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Interface Shear Transfer on Composite Concrete Members

by Pedro M. D. Santos and Eduardo N. B. S. Júlio

The "shear-friction theory" was adopted in design codes to predict the longitudinal shear strength between parts of concrete members cast at different times. This is a relevant subject for different situations, such as the connection between precast members with cast-in-place parts and strengthening of existing reinforced concrete (RC) members with a new concrete layer.

The bond strength of the interface is controlled by parameters such as the substrate roughness, curing conditions, and the material strength and stiffness of both concrete layers, among others. Some of these—for example, differential shrinkage due to different curing conditions, and differential stiffness due to the difference in the Young's modulus of each layer—are not addressed in codes.

This paper presents recommendations and proposes an alternative design approach for concrete-to-concrete interfaces that can be adopted in future revisions of ACI 318 and Eurocode 2. A comparison between ACI 318 and Eurocode 2 is presented.

Keywords: codes; interface; repair; roughness; shear; shrinkage; standards; stiffness.

INTRODUCTION

The "shear-friction theory" is currently adopted in all major design codes of concrete structures, such as ACI 318,[1] Eurocode 2,[2] and *fib* Model Code 2010,[3] among others, to predict the longitudinal shear strength between concrete layers cast at different times. It is most adequate for designing precast members with cast-in-place parts as well as existing concrete members—for example, bridge decks strengthened with a concrete overlay.

This theory was initially proposed by Birkeland and Birkeland[4] to predict the shear strength of new-to-old concrete interfaces. Currently, the following four parameters are considered: 1) the compressive strength of the weakest concrete; 2) the normal stress at the interface; 3) the shear reinforcement crossing the interface; and 4) the roughness of the substrate surface. The design philosophy assumes that, due to relative slippage between old and new concrete layers, the interface crack width increases, leading to shear reinforcement yielding in tension, thus compressing the interface, and the shear forces are transferred by friction. A "saw-tooth model" is usually adopted to exemplify this concept (Fig. 1).

Because composite concrete structures are the main field of application of the shear-friction theory, parameters such as the differential shrinkage and the differential stiffness should be considered in design. Nevertheless, current design codes neglect these two parameters and, therefore, the proposed design expressions cannot be as accurate as desired.

This paper presents the second part of a large experimental study conducted to assess the longitudinal shear strength between concrete layers cast at different times. An overview of the shear-friction theory is made. The provisions of ACI 318[1] are discussed and compared with those in Eurocode 2.[2] Finally, the authors propose an alternative design approach for concrete-to-concrete interfaces that can be incorporated in future revisions of ACI 318[1] and Eurocode 2.[2]

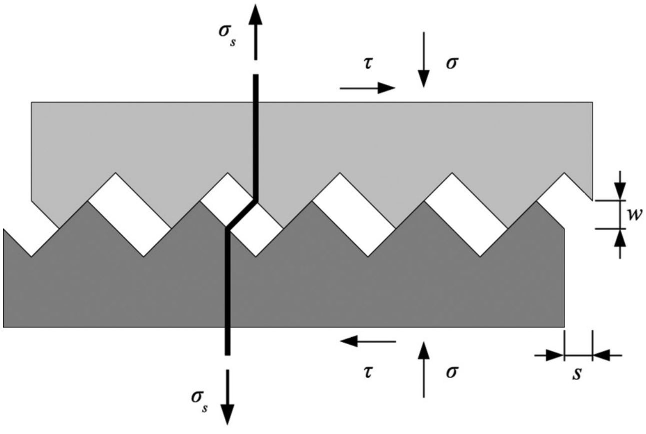

Fig. 1—Saw-tooth model.

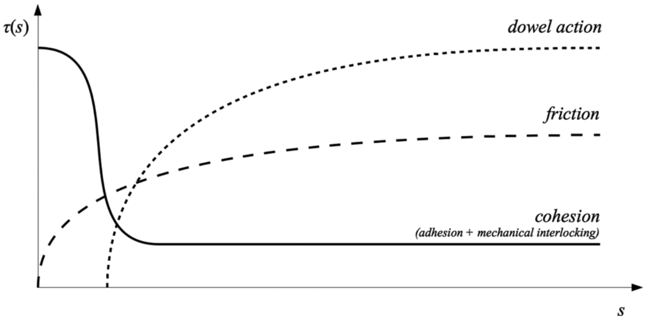

Fig. 2—Load transfer mechanisms.

RESEARCH SIGNIFICANCE

The load transfer mechanism of shear forces between concrete parts cast at different times (Fig. 2) results from the combination of three components: cohesion, friction, and dowel action. While cohesion is considered by some design codes, dowel action is usually neglected. Friction is the only component present in all shear-friction design provisions.

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The bond strength is controlled by parameters such as surface preparation, weakest concrete strength, shear reinforcement, differential shrinkage, and differential stiffness. The authors experimentally investigated the influence of these parameters on the behavior of composite reinforced concrete (RC) members. An alternative design approach for concrete-to-concrete interfaces is proposed.

