surface profile (shape) was the subject of extensive investigations by *Randl* and *Wicke* [41], in which various methods of surface preparation were considered, relating to the techniques used in practice (including sandblasting and high pressure waterjet – HPW). Those tests showed that increasing the surface roughness leads to an increase in the load capacity of the interfaces between concretes cast at different times, with the maximum force recorded at the slip of 0.2÷1.0 mm and 0.5÷1.5 mm, depending on the roughness R_{pm} equal to 2.7 mm (HPW) and 0.5 mm (sandblasting) respectively. In low reinforced elements with smooth and sand-blasted joint surface, a sharp drop in the registered load was observed, followed by a slip accompanied with only a slight opening of the interface (crack width up to 0.3 mm), which proves the leading contribution of the dowel action. In case of elements with a roughened surface using HPW method, the drop in load was much lower, which was explained by the influence of the mechanical adhesion (Verhakungskohäsion), mainly related to the interlocking of aggregate particles after breaking the specific adhesion forces (with a slip of 0.05 mm).

In the studies of *Costa* et al. [6] it was confirmed that increasing roughness of surface leads to an increase in the shear resistance, however, the effect of this parameter was closely related to the aggregate composition, as mentioned earlier in subsection 2.2.3. The authors proposed equations describing the coefficients of adhesion c and friction μ as a function of the average profile depth R_{pm}:

$$c = 0.86R_{pm}^{0.48}, \qquad \mu = 1.16R_{pm}^{0.04} \quad \text{– ordinary concretes} \tag{2.3}$$

$$c = 1.25R_{pm}^{0.34}, \qquad \mu = 1.16R_{pm}^{0.04} \quad \text{– lightweight aggregate concretes} \tag{2.4}$$

Research of *Gołdyn* and *Urban* [13] demonstrated that method of surface preparation may also play a significant role – especially when the concrete overlay is characterized by higher strength than concrete of substrate. Ultimate shear stress of interfaces with rough surface were 10 or even 43% lower than shear resistances of the interface with smooth surface, what was seen in the local damage of the concrete structure, related to the invasive method of surface roughening (milling).

Mohamad et al. [35] considered only non-reinforced interfaces, using various methods of surface preparation: intact after casting concrete (left as-cast), deep groove, indented, wire-brushing in longitudinal and transverse direction, and differentiating stress acting normal to the shear plane ($\sigma_n = 0 \div 1.5$ MPa). In the case of wire-brushing surfaces, the ultimate shear stress ranged from 3 to over 6 MPa (depending on the stress level σ_n), while in elements with a smooth surface they reached 0.7 to 2.0 MPa (and therefore were 3÷5 times lower). Using the regression method, the authors proposed formulae describing quantities such as the adhesion coefficient c and the friction coefficient μ as a function of the average profile depth R_{pm}:

$$c = 0.2363 e^{0.237 R_{pm}} \tag{2.5}$$

$$\mu = 0.8766\, R_{pm}^{0.3978} \tag{2.6}$$

In the paper [35], an extensive analysis of the relationship between the parameters describing the profile (roughness) of the surface and the shear resistance was also performed. Among the 14 considered parameters, the most accurate were the mean peak height R_{pm} (correlation $R^2 = 0.90 \div 0.92$) and the mean peak-to-valley height R_z (correlation $R^2 = 0.60 \div 0.85$).

Research of *Gohnert* [10] indicated that the ultimate stress is a function of the surface profile, (expressed as the mean peak-to-valley height R_z) rather than the compressive strength of concrete. Based on the analysis of the test on 90 push-off elements constituting a connection of precast beams for suspended beam and block floors with ordinary concrete overlay, the following function describing the stress (in the range $R_z = 0 \div 4.2$ mm) has been proposed:

$$\tau_i = 0.2090 R_z + 0.7719 \tag{2.7}$$

As research [11] and further experimental campaign demonstrated, the composition of the aggregate may also affect the shear resistance in ordinary concretes, when crushing of the aggregate is not observed. Figure 6 shows the relationship between effective shear reinforcement ratio ρ_s (reflecting inclination of the rebars to the shear plane) and ratio of load carrying capacity of element with transverse reinforcement and control specimen without reinforcement (F_s/F_0).

