Reliable Fastening Design for Concrete Composite Structures

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Summary

In recent years the composition of differently aged concrete has gained importance. The formability of concrete allows for different types of structural members like flanges, columns and others to be augmented, giving new life and in many cases a completely different appearance to the structure. A careful design of the interface and the connecting fasteners is crucial for the functioning of the internal load transfer and the activation of new concrete layers. One of the main problems encountered is the transfer of shear stresses. The loadbearing behaviour of shear joints between concrete members of various ages was studied experimentally in the laboratories of the HILTI Corporation. From these tests a design concept has been derived which allows for a realistic prediction of the different components of shear resistance at a standardized level of safety.

Keywords: repair, retrofitting, concrete overlay, shear joints, interface, load transfer, dowel action

1. Fundamentals

Repair and retrofitting of old concrete structures becomes more and more an issue in daily engineering practice. Due to increasing traffic loads existing structures are strengthened by placing new concrete overlays. By these means the effective member depth and thus the flexural resistance of the cross section can be increased. While tensile forces can be transferred by appropriate reinforcing elements, the shear transfer needs a special attention as it depends on the interaction of connecting elements and concrete interface.

The working principle of reinforced shear joints can be explained most simply by means of the so-called 'shear friction theory' which was developed towards the end of the 1960's in the US [1]. In this respect, the joint is described by a simple saw-tooth model. According to this model, when a surface is rough, shear stressing causes not only parallel displacement, but also the joint to open which sets up tensile stresses in rebars passing between the two surfaces. These, in turn, create equalizing compressive stresses in the joint, permitting frictional forces to be set up. To date, it has been assumed that the full tensile yield strength of the rebar steel, as the reaction force, can be used at the joint, and design approaches are along these lines in standards and codes, such as Eurocode EC2 [2].

2. Experimental background

The loadbearing behaviour of shear-stressed joints between concrete of various ages was studied experimentally. Overall, 83 shear tests were carried out in five test series. A concrete baseplate, measuring 0.26 x 1.30 x 4.69 m, was cast in advance for each series and intended to represent the existing member. The surfaces of 3 of the 5 baseplates were synthetically roughened with jets of high-pressure water (HPW) or by sand blasting. During 2 series, the side of the baseplate without forming (topside when concreting) and during a further series, the formed undersides were used. The new layer of concrete (the overlay) was simulated by subsequently concreted-on cuboids, measuring 0.20 x 0.30 x 0.40 m (Fig. 1).

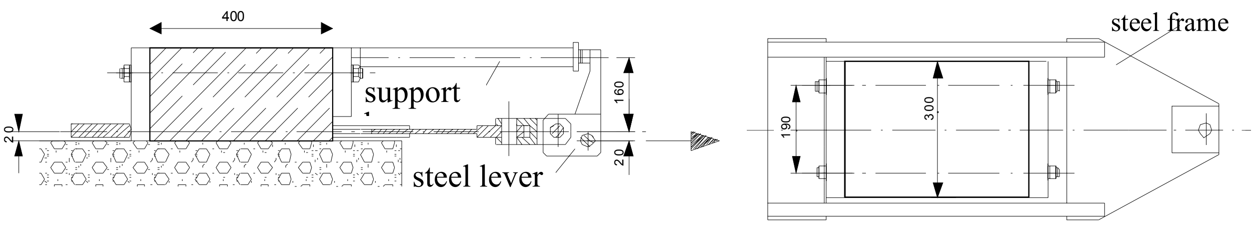

Fig. 1 Test arrangement [3]

Rebars made of ribbed steel of the BSt 500 grade were positioned at right angles to the joint surfaces. The straight rebar ends were bonded with a synthetic-resin adhesive in holes predrilled in the baseplate. Often, in field practice, the anchorage lengths in new concrete are limited by the small thickness of the overlay. To allow for this, a head (diameter 3 times bar diameter) was forged or welded onto the rebars, except for the tests with the formed surface, and, at the same time, the anchorage depth in the overlay was reduced to 5 to 6 ø. The anchorage depth in the baseplate was varied between 5 and 20 ø. As the bond strength can be greatly reduced by dirt and grime on construction sites and, in addition, cracking is possible due to restraints from shrinkage and temperature changes, three coats of form oil were brushed on the surface as bond separator.