DESIGN EXPRESSIONS FROM RESEARCH

The design expression of the shear-friction theory, proposed by Birkeland and Birkeland,[4] for the assessment of the ultimate longitudinal shear stress at concrete-to-concrete interfaces is given by Eq. (1). The coefficient of friction depends on the surface preparation method and assumes the following values: 1) $\mu = 1.7$ for monolithic concrete; 2) $\mu = 1.4$ for artificially roughened construction joints; and 3) $\mu = 0.8$ to 1.0 for ordinary construction joints and for concrete-to-steel interfaces.

$$v_u = \mu\rho f_y \tag{1}$$

Several researchers[5] suggested modifications to this design expression to increase its accuracy and to include other parameters, such as cohesion of the interface (corresponding to adhesion and aggregate interlock); the weakest concrete strength; and the dowel action due to the deformation (by shear, bending, and tension) of reinforcing bars. The most significant contributions are presented in the following paragraphs.

Mattock and Hawkins[6] proposed the design expression (Eq. (2) and (3)) usually known as the “modified shear-friction theory.” The first term explicitly gives the contribution of the cohesion of the interface, assumed constant and equal to 1.38 MPa (200 psi), while the second term is due to clamping stresses. The coefficient of friction is considered constant and equal to 0.8.

$$v_u = 1.38 + 0.8(\rho f_y + \sigma_n) \quad \text{(MPa)} \tag{2}$$

$$v_u = 200 + 0.8(\rho f_y + \sigma_n) \quad \text{(psi)} \tag{3}$$

Loov[7] proposed the first design expression (Eq. (4)) to explicitly include the concrete strength. The constant *k* was considered equal to 0.5 for initially uncracked interfaces. The proposed design expression can also be used with any consistent system of units (SI or U.S. Customary).

$$v_u = k\sqrt{f_c\left(\rho f_y + \sigma_n\right)} \tag{4}$$

Walraven et al.[8] proposed an expression using an innovative approach known as the “sphere model” to take into account the interaction between the aggregates, the binding paste, and the interface zone. The resulting nonlinear design expression (Eq. (5) to (10)) was calibrated using the results of a large experimental study conducted using push-off specimens with initially cracked interfaces

$$v_u = C_1\left(\rho f_y\right)^{C_2} \quad \text{(MPa)} \tag{5}$$

$$C_1 = 0.822 f_c^{0.406} \quad \text{(MPa)} \tag{6}$$

$$C_2 = 0.159 f_c^{0.303} \quad \text{(MPa)} \tag{7}$$

$$v_u = C_1\left(0.007\rho f_y\right)^{C_2} \quad \text{(psi)} \tag{8}$$

$$C_1 = 15.686 f_c^{0.406} \quad \text{(psi)} \tag{9}$$

$$C_2 = 0.0353 f_c^{0.303} \quad \text{(psi)} \tag{10}$$

Tsoukantas and Tassios[9] were the first researchers to refer to and study the contribution of the dowel action mechanism to the total shear strength of the interface. Later, Randl[10] proposed a design expression (Eq. (11)) that explicitly includes the contribution of the three load transfer mechanisms[11]: 1) *cohesion*, due to the contribution of adhesion and aggregate interlocking; 2) *friction*, due to the longitudinal relative slip between concrete layers and thus influenced by the surface roughness and the normal stress at the shear interface; and 3) *dowel action*, due to the contribution of the flexural resistance of the shear reinforcement crossing the interface.

$$v_u = \underbrace{cf_c^{1/3}}_{\text{cohesion}} + \underbrace{\mu\left(\rho k f_y + \sigma_n\right)}_{\text{friction}} + \underbrace{\alpha\rho\sqrt{f_y f_c}}_{\text{dowel action}} \le \beta\nu f_c \tag{11}$$

The parameters of the design expression proposed by Randl[10] are presented in Table 1. Randl[10] also proposed that surface roughness should be evaluated using ASTM E965(2001)[12] by means of the Sand Patch Test.

DESIGN EXPRESSIONS FROM CODES

ACI 318[1] considers that a crack across a given plane can occur due to an existing or potential crack, an interface between different materials, or an interface between two concretes cast at different times. The ultimate longitudinal shear stress at the concrete-to-concrete interface is given by the contribution of friction (Eq. (12)). Cohesion and dowel action are not explicitly considered.

$$v_u = \rho f_y(\mu\sin\alpha + \cos\alpha) \tag{12}$$

Four surface conditions are considered: 1) concrete placed against hardened concrete with the surface clean but not intentionally roughened ($\mu = 0.6\lambda$); 2) concrete placed against hardened concrete with the surface clean and intentionally roughened to a full amplitude of 6.35 mm (0.25 in.)