Increasing the amount of shear reinforcement resulted in increase in the load carrying capacity. However, differences between the results for elements of the test series M, MP (with reinforcement perpendicular to the shear plane – $\alpha = 90°$) and MU (with reinforcement inclined with an angle of $\alpha = 60°$ to the shear plane) with similar shear reinforcement ratios, reaching 16÷30%, were stated. The reasons for this state of affairs can be found i.e.

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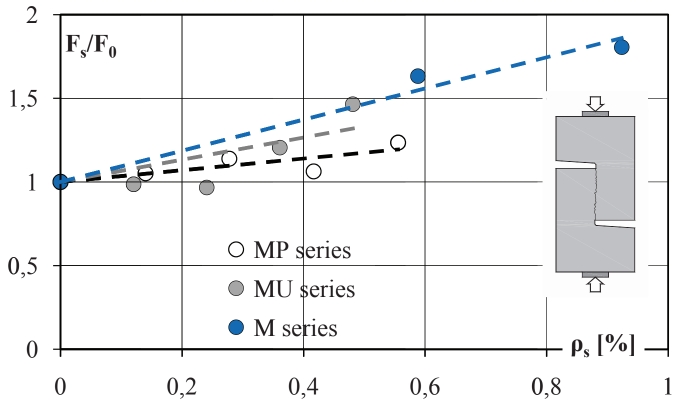

Fig. 6. Effect of shear reinforcement on load carrying capacity of the interface in monolithic concrete (according to [11] and results of unpublished tests)

in the different particle size distribution of the aggregate used in concrete of subsequent test series (crushed or pebble aggregate was used), which resulted in various mechanical adhesion. Figure 7 shows the differentiation of the profiles of failure surfaces in individual test series.

a) b) c)

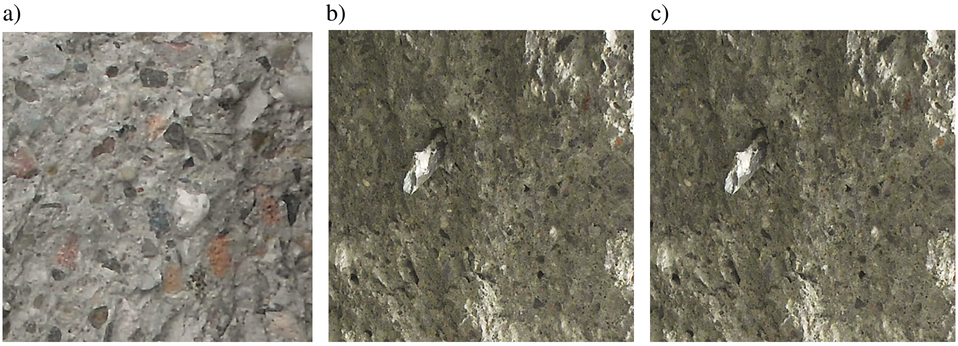

Fig. 7. View of the shear planes after failure of the specimens: a) M series, b) MP series, c) MU series

2.3. Composite beams

Most of the investigations was carried out on push-off elements with various shapes and reinforcement. The design of the test setups made it possible in some cases to include normal stress to the contact surface, which introduced additional frictional forces at the interface. The great advantage of such type of test is the ease of specimen preparation, which affects the number of test series and the ability to analyse various parameters. However, due to size of elements, it is possible to reflect only parts of joints between concretes in the real structures. In case of composite elements, however, it is also particularly important to assess the support zones, where the interaction of longitudinal and transverse shear takes place. For this reason, the results of tests carried out on composite beams made of concretes cast at different times are important. At this point, investigations carried out by *Halicka* et al. can be mentioned, in which such parameters as: shape of composite beam (rectangular [15] or T-shaped [16, 21]), the method of surface preparation, the intensity of the interface reinforcement, adhesion conditions (full or limited adhesion with PVC foil or release agent) [21] and location of the interface in the cross section of the composite element [16].