3. Results of shear tests

3.1. Tests with high-pressure water blasted and sand-blasted surfaces

The characteristic behaviour of the rough joints can be depicted by a typical plot of force versus displacement (Fig. 2). After exceeding the maximum, $F_{max,1}$, the force first decreased, but, with increasing deformation, a gradual increase in shear resistance could be observed again with most specimens.

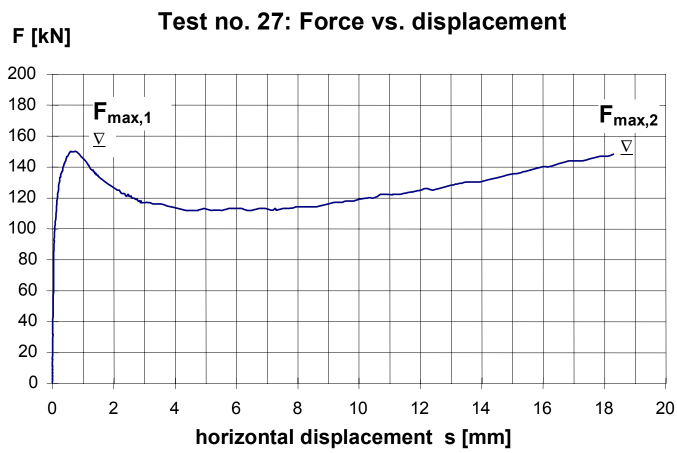

Fig. 2 Load-displacement diagram (high-pressure water blasted, 2 ø 12) [3]

Fig. 3 Fracture faces in test 25 (high-pressure water blasted, 2 ø 12) [3]

The reason for this was increasing kinking of the joint reinforcement when displacement was large: Owing to the large amount of displacement of the joint edges relative to each other and the associated inclination of the reinforcement, the tensile force in the rebar increasingly develops a component of force parallel to the joint. As a rule, the final mode of failure during the tests ($F_{max,2}$) using normal strength concrete and greater anchorage depths in the baseplate, $l_b \geq 9$ ø, was fracturing of the steel just below or above the shear plane (see Fig. 3 for an example). On the other hand, during the tests with concrete of lower strength, the resistance broke down mostly after a displacement of about 10 to 20 mm when the head was pulled out of the overlay.

When horizontal displacement takes place, the joint can clearly widen, the amount being somewhat larger with the high-pressure water blasted surfaces than with the sand-blasted ones. Special strain gauges recorded the pertaining percentage elongation (strain) of the joint reinforcement, these being inserted so far into centric holes in the rebars that they were situated at the level of the joint. It was found that the mean strains remained clearly below the yield figure of approx. 2.5‰. This behaviour was observed to the same extent with the high-pressure water blasted and the sand-blasted joints, the mean value of strain being in each case approximately 50% of the yield strain of the reinforcement. No influence from the anchorage depth or the strength of concrete could be determined, the steel thus being utilised only to 50% when the strength of concrete was higher or lower. The reason for this is simultaneous stressing of the rebar due to bending and normal force so that full reserves in the forming plastic hinge are not available for either of the two components [3].