Table 1—Values of design parameters proposed by Randl[10]

Surface preparation	Surface roughness R, mm	Coefficient of cohesion c	Coefficient of friction μ		k	α	β
			$f_c \geq$ 20 MPa	$f_c \geq$ 35 MPa			
High-pressure water-blasting	$\geq$ 3.0	0.4	0.8	1.0	0.5	0.9	0.4
Sand-blasting	$\geq$ 0.5	0.0	0.7	0.7	0.5	1.1	0.3
Smooth	—	0.0	0.5	0.5	0.0	1.5	0.2

Notes: 1 mm = 0.039 in.; 1 MPa = 145 psi.

($\mu = 1.0\lambda$); 3) concrete placed monolithically ($\mu = 1.4\lambda$); and 4) concrete anchored to as-rolled structural steel by headed studs or reinforcing bars ($\mu = 0.7\lambda$). The parameter λ is a modification factor related to concrete density and shall be taken equal to 1.00 for normalweight concrete, and 0.75 for all lightweight concrete. When normalweight and lightweight aggregates are used together, the modification factor λ should be determined considering the volumetric proportions of each aggregate type, without exceeding 0.85.

According to ACI 318,[1] for normalweight concrete placed monolithically or cast against hardened concrete with surface intentionally roughened, the ultimate longitudinal shear stress is upper limited by the minimum value given by $0.2f_c$, $(3.3 + 0.08f_c)$ MPa ($(480 + 0.08f_c)$ psi) and 11 MPa (1600 psi). For the remaining cases, the ultimate longitudinal shear stress is upper limited by the minimum value given by $0.2f_c$ and 5.52 MPa (800 psi). The yield strength of the reinforcement shall not be taken greater than 414 MPa (60,000 psi).

Eurocode 2[2] adopted a design expression (Eq. (13)) that explicitly considers the contribution of cohesion and friction. Dowel action is not explicitly considered. The first term is linked to the contribution of cohesion; the second term is related to the contribution of friction due to external compressive stresses; and the third term, also related to friction, gives the contribution of the compression due to the tensioned shear reinforcement that crosses the interface.

$$v_u = cf_{ctd} + \mu\sigma_n + \rho f_y(\mu\sin\alpha + \cos\alpha) \leq 0.5\nu f_{cd} \qquad (13)$$

The contribution of cohesion is variable because it is a function of the weakest concrete strength. Four conditions are adopted for the surface roughness: *very smooth*; *smooth*; *rough*; or *indented*. The *very smooth* surface is considered as a surface cast against steel, plastic, or specially prepared wooden molds ($c = 0.025$ to 0.1; $\mu = 0.5$). The *smooth* surface is a slipformed or extruded surface or a free surface left without further treatment after vibration ($c = 0.2$; $\mu = 0.6$). The *rough* surface is a surface that has at least 3 mm (0.12 in.) roughness at approximately 40 mm (1.57 in.) spacing, achieved by raking, exposing of aggregate, or other methods giving an equivalent behavior ($c = 0.4$; $\mu = 0.7$). The *indented* surface is a surface with indentations (shear keys) complying with a specific geometry defined by the code ($c = 0.5$; $\mu = 0.9$).

Comparing the shear-friction provisions of both ACI 318[1] and Eurocode 2,[2] it can be stated that the surface roughness is the most influential parameter on the shear strength. It is qualitatively assessed in all design codes[1,2] by adopting a visual inspection because no method or device is proposed to perform a quantitative assessment. Nevertheless, it must be highlighted that the new *fib* Model Code 2010[3] already considers the use of a roughness parameter (average roughness R_a) to characterize the surface roughness. Each design code presents its own roughness classification, leading to different values of both coefficients of cohesion and friction, thus leading to different values of the bond strength of the interface for the same surface condition.

A needed improvement to all design codes is the adoption of a common quantitative approach to characterize the surface roughness, instead of using a purely qualitative assessment. It will then be possible to increase the design accuracy because both coefficients of cohesion and friction are determined for each particular situation using the same methodology and assessment principles. Moreover, it will be possible to avoid considering minimum values for roughness amplitude, as is currently proposed by some design codes.[1,2]

Finally, it should be mentioned that the absence of any provision concerning both differential shrinkage and differential stiffness is common to all design codes. Because the interface shear strength is of major significance in composite RC members, design codes should mention, at least, the need to consider the influence of both differential shrinkage and differential stiffness in the design, as these generate high stresses at the interface. Moreover, both parameters should be evaluated for each specific condition on site.

RECENT INVESTIGATIONS ON INTERFACE SHEAR TRANSFER

Aiming to overcome some weaknesses present in the shear-friction provisions of design codes, the authors developed a nondestructive in-situ method[13] to perform a quantitative assessment of the roughness of a concrete surface. This is based in an innovative measuring device, called 2D Laser Roughness Analyser (2D-LRA method), specifically developed with this aim.

This new method proved to be effective because it is possible to: 1) obtain two-dimensional (2-D) profiles of the surface texture; 2) compute roughness parameters from these; and 3) to correlate the latter with the bond strength of the concrete-to-concrete interface, both in shear and in tension, with high coefficients of correlation ($R^2 > 0.85$). Furthermore, the method combines all the advantages, with even higher accuracy, and overcomes all the disadvantages of existing methods[14] such as: 1) the Sand Patch Test proposed by the ASTM E965[12]; 2) the Concrete Surface Profiles (ICRI)[15]; and 3) the first method developed by the authors.[16] The method can be used to implement a quality control scheme on the preparation of concrete surfaces that will receive a new concrete layer, both in industrial facilities and on site.