The results of tests on rectangular beams with an interface located at about 1/3 of the cross-section height [15] demonstrated significant differences in the crack development and the failure mechanism depending on the interface shear resistance – see Fig. 8. In the elements of CB/A+S series (shear capacity dependent on adhesion and contribution of stirrups), formation of diagonal cracks was first observed (at 75÷80% of the destructive force) and then the gradual development of cracks at the interface was visible. No slip between "old" and "new" concrete was stated. Also in the elements of the CB/A series (shear capacity dependent only on the adhesive forces), the development of diagonal cracks was initially observed (at 70÷80% of destructive force). After the crack reached the interface, it propagated towards the loading point and the support (short before failure). No slip between the base element and the overlay was observed. In the elements of CB/S series (shear capacity dependent only on the contribution of stirrups), initially interface was cracked (at 46÷68% of destructive force) and then gradual development of the cracks both towards span and supports was observed. Development of the crack at the interface lead to delamination accompanied with clear slip between "old" and "new" concrete. The formation of diagonal cracks was observed only at the load constituting 68÷81% of the destructive force.

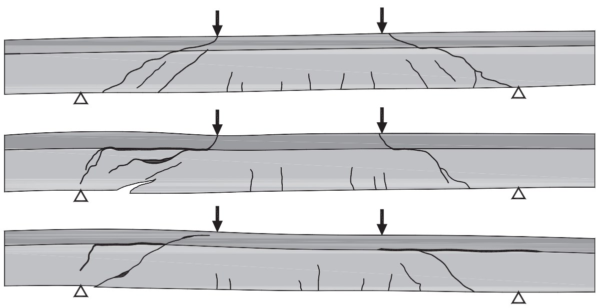

Fig. 8. Crack pattern after failure of the selected composite beams in the tests of *Halicka* [15]

The crack development also affected forces recorded in the transverse reinforcement. In the elements of CB/A+S series, a gradual cooperation of the stirrups was observed only as the crack developed. On the other hand, in the elements of CB/S series, uniform increase in strains was recorded in all stirrups located in the support zone. The observed differences in the behavior of the elements were reflected in the destructive forces – the

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highest values were recorded for the beams of CB/A+S series. They were higher in relation to the destructive forces of the CB/A and CB/S series specimens by approximately 12 and 22%, respectively.

The investigations [21] demonstrated the effect of interface location on the crack development and load carrying capacity of composite beams. Cracks were observed at the earliest in case of elements with interface located in the vicinity of the flange (at 55÷58% of destructive force). The interface was cracked as a result of development of a diagonal crack. In elements with a contact located in the web, cracking of the interface was observed at 74÷90% of the destructive force and resulted from joining of two diagonal cracks forming independently in the "old" and "new" concrete. The observations made in the experimental investigations [14, 15] allowed to formulate a description of two basic failure mechanisms of composite beams:

– initiated by cracking of the interface, when the further behaviour depends on the relationship between the stiffness of primary element $(EJ)_0$ and concrete overlay $(EJ)_n$,
– initiated by diagonal crack followed by cracking of the interface; the composite member exhibits a behaviour similar to a monolithic beam.

3. Codes of practice

3.1. ACI 318-19 procedure

The description of shear stress in ACI 318-19 [2] is based on the original shear-friction theory. It is assumed that in the limit state, the reinforcement crossing shear plane yields, while the friction depends on the surface preparation method (in monolithic concrete or between concretes cast at different times) and the resistance is given as follows:

$$v_u = \lambda \cdot \mu \rho_s f_y \le v_{u,\max} \tag{3.1}$$

$$\lambda = \begin{cases} 1.0 & \text{for NWAC} \\ 0.85 & \text{for SLWAC} \\ 0.75 & \text{for ALWAC} \end{cases}, \qquad \mu = \begin{cases} 1.7 & \text{monolithic concrete} \\ 1.4 & \text{artificially roughened joint} \\ 1.0 & \text{non-prepared surface} \end{cases}$$

$$v_{u,\max} = \min \begin{cases} 0.2 f_c' & \\ 3.3 + 0.08 f_c' & \text{for NWC} \\ 5.5\ \text{MPa} & \text{for LWAC} \end{cases}$$

where: λ – reduction factor, μ – coefficient of friction, ρ_s – shear reinforcement ratio, f_y – yield strength.

The parameter μ is equivalent friction coefficient and also expresses the effects resulting from aggregate interlock and dowel action. The coefficient λ, reflecting the result of the tests by *Mattock* et al. [32], includes the aggregate composition and lower shear resistance of LWA concretes (from 15 to 25%) with respect to ordinary ones.