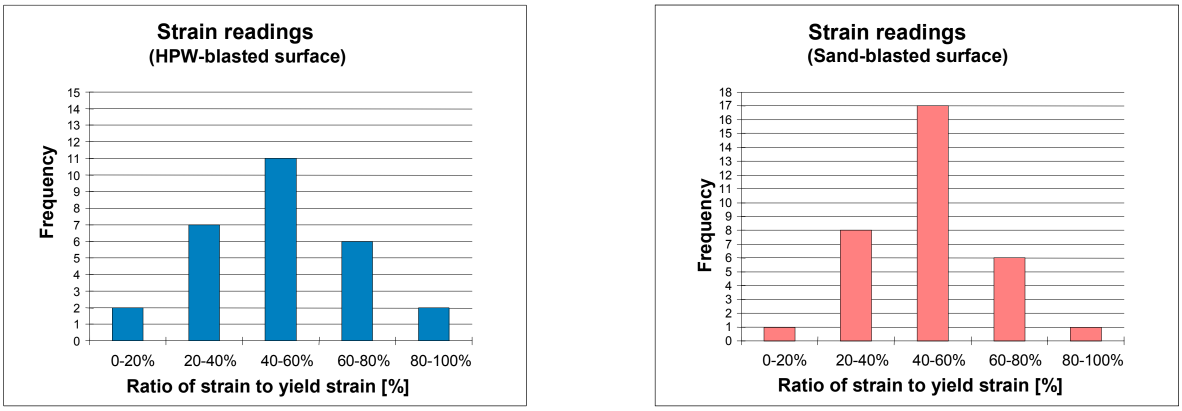

Fig.4 Strain of joint reinforcement (high-pressure water blasted and sand blasted)

When surfaces are sand-blasted or HPW-blasted and, in the latter case, the degree of reinforcement is higher than about 0.2%, load transfer takes place primarily by friction and the flexural resistance of the connecting steel. In the tests with HPW-blasted joints, a considerable load (ultimate state) was also achieved with very low degrees of reinforcement (< 0.2%) despite careful treatment with the bond separator. Apparently, a third component of resistance was decisively involved, which can be most appropriately described here as 'interlocking cohesion': Owing to the excessive and irregular roughness of the high-pressure water blasted surfaces, keying and undercutting effects took place which were then decisive for the shear resistance when the degree of reinforcement was low. If the respective shear stresses at failure are compared for the higher and lower strengths of concrete, an increase in this cohesion effect is found to the extent of approximately the cube root of the strength of the concrete.

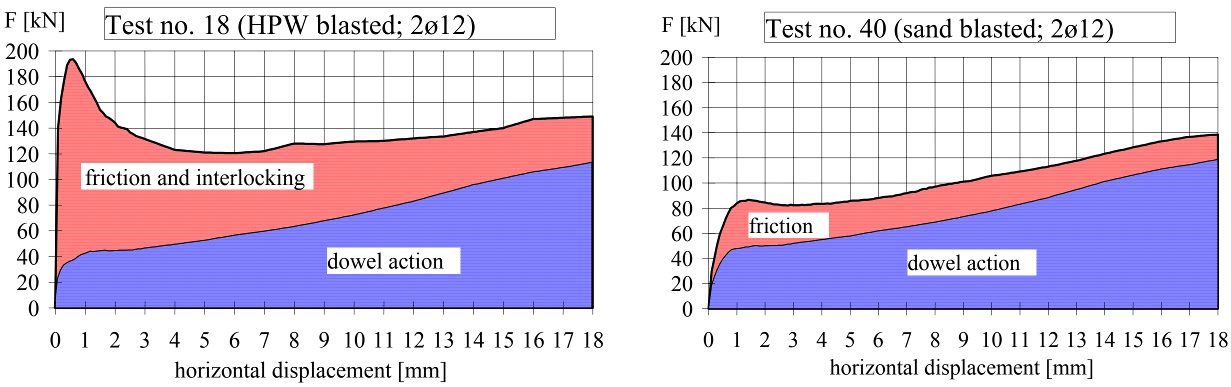

Fig. 5 Components of resistance (high-pressure water blasted and sand-blasted surface)

The various components of resistance can be determined by reverse calculation on comparing the tests carried out with smooth and with rough surfaces (Fig. 5), while allowing for the measured amounts of corresponding strains and displacements [3].

During some single tests, joints without reinforcement and bond separator were also studied. Failure took the form of a brittle fracture and the displacement on overcoming the bond was merely 0.05 mm. With the high-pressure water blasted joints, the shear stress at failure was between 2.2 and 3.7 N/mm² and with the sand-blasted joints between 2.2 and 2.8 N/mm². In these cases, however, unlike the case of a cracked joint, considerable scaling effects must be expected because the shear stresses are transferred primarily in the edge zones. It was striking that even with the much more pronounced roughness of the high-pressure water blasted surfaces a slight increase in the bond shear strength could be achieved only with the higher strengths of concrete. Greater roughness, therefore, does not always go hand in hand with a corresponding improvement of the bonding.