Subsequently, a large experimental study[17] was developed to characterize the influence of: 1) substrate surface preparation; 2) differential shrinkage; and 3) differential stiffness,

Table 2—Constituents of concrete mixture

Constituent	Diameter range, mm (in.)	Quantity, kg (lb)
Fine sand	0.074 to 9.52 (0.0029 to 0.375)	295 (650)
Coarse sand	0.074 to 9.52 (0.0029 to 0.375)	640 (1421)
Fine limestone	1.19 to 19.1 (0.047 to 0.752)	375 (827)
Coarse limestone	4.76 to 19.1 (0.187 to 0.752)	545 (1202)
Portland cement type I 52.5R	—	350 (772)
Commercial admixture	—	3.675 (8.102)
Water	—	150 (331)

Table 3—Compressive strength and age of concrete at test date

Series	Layer	Compressive strength		Age at test, days
		Average, MPa (ksi)	Coefficient of variation, %	
L28	Substrate	79.3 (11.5)	6.03	56
	Added	66.4 (9.6)	3.18	28
L56	Substrate	86.0 (12.5)	3.50	84
	Added	80.5 (11.7)	1.12	28
L84	Substrate	86.4 (12.5)	2.29	112
	Added	72.6 (10.5)	5.27	28
E28	Substrate	78.9 (11.5)	6.88	56
	Added	68.3 (9.9)	1.05	28
E56	Substrate	77.6 (11.3)	1.98	84
	Added	71.1 (10.3)	5.30	28
E84	Substrate	81.9 (11.9)	0.30	112
	Added	69.9 (10.1)	6.17	28

on the bond strength of the interface between concrete layers cast at different times.

The first part of this research was recently published.[18] The second part, presented herein, gives an alternative design approach for practicing engineers. For this reason, only a short description of the experimental study is presented herein, including: 1) materials used; 2) adopted bond tests; 3) surface preparation techniques; 4) difference of ages between concrete layers; and 5) curing conditions, among others. The most significant results are highlighted to support the conclusions drawn and the recommendations made ahead. For the presented reasons, more detailed data and background information can be found in previous works published by the authors.[17,18]

The constituents of the adopted concrete mixture are given in Table 2. A commercial admixture was used to increase the hardened concrete's initial strength by reducing the water needed but keeping the workability of the fresh concrete. The maximum size of the aggregates was 19.1 mm (0.75 in.), the predicted void volume was 20*l*, and the predicted compacity was 0.822. The compressive strength of both concrete layers is given in Table 3.

The slant shear test and the splitting test were selected to assess the bond strength of the concrete-to-concrete interface, in shear and in tension, respectively. For the slant shear test[19] (Fig. 3(a), a prismatic specimen of 150 x 150 x 450 mm^3 (5.91 x 5.91 x 17.72 in.^3) was adopted with the shear plane at 30 degrees to the vertical. For the splitting test[20] (Fig. 3(b), cubic specimen of 150 mm (5.91 in.) with the interface at middle height was adopted. Cubic specimens of 150 mm (5.91 in.) were adopted for standard quality control.[21]

Fig. 3—Concrete specimens: (a) slant shear; and (b) splitting.

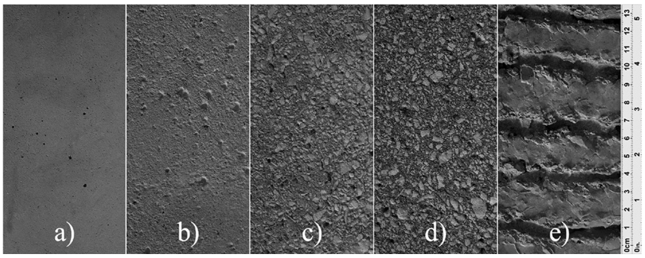

Fig. 4—Surface preparation: (a) left as-cast; (b) wire-brushing; (c) sand-blasting; (d) shot-blasting; and (e) hand-scrubbed. (Note: Pictures at scale.)

The interface surface between the substrate and the added concrete layer was prepared using different methods (Fig. 4). Left-as-cast (LAC) against steel formwork was considered the reference situation. The roughness of the hardened concrete was increased by: 1) wire-brushing (WB); 2) sand-blasting (SAB); and 3) shot-blasting (SHB). Hand-scrubbing (HS), a technique commonly used to prepare fresh concrete members in the precast industry (also known as "raking") was also considered.