3.2. AASHTO recommendations

The ultimate shear stress according to AASHTO-LRFD [1], similar as [2], are formulated on the basis of the shear-friction theory – however, including the adhesive forces. Contrary to [7], a constant value of adhesion, which is depended only on the method of the surface preparation, has been introduced. The coefficients of friction μ correspond to unprepared or deliberately roughened surfaces according to [2] and the ultimate shear stress at the interface is equal to

$$v_u = c + \mu \rho_s f_y \leq \min \begin{cases} K_1 \cdot f_c \\ v_{u,\max} \end{cases} \tag{3.2}$$

where: c – adhesion, μ – coefficient of friction, K_1 – means factor reflecting fraction of concrete strength available to resist interface shear, $v_{u,\max}$ – limiting interface shear resistance (due to crushing of the aggregate), see Table 2.

Table 2. Coefficients corresponding to different interface conditions according to AASHTO-LRFD [1]

Type of connection	NWC placed monolithically	Cast-in-place concrete slab on clean concrete girder surfaces	NWC placed against surface intentionally roughened ($R_z > 6.4$ mm)	LWAC monolithic or placed against roughened surface ($R_z > 6.4$ mm)	Concrete placed against a clean surface not intentionally roughened ($R_z < 6.4$ mm)
c [MPa]	2.76	1.93	1.65	1.65	0.52
μ	1.4	1.0	1.0	1.0	0.6
K_1	0.25	0.3	0.25	0.25	0.2
$v_{u,\max}$ [MPa]	10.34	12.4^{NWC}/ 9.0^{LWAC}	10.34	6.89	5.52

3.3. EN 1992-1-1 procedure

Procedure EN 1992-1-1 [7] makes the shear resistance at the interface dependent on adhesion, friction, aggregate interlock and dowel action of the reinforcement, while the simultaneous action of all the above-mentioned mechanisms is assumed

$$v_{Rdi} = \underbrace{c f_{ctd}}_{\text{adhesion}} + \underbrace{\mu \sigma_n}_{\text{friction}} + \underbrace{\rho f_{yd} (\mu \sin \alpha + \cos \alpha)}_{\substack{\text{friction (clamping effect)/} \\ \text{dowel action}}} \leq 0.5 v \cdot f_{cd} \tag{3.3}$$

where: c – factor reflecting adhesion forces (see Table 3), μ – coefficient of friction, σ_n – stress acting perpendicular to the shear plane, (not more than $0.6 f_{cd}$), ρ_s – shear reinforcement ratio (inclined at an angle of α to the interface), f_{yd} – yield strength, $v \cdot f_{cd}$ – compressive strength of the cracked concrete.

3.4. prEN 1992-1-1 code draft

Significant changes in the calculation of the load capacity of the concrete interfaces were introduced by the prEN 1992-1-1 [5], formulated on the basis of research [40, 41] and fib Model Code 2010 [9] rules. Two basic types of interface behaviour were distinguished – rigid and non-rigid. Depending on the classification, two different design approaches were introduced, taking into account the non-simultaneous action of the different effects. In case of rigid behaviour the shear resistance depends mainly on adhesion and friction, however, for reinforced interfaces ($\rho_s > 0$) yielding of the reinforcement is assumed (a connection of precast elements is indicated as an example)

$$\tau_{Rdi} = \underbrace{c_{v1}\sqrt{f_{ck}}/\gamma_c}_{\text{adhesion}} + \underbrace{\mu_v \sigma_n}_{\text{friction}} + \underbrace{\rho_s f_{yd}\left(\mu_v \sin\alpha + \cos\alpha\right)}_{\substack{\text{friction (clamping effect)}\\ \text{dowel action}}} \leq 0.25 f_{cd} \tag{3.4}$$

In case of ductile behaviour (as an example, concrete overlay cast on reinforced concrete slabs is indicated, where cracks due to shrinkage may occur) it is assumed that the load capacity will depend primarily on the dowel action and aggregate interlock

$$\tau_{Rdi} = \underbrace{c_{v2}\sqrt{f_{ck}}/\gamma_c}_{\text{aggregate interlock}} + \underbrace{\mu_v \sigma_n + k_v \mu_v \rho_s f_{yd}}_{\text{friction}} + \underbrace{k_{\mathrm{d}} \rho_s \sqrt{f_{cd} f_{yd}}}_{\text{dowel action}} \leq 0.25 f_{cd} \tag{3.5}$$

The k_v and k_d coefficients (interaction factors) limit the load capacity of the reinforcement due to the complex stress state resulting from the simultaneous tension (clamping effect) and bending (kinking effect) of the rebars. For this reason yielding of shear reinforcement cannot be assumed.