3.2. Tests with smooth joints

Tests with smooth joints show that the force does not reach a pronounced peak right at the beginning, as it does with the rougher joint surfaces, but the curve has a kind of 'yield plateau' instead, which is again described as $F_{max,1}$. The shear resistance here comes almost entirely from the effect of the reinforcement itself: The yield plateau can be attributed to the formation of plastic hinges in the rebar beneath or below the shear plane. The force $F_{max,1}$ was proportional to the cross-sectional area of the steel and more or less increased with the square root of the strength of the concrete. The reduction of the anchorage depth from 17 ø to 6 ø had no influence on the magnitude of $F_{max,1}$. Provided there is no external tensile force acting, an anchorage length of 6 ø is sufficient to achieve the full shear resistance.

4. Design approach

As described in the previous section, the shear resistance of a reinforced joint with broken bond, depending on the surface roughness, is made up of various components: friction, cohesion due to interlocking and the flexural resistance of the rebars passing across the joint (dowel action). The rougher the joint surfaces are, all the higher will be the friction and cohesion. On the other hand, the dowel action predominates with smooth joints.

The Coulomb shear friction hypothesis offers itself as the failure criterion to describe the resistance resulting from friction and cohesion:

$$\tau = c + \mu \cdot \sigma \tag{1}$$

The flexural resistance of the reinforcement across a joint can be approximated from the bending resistance of a cantilever beam on elastic foundation and simplified as follows [3, 5]:

$$F = k \cdot A_s \cdot \sqrt{f_{c,cube}} \cdot \sqrt{f_y} \tag{2}$$

The overall resistance of reinforced and non-reinforced shear joints is obtained by combining Coulomb friction hypothesis with the formula (2):

$$\tau_u = \tau_{coh} + \mu \cdot \sigma_n + \alpha \cdot \rho \cdot \sqrt{f_{c,cube} \cdot f_y} \tag{3}$$

The values for the free parameters, τ_{coh}, μ and α, in formula (3), were calculated via extensive regression analyses from the test results. The coefficient α makes allowance for the reduction in dowel action due to the tensile loading being superimposed.

For design the lower fractiles must be determined and partial safety factors assigned to them. The material design characteristics for friction and dowel action are derived then by applying the partial safety factors $\gamma_c = 1.5$ for concrete and $\gamma_s = 1.15$ for steel. The cohesion effect due to interlocking, τ_{coh}, represents a characteristic of concrete, on the one hand, and, on the other hand, it depends a great deal on the surface roughness structure and thus on work execution so that the scatter is relatively large. Consequently, a higher partial safety factor, $\gamma_{coh} = 1.5 \cdot 1.3 \approx 2.0$, is recommended. In keeping with observations during the shear tests, τ_{coh} is taken to be proportional to the cube root of the strength of the concrete. The final design formula for the transferable shear stress in a joint between old and new concrete results then [3, 4]:

$$\boxed{\tau_{Rd} = \frac{c \cdot f_{ck}^{1/3}}{\gamma_{coh}} + \mu \cdot \left(\rho \cdot \kappa \cdot \frac{f_{yk}}{\gamma_s} + \sigma_n \right) + \alpha \cdot \rho \cdot \sqrt{\frac{f_{yk}}{\gamma_s} \cdot \frac{f_{ck}}{\gamma_c}} \quad \leq \quad \beta \cdot \nu \cdot \frac{f_{ck}}{\gamma_c}} \tag{4}$$

Where:

f_{ck}.... characteristic cube compressive strength of the concrete [N/mm²]
f_{yk}.... characteristic yield strength (point) of joint reinforcement [N/mm²]
μ...... coefficient of friction
ρ...... degree of joint reinforcement ($\rho = A_s / A_c$; A_s: cross-sectional area of steel, A_c: shear plane)
κ...... coefficient of efficiency for tensile force that can be activated in reinforcement
σ_n.... compressive stress due to external normal force (minimal guaranteed value)
α...... coefficient for flexural resistance of reinforcement (dowel action)
β...... coefficient allowing for angle of concrete diagonal strut
ν.......reduction factor for strength of concrete diagonal strut according to [2]

Table 1: Values for constants in design formula (4)

Surface roughness (R: mean roughness from 'sand-patch' method)		c (f_{ck} in N/mm²)	μ $