After surface preparation, the roughness was quantified using the 2D-LRA method.[13] An evaluation length of 150 mm (5.91 in.) was considered and 10 texture profiles were measured for each surface condition. Several roughness parameters were directly computed from the primary profile—that is, without filtering.[22] The roughness parameter, mean valley depth R_{vm}, was adopted to be correlated with the bond strength of the concrete-to-concrete interface because it was the one that presented the highest coefficient of correlation ($R^2 > 0.95$) and is based in average values of the profile, which allows the reduction of the influence of the surface irregularities such as air voids and overly exposed aggregates. Mean valley depth R_{vm} is given by

$$R_{vm} = \frac{1}{5}\sum_{i=1}^{5} v_i \tag{14}$$

Table 4—Mean valley depth R_{vm}

Surface preparation	Mean valley depth R_{vm}	
	Average, mm (in.)	Coefficient of variation, %
LAC	0.056 (0.002204)	24.1
Wire-brushing	0.182 (0.007165)	14.1
Sand-blasting	0.771 (0.030354)	15.1
Shot-blasting	0.809 (0.031850)	21.4
Hand-scrubbing	3.792 (0.149213)	14.6

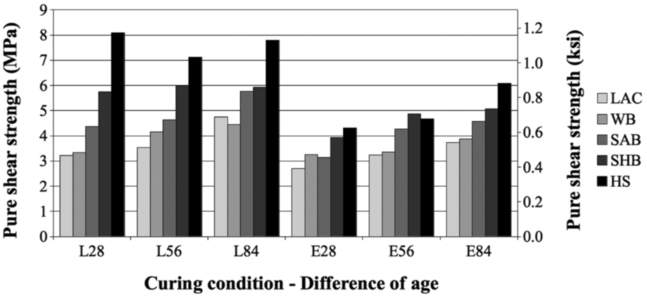

Fig. 5—Shear strength of specimens.

The average and the coefficient of variation of R_{vm} for each surface condition are given in Table 4.

To maximize the differential shrinkage between concrete layers, three different situations were adopted for the time gap between casting the substrate and the added concrete layer: 28, 56, and 84 days. Two different curing conditions were also considered: 1) inside the laboratory (L series, named L28, L56, and L84) under normal conditions of work; and 2) exterior conditions (E series, named E28, E56, and E84) with the specimens directly exposed to environmental conditions such as solar radiation, rain, and wind. Temperature and relative humidity were recorded in both situations.[18]

A total of 300 specimens were produced, adopting five slant shear specimens and five splitting specimens for each considered situation (varying the surface preparation, the difference of age between the substrate and the added concrete, and the curing condition). The hand-scrubbed surfaces were prepared while concrete was still fresh, whereas the remaining surface treatments were executed on hardened concrete. After demolding, each half-specimen was subjected to one of the surface treatments described, cleaned with compressed air to remove dust, and stored again under the specified curing conditions. Twenty-eight days later, a new concrete layer was added and specimens were stored again under the same curing conditions. When the added concrete reached 28 days of age, the specimens were tested in compression using a standard testing machine.[19-21]

Two distinctive failure modes were observed: adhesive, by interface debonding; and cohesive, by crushing of the concrete bulk. All splitting specimens presented adhesive failures while the slant shear specimens presented both adhesive and cohesive failures.

The results obtained with the tested concrete specimens cannot be directly compared because, besides the differences between failure modes, small differences between concrete strengths were also measured because several batches were made to cast all specimens. For this reason, the Mohr-Coulomb criterion was adopted and the pure shear strength of the interface—that is, without normal stress acting at the interface—was computed for all slant shear specimens (Fig. 5). The failure envelope of the interface was defined using the experimental values of the bond strength in shear and in tension and assessed using the slant shear test and the splitting test, respectively. The failure stress in compression, obtained for the specimens presenting a cohesive failure, was also used to define the failure envelope of these specimens. Then, for each specimen, the Mohr circle was determined and the pure shear strength computed. The average of each set of five specimens is considered in the analysis.

It can be concluded that the bond strength of the interface, herein assumed as the pure shear strength, increases with the increase of the surface roughness for both curing conditions. It can also be observed that the bond strength increases with the increase of the difference of ages between the substrate concrete and the added concrete layer, also for both curing conditions. These results were not expected because the bond strength of the interface is supposed to decrease with the increase of differential shrinkage, which obviously increases with the difference of ages between concrete layers. To analyze these experimental observations, a numerical study[17,18] was conducted. This corroborated and explained the unexpected experimental results. The increase of the bond strength with time is due to a particular stress state at the interface of the slant shear specimen, induced by differential shrinkage, of opposite sign of the stress state induced by loading.

The adopted curing conditions were also revealed to have a significant influence. The specimens stored in the exterior of the laboratory led, as expected, to lower values of the pure shear strength, with an average decrease of 1.12 MPa (162 psi), corresponding to an average decrease of 19%. It must be highlighted that, although the average values were similar, the daily fluctuations of temperature and relative humidity were quite distinctive.[18]

The need to use several batches to cast all specimens led to small differences in the compressive strength at the test date (Table 3) and, consequently, to a different Young's modulus of each concrete layer. Therefore, a differential stiffness was produced on the composite concrete member.