Table 3 summarizes the coefficients used in the discussed codes of practice. A similar definition of the friction coefficients μ was provided. The differences in the values of c and c_{v1} result from replacing in [5] the concrete tensile strength with the square root of the compressive strength.

Table 3. Coefficients depending on the surface preparation according to [5] and [7]

Surface preparation	EN 1992-1-1 [7]		prEN 1992-1-1:2020 [5]				
	μ	c	μ	c_{v1}	c_{v2}	k_v	k_d
Very rough	0.9	0.5	0.9	0.19	0.15	0.5	0.9
Rough	0.7	0.4	0.7	0.15	0.075	0.5	0.9
Smooth	0.6	0.2	0.6	0.075	0	0.5	1.1
Very smooth	0.5	≤ 0.1	0.5	0.0095	0	0	1.5

3.5. Design provisions in the light of test results

The discussed design principles were assessed in the light of 184 results of experimental investigations on push-off specimens (described in [10, 11, 17, 19, 23, 29, 44]).

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Elements with initially uncracked interfaces made of concrete with a compressive strength f_{cm}18÷112 MPa and characterized by a shear reinforcement ratio of $\rho_s = 0 \div 2,64\%$ were taken into account. Mean values of strength parameters of concrete and reinforcing steel with partial safety factors equal to 1.0 were assumed. Figure 9 presents a comparison of the experimental shear stress τ_{exp}, estimated by dividing the destructive force by interface area, and the theoretical shear resistance τ_{calc}.

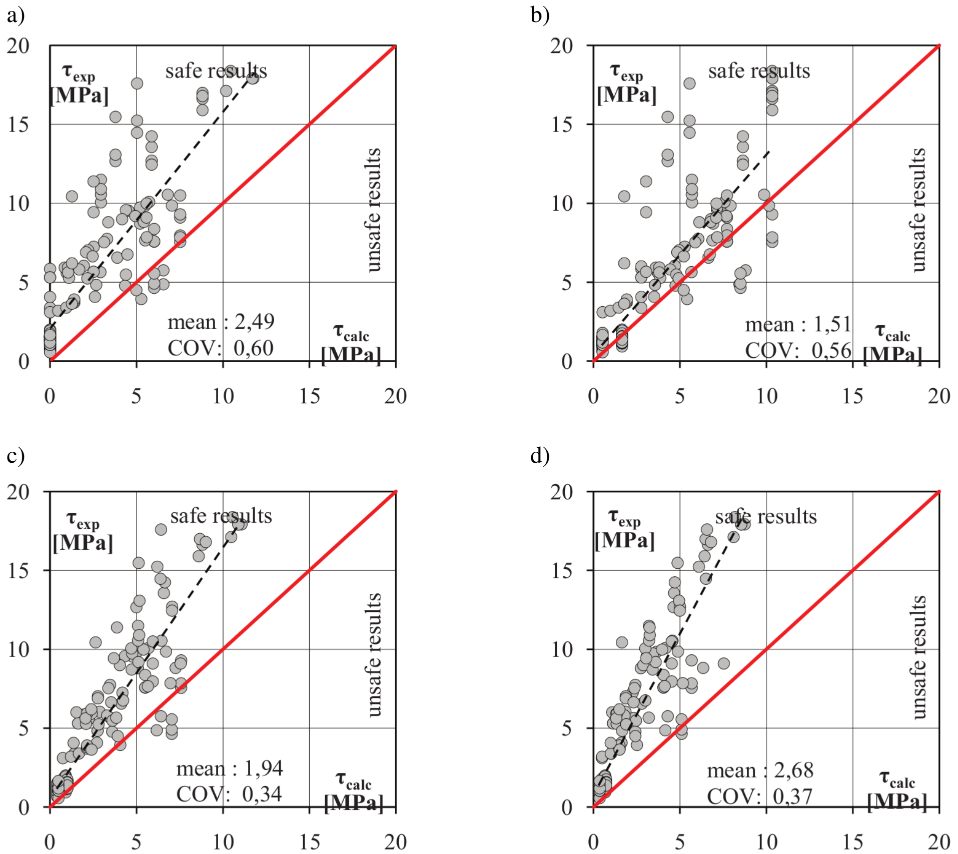

Fig. 9. Comparison between results of the tests and predictions of the codes of practice: a) ACI 318-19 [2], b) AASHTO-LRFD [1], c) EN 1992-1-1 [7], d) prEN 1992-1-1 [5]

In case of calculations according to EN 1992-1-1 [7], prEN 1992-1-1 [5] and ACI 318-19 [2], the vast majority of results were on the safe side ($\tau_{\text{exp}} > \tau_{\text{calc}}$). The results of calculations according to EN 1992-1-1 and prEN 1992-1-1, however, should be considered as conservative, especially in case of contacts characterized by high experimental loads. The mean $\tau_{\text{calc}}/\tau_{\text{exp}}$ ratio was 1.94 and 2.68, for the procedure [7] and [5] respectively. Predictions of the ACI 318-19 [2] standard also turned out to be quite conservative, especially in the case of elements without shear reinforcement, when theoretical shear resistance

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was equal to zero. The lowest difference between results of the tests and calculations was obtained for the AASHTO-LRFD [1] procedure ($\tau_{\text{calc}}/\tau_{\text{exp}}$ = 1.51), which, however, did not prove its better compliance. A significant number of points was on the unsafe side and the results were characterized by a relatively high scatter (COV = 56%). The reasons for this should be seen primarily in the arbitrarily determined value of the adhesion forces, dependent only on the surface classification and not related to the strength parameters of substrate and overlay concrete.

4. Conclusions

Previous experimental investigations demonstrated that the shear transfer mechanism between concretes cast at different times is a complex issue. The surface profile (roughness) and the intensity of the transverse reinforcement are among the most important factors affecting the shear resistance and behaviour of the interface. Particularly useful from a practical point of view are recommendations for the classification of surfaces based on measurable parameters – mean peak height R_{pm} and the mean peak-to-valley height R_z. Increasing the roughness of the substrate leads to an increase in the shear resistance, although the aggregate composition is also important in this respect. In some cases (lightweight aggregate concretes, high-strength concretes, recycled aggregate concretes), aggregate crushing and breaking may occur, which reduces the effect of aggregate interlock.

Reinforcement crossing the interface inhibits the crack opening and limits the mutual displacement of the contact surface – provided that the reinforcement is sufficient to carry tensile forces occurred after cracking of the interface. Strain measurements showed that in most cases shear reinforcement do not yield at cracking – the activation of the reinforcement requires slip at the interface when the adhesive forces are no longer active. For this reason, the proposed changes, introduced in the prEN 1992-1-1 [5], consisting in distinguishing between rigid and non-rigid behaviour and taking into account different mechanisms when determining the ultimate stress, seem justified.

The existing design procedures apply various approaches for calculating shear resistance, based on the shear-friction theory only or taking into account the contribution of adhesion, friction, aggregate interlock and dowel action. The comparison between results of the tests and calculations showed, however, that shear transfer mechanism has still not been described precisely enough. Predictions of design procedures are generally safe, but often lead to conservative results, which may be due to a generalized approach based on a categorized assessment of the surface preparation.

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Nośność styków pomiędzy betonami układanymi w różnym czasie w świetle wyników badań eksperymentalnych i procedur projektowych

Słowa kluczowe: naprężenia graniczne, shear-friction, badanie push-off, styk w betonie, beton układany w różnym czasie, nadbeton

Streszczenie:

W artykule przedstawiono wybrane zagadnienia związane z nośnością styków pomiędzy betonami układanymi w różnym czasie. Konieczność zapewnienia właściwego zespolenia pomiędzy "starym" i "nowym" betonem zachodzi nie tylko w przypadku konstrukcji nowo wznoszonych, w których stosuje się elementy prefabrykowane, lecz także w obiektach już istniejących, gdy zachodzi potrzeba wzmocnienia konstrukcji.