The experimental observations corroborated previous studies[23,24] because broken corners were obtained at both ends of the interface, located precisely where stress concentrations are observed due to the increase of the differential stiffness. An increase in the number of cohesive failures with the increase of the difference between the Young's modulus of both concrete layers, and with the increase of the surface roughness of the interface, was observed.[18] This means that a composite RC member can be designed to present a monolithic behavior by acting on the differential stiffness and interface surface preparation.

PROPOSED DESIGN APPROACH

Based on the shear-friction provisions of current design codes[1-3] and taking into account the studies conducted by

Table 5—Coefficients of cohesion and friction

Series	Surface preparation method									
	LAC		WB		SAB		SHB		HS	
	c	μ	*c*	μ	*c*	μ	*c*	μ	*c*	μ
L28	0.71	1.18	0.74	1.25	0.97	1.42	1.27	1.48	1.79	1.50
L56	0.68	1.09	0.80	1.30	0.89	1.39	1.15	1.46	1.37	1.45
L84	0.98	1.41	0.92	1.43	1.19	1.46	1.23	1.48	1.61	1.50
E28	0.59	1.10	0.70	1.15	0.68	1.14	0.85	1.24	0.93	1.26
E56	0.68	1.14	0.71	1.22	0.90	1.29	1.02	1.32	0.98	1.39
E84	0.79	1.30	0.83	1.31	0.97	1.34	1.08	1.38	1.30	1.44
Average	0.74	1.21	0.78	1.28	0.93	1.34	1.10	1.39	1.33	1.42
Coefficient of variation, %	18.5	10.5	10.8	7.5	17.7	8.7	13.8	7.1	25.4	6.5

the authors[13,14,16-18,22] and other researchers,[10,23,24] an alternative design approach for concrete-to-concrete interfaces is proposed, aiming to increase both the accuracy and economy of the design.

The design approach proposed herein is based on two basic principles: 1) characterization and measurement of the surface roughness to predict both coefficients of cohesion and friction for each specific surface condition, without imposing a minimum roughness amplitude; and 2) consideration of two different levels of shear stresses at the interface, with and without the need to provide longitudinal shear reinforcement.

Based in the design expression proposed by Eurocode 2[2] (Eq. (13)) and adopting the average values computed for the pure shear strength and the bond strength of the interface in tension (Fig. 6), the coefficient of cohesion (Table 5) corresponding to each specimen series and surface preparation method was determined by

$$c = \frac{v_u}{f_{ctm}} \tag{15}$$

Then, using the computed coefficients of cohesion and considering the average shear and normal stresses at the interface of the slant shear specimens, the coefficient of friction (Table 5) was determined for each specimen series and surface preparation method by

$$\mu = \frac{v_u - cf_{ctm}}{\sigma_n} \tag{16}$$

Adjusting a power function (Fig. 7) to the experimental values from Tables 4 and 5, considering the average of each set of five specimens, it is possible to predict the coefficients of cohesion (Eq. (17) and (18)) and the coefficients of friction (Eq. (19) and (20)) by measuring the surface texture and computing the corresponding R_{vm}. The adopted function was the one that ensures the best coefficient of correlation (R^2)—0.92 and 0.94 for the coefficient of cohesion and the coefficient of friction, respectively.

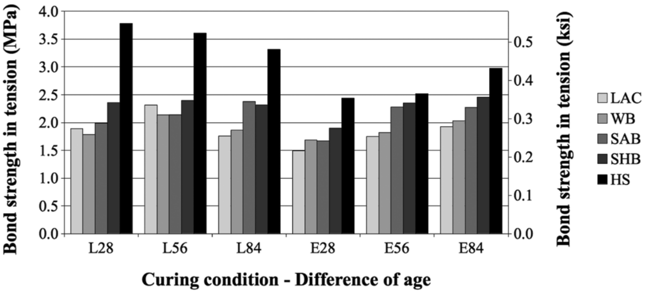

Fig. 6—Bond strength of interface in tension.

$$c_d = \frac{1.06R_{vm}^{0.15}}{\gamma_{coh}} \quad \text{(mm)} \tag{17}$$

$$c_d = \frac{1.70R_{vm}^{0.15}}{\gamma_{coh}} \quad \text{(in.)} \tag{18}$$

$$\mu_d = \frac{1.37R_{vm}^{0.04}}{\gamma_{fr}} \quad \text{(mm)} \tag{19}$$

$$\mu_d = \frac{1.56R_{vm}^{0.04}}{\gamma_{fr}} \quad \text{(in.)} \tag{20}$$

Considering the uncertainty existing on both coefficients, influenced by the surface preparation method and by the concrete properties (namely, the hardness of the aggregates and binding paste), a partial safety factor γ is applied in accordance with the philosophy employed in the Eurocodes. The partial safety factors were determined for the ultimate limit state (ULS), according to the Eurocode 0,[25] adopting