Tematyka nośności styków pomiędzy betonami układanymi w różnym czasie stanowi przedmiot badań eksperymentalnych prowadzonych od lat 60. ubiegłego wieku. Za pierwszy model opisujący zachowanie styków uznaje się teorię *shear-friction*, przedstawioną przez *Masta* i opisaną szczegółowo przez *Birkelanda* i *Birkelanda*. Model ten stosowany jest do dnia dzisiejszego w procedurach obliczeniowych ACI 318 i AASHTO-LRFD, jednak badania eksperymentalne prowadzone na przestrzeni lat wykazały potrzebę rewizji przyjętych założeń, polegających m.in. na uwzględnieniu sił adhezji czy kompozycji stosu okruchowego.

Dotychczasowe badania, prowadzone m.in. na elementach typu *push-off* i belkach zespolonych, pozwoliły na scharakteryzowanie parametrów, które w głównej mierze wpływają na nośność i zachowanie styków pomiędzy betonami układanymi w różnym czasie. Można do nich zaliczyć: wytrzymałość betonu, przekrój zbrojenia przecinającego powierzchnię styku, kompozycję stosu okruchowego, szorstkość powierzchni zespolenia, lokalizację styku na wysokości belki a także relację pomiędzy sztywnością elementu pierwotnego i warstwy nadbetonu. Jak wykazały dotychczasowe badania, siły adhezji są proporcjonalne do wytrzymałości betonu na rozciąganie (ściskanie). Wpływ wytrzymał ości betonu jest jednak warunkowany sposobem przygotowania powierzchni. Wytrzymałość betonu może odgrywać także istotną rolę w przypadku styków silnie zbrojonych, bowiem determinuje ona nośność ukośnych krzyżulców ściskanych. Dlatego też w niektórych pracach i procedurach obliczeniowych zaleca się ograniczenie naprężeń stycznych do wartości $0,2\div0.3 f_c$.

W literaturze panuje zgodny pogląd, że zwiększanie ilości zbrojenia poprzecznego prowadzi do wzrostu nośności styku pomiędzy betonami, co wynika głównie z ograniczania rozwoju szerokości rysy (*clamping effect*) i hamowania wzajemnego poślizgu powierzchni styku (*dowel acton*). Wyniki większości dotychczasowych badań wykazały jednak, iż zbrojenie włącza do współpracy dopiero po zarysowaniu styku, bowiem jego aktywacja wynika m.in. z rozwierania styku. *Randl* i *Wicke* stwierdzili, że w momencie zarysowania naprężenia osiągały z reguły około 50% granicy plastyczności. Z tego względu w wielu pracach zaleca się rozdzielanie siładhezji i udziału zbrojenia poprzecznego, jako mechanizmów wzajemnie się wykluczających. Intensywność zbrojenia poprzecznego wpływa przede wszystkim na zachowanie styku po zarysowaniu – warunkuje ona spadek siły, który może sięgać od kilku do nawet kilkudziesięciu procent, a także nośność resztkową i zdolność styku

do deformacji. W przypadku styku niezbrojonego zarysowanie jest jednoznaczne ze zniszczeniem połączenia. Podobna sytuacja ma miejsce w stykach słabo zbrojonych, gdy bezpośrednio po zarysowaniu dochodzi do zerwania prętów. Mimo tego, że nośność zależna jest przede wszystkim od siładhezji, styk wykazuje zachowanie ciągliwe. W stykach umiarkowanie zbrojonych bezpośrednio po zarysowaniu następuje spadek rejestrowanej siły, przy czym następnie obserwuje się poślizg przy stałym lub nieznacznie zwiększającym się obciążeniu rezydualnym natomiast zniszczenie styku jest poprzedzone stopniowym zrywaniem kolejnych prętów. W elementach ze stykami bardzo silnie zbrojonymi zniszczenie może być natomiast następstwem zniszczenia betonu (miażdżenie ukośnych krzyżulców), utraty zakotwienia lub zerwania zbrojenia poprzecznego. Co istotne, badania *Julio* i in. wykazały, że sposób osadzenia zbrojenia poprzecznego wpływa jedynie w bardzo ograniczonym stopniu na nośność połączenia – stwierdzono różnicę nośności styków ze zbrojeniem ułożonym przed zabetonowaniem elementu podłoża i styków w modelach z prętami poprzecznymi wklejanymi w stwardniały beton sięgającą około 7÷8%.

Skład granulometryczny kruszyw stosowanych w betonie wpływ na efekt zazębiania a tym samym na tzw. kohezję mechaniczną. W miarę rozwoju szerokości rysy siły przyczepności ulegają zmniejszeniu, co wynika ze spadku udziału drobniejszych frakcji kruszywa w mechanizmie zazębiania. Okazuje się jednak, że duży wpływ na nośność i zachowanie styku mają również właściwości mechaniczne kruszywa, co uwidacznia się szczególnie w przypadku lekkich betonów kruszywowych – badania *Mattocka* i in. pokazały, że przy tej samej intensywności zbrojenia poprzecznego graniczne naprężenia styczne mogą być niższe nawet o 3÷34% względem betonu zwykłego. W betonach o wysokiej wytrzymałości i lekkich to kruszywo stanowi najsłabszy element kompozytu, który może ulegać pękaniu, osłabiając tym samym efekt zazębiania kruszywa. Podobna sytuacja może mieć miejsce w betonach z kruszywem z recyklingu, w których pierwsze rysy formują się obrębie stref styku starej zaprawy i kruszywa co przekłada się na mniejszą efektywną średnicę ziaren i osłabienie efektu zazębiania kruszywa. Skutkuje to obniżeniem sił rysujących o 12 do 20% i niszczących o 7 do 20% w zależności od stopnia zastąpienia kruszywa naturalnego kruszywem z odzysku.

Zwiększenie szorstkości powierzchni prowadzi do wzrostu nośności styków pomiędzy betonami układanymi w różnym czasie, co wiąże się przede wszystkim z udziałem mechanizmu zazębiania kruszywa. Najlepsze efekty uzyskiwano w przypadku piaskowania, frezowania oraz stosowania strumienia wody pod wysokim ciśnieniem. Inwazyjne metody obróbki powierzchni takie jak np. groszkowanie mogą jednak powodować osłabienie struktury betonu podłoża i tym samym prowadzić do obniżenia nośności styku, szczególnie w przypadku, gdy nadkład charakteryzuje się większą wytrzymałością niż beton podłoża. W pracy *Mohamada* i in. przeanalizowano 14 parametrów charakteryzujących profil powierzchni przy czym za najbardziej trafne uznano średnią gł ębokość R_{pm} oraz średnią wysokość grzbietu względem doliny R_z.

W celu oceny procedur projektowych AASHTO-LRFD, ACI 318-19, EN 1992-1-1 i prEN 1992-1-1 porównano teoretyczne nośności styków z wynikami 184 badań eksperymentalnych na elementach typu *push-off*, wykonanych z betonu o wytrzymałości na ściskanie f_{cm}18÷112 MPa i charakteryzujące się zbrojeniem ρ_s0÷2.64%. Stwierdzono, że większość metod pozwala na bezpieczne oszacowanie nośności styków – zbrojonych jak i bez zbrojenia poprzecznego. W przypadku obliczeń wedł ug EN 1992-1-1, prEN 1992-1-1 oraz ACI 318-19 zdecydowana większość wyników znalazła się po stronie bezpiecznej. Wyniki obliczeń według wymienionych procedur należy jednak ocenić jako konserwatywne, szczególnie w przypadku styków charakteryzujących się w badaniach wysoką nośnością. W zależności od rozważanej procedury obliczeniowej uzyskano średnic stosunek nośności eksperymentalnej τ_{exp} do teoretycznej τ_{calc} równy 1.51÷2.68. Najmniejszą średnią różnicę pomiędzy wynikami badań i obliczeń uzyskano w przypadku procedury AASHTO-LRFD ($\tau_{\mathrm{calc}}/\tau_{\mathrm{exp}}$ = 1.51), co jednak nie świadczyło o lepszym sformułowaniu jej zasad. Znaczna liczba wyników znalazła

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się bowiem po stronie niebezpiecznej a rezultaty charakteryzował stosunkowo duży rozrzut (COV = 56%), czego przyczyn upatrywać należy przede wszystkim w arbitralnie ustalonej wartości sił adhezji, zależnych wyłącznie od klasyfikacji podłoża i niepowiązanych z parametrami wytrzymałościowymi betonu podł oża i nadkładu. Przeprowadzone obliczenia wykazały, że mechanizm transferu siłnie zostałdotychczas sformułowany wystarczająco precyzyjnie w procedurach projektowych, co może wynikać m.in. z uogólnionego podejścia bazującego na kilkustopniowej ocenie warunków podłoża.

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