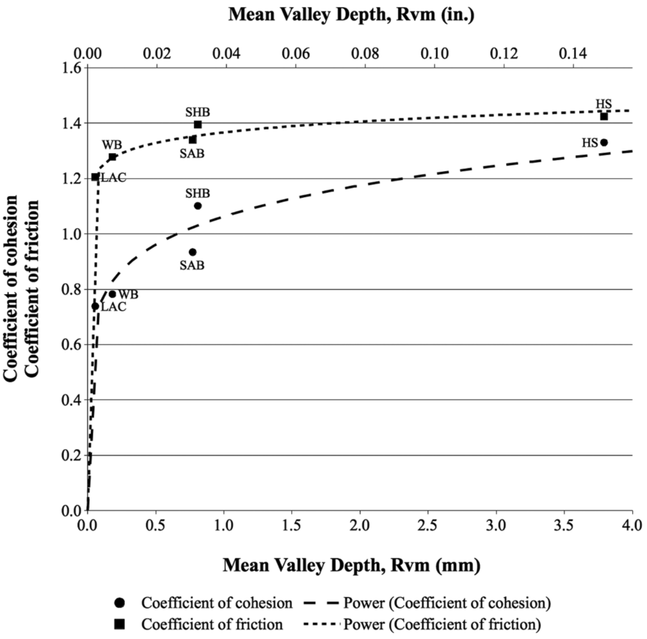

Fig. 7—Correlation between mean valley depth R_{vm} and coefficients of cohesion and friction.

the values of the coefficient of variation calculated for the coefficients of cohesion and friction (Table 5), and are given by

$$\gamma = \frac{\mu - k \cdot \sigma}{\mu - \alpha \cdot \beta \cdot \sigma} = \frac{1 - k \cdot V_R}{\mu - \alpha \cdot \beta \cdot V_R} \qquad (21)$$

where k is taken as 1.65 for the 5% fractile of the normal distribution; α is a weighting factor, taken as 0.8; and β is the reliability factor, taken as 3.8, which corresponds to a probability of failure of 7.5×10^{-5}. Thus, the maximum value of the partial safety factor was adopted—namely, 2.6 and 1.2 for the coefficient of cohesion and coefficient of friction, respectively.

For uniform interface surfaces, the roughness must be measured with a minimum accuracy of 10 micrometers (0.000394 in.), using an evaluation length not less than 100 mm (3.94 in.). Then, R_{vm} has to be determined as the average value obtained considering at least ten 2-D primary profiles. Uniform interface surfaces can be considered as those resulting from as-cast against steel, plastic, or specially prepared wooden molds; slipformed or extruded surface; free surface left without further treatment after vibration; or surfaces prepared by wire-brushing, sand-blasting, shot-blasting, water-blasting, or other equivalent methods. It must be highlighted that the predicted coefficients of friction for smooth surfaces such as LAC, wire-brushed, or lightly sand-blasted/shot-blasted are in agreement with other studies,[26] presenting values that can be near or even higher than 1.4.

It is suggested that when no reinforcement crossing the interface is provided, the ultimate longitudinal shear stress at the concrete-to-concrete interface should be given by

$$v_u = c_d f_{ctd} \le v_{u,max} \qquad (22)$$

In this situation, the ultimate longitudinal shear strength of the interface is given solely by the contribution of cohesion. Relative slippage does not occur between concrete parts and the interface is not cracked.

When reinforcement crossing the interface is provided and/or necessary, the ultimate longitudinal shear stress at the concrete-to-concrete interface is given by

$$v_u = \mu_d \sigma_n + \rho f_y (\mu_d \sin\alpha + \cos\alpha) \le v_{u,max} \qquad (23)$$

In this situation, a relative slippage can occur between concrete parts and, therefore, shear forces are transmitted by friction. The contribution of cohesion is considered null. The reinforcing bars are deformed in shear and bending (due to the relative slippage between concrete layers) and tensioned (due to dilatancy). The latter term is the normal opening of the interface due to the progressive relative slippage. Additional compression due to external loads can also be considered if these are kept constant.

It should be highlighted that for normalweight concrete, when cast against hardened concrete with a roughened surface, the upper limit $v_{u,max}$ proposed by ACI 318[1] is valid.

The proposed methodology is adequate for interface surfaces presenting a uniform texture. For other cases, such as indented surfaces with shear keys, the values of the coefficients of cohesion and friction should be evaluated for each specific case.

Because recent investigations[18] proved that differential shrinkage and differential stiffness can have a significant influence on the shear strength of the interface between concrete parts cast at different times, these effects should at least be mentioned in codes.

Finally, other requirements should also be satisfied. For instance, in cases where the interface can be significantly cracked, the coefficient of cohesion should be taken as zero, for all types of interface surfaces, as a conservative measure. When fatigue or dynamic loads exist on a structure, the design of the interface of the composite member for shear should not consider the contribution of the load transfer mechanism, usually called cohesion. In these conditions, the use of shear reinforcement is compulsory. The contribution of cohesion should not be considered for the shear strength of brackets, corbels, and ledges. Therefore, the use of reinforcement should also be compulsory in these cases.

CONCLUSIONS

From the analysis of codes and published research, it can be concluded that the roughness of the concrete substrate has a significant influence on the bond strength of concrete-to-concrete interfaces. Moreover, it can be stated that in all design codes, this parameter is qualitatively assessed and each one presents its own classification. Furthermore, it can be seen that, even though it is recognized that the load transfer mechanism at the concrete-to-concrete interface is due to cohesion, friction, and dowel action, the latter is not explicitly considered in design codes. The new *fib* Model Code 2010[3] is the first code to present a design expression to predict the interface shear strength as the sum of the three load transfer mechanisms.

Common to all design codes is the absence of any provision related to the curing conditions and, therefore, differential shrinkage. Also neglected are the properties of the added concrete and, thus, differential stiffness is not considered either. The latter is given by the difference between the Young's modulus of each concrete layer of the composite RC member.

Taking into account the weaknesses of the design codes referred to, it is proposed that a quantitative methodology be adopted to avoid the subjective assessment of the surface roughness. For that purpose, the authors developed a measuring device—a 2D Laser Roughness Analyzer—and proposed an innovative and nondestructive method to predict the bond strength of concrete-to-concrete interfaces. This method proved to be effective, presenting the advantages of existing methods and overcoming all the disadvantages.

Previous experimental studies conducted by the authors showed that the curing conditions have a significant influence on the bond strength of the concrete-to-concrete interface, particularly the daily fluctuations of both temperature and relative humidity. As expected, the bond strength of the concrete-to-concrete interface increased with the increase of the surface roughness. The surface preparation proved to have a significant influence on the achieved bond strength, as demonstrated by comparing the bond strength of the LAC surface with the remaining four surface conditions. An increase of the bond strength higher than 100% can only be obtained by making an adequate surface preparation.

The experimental study also proved that both differential shrinkage and differential stiffness can have a significant influence on the behavior of concrete-to-concrete interfaces—namely, on the bond strength and, as a consequence, on the failure mode. Therefore, it can be stated that the design expressions of current design codes should be improved to incorporate the consideration of these parameters, thus increasing their accuracy.

The design proposal presented by the authors is based in the current shear-friction provisions of ACI 318[1] and other codes.[2,3] A more economic design of concrete-to-concrete interfaces can be achieved because: 1) roughness is quantified for each specific surface condition; 2) cohesion is considered in design; 3) the concrete strength class is considered when computing the contribution of cohesion; 4) compression stresses on the interface due to external loading are considered; and 5) reinforcement is designed only when necessary (for higher shear stresses). Following the proposed methodology, the coefficients of cohesion and friction can be more accurately determined. Moreover, the presented proposal can be easily implemented in a future revision of ACI 318[1] and other codes.[2,3] Design expressions are given for this purpose.

AUTHOR BIOS

Pedro M. D. Santos *is an Adjunct Professor in the Civil Engineering Department of the School of Technology and Management of the Polytechnic Institute of Leiria, Leiria, Portugal. He received his MSc and PhD from the University of Coimbra, Coimbra, Portugal, in 2005 and 2009, respectively. His research interests include precasting and strengthening/repair of reinforced concrete structures.*

ACI member **Eduardo N. B. S. Júlio** *is a Full Professor in the Civil Engineering Department of Superior Técnico of the University of Lisbon, Lisbon, Portugal. He is the head of ICIST, the largest Portuguese research center in the civil engineering field. He is member of ACI Committee 364, Rehabilitation.*

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NOTATION

c = coefficient of cohesion

c_d = design value of coefficient of cohesion

f_c = concrete compressive strength

f_{cd} = design value of concrete compressive strength

f_{ctd} = design value of concrete tensile strength

f_{ctm} = mean value of concrete tensile strength

f_y = yield strength of reinforcement

k = constant (Loov's expression); coefficient of efficiency related with the reinforcement (Randl's expression); fractile of normal distribution

R_{vm} = roughness parameter *mean valley depth* of primary profile of surface

s = relative slippage

V_R = coefficient of variation

v = strength reduction factor

v_i = valley depth of cut-off length (1/5 of evaluation length)

v_u = ultimate longitudinal shear stress at concrete-to-concrete interface

$v_{u,max}$ = upper limit of the ultimate longitudinal shear stress at the concrete-to-concrete interface

w = dilatancy

α = coefficient for dowel action (Randl's expression); angle between shear reinforcement and shear plane; weighting factor (taken as 0.8 for partial safety factors)

β = coefficient allowing for angle of concrete diagonal strut (Randl's expression); reliability factor

γ_{coh} = partial safety factor for coefficient of cohesion

γ_{fr} = partial safety factor for coefficient of friction

μ = coefficient of friction, average

μ_d = design value of coefficient of friction

λ = factor related to concrete density

ρ = reinforcement ratio

σ = normal stress; standard deviation (for partial safety factors)

σ_n = normal stress acting on interface due to external loading

σ_s = normal stress acting on shear reinforcement

τ = shear stress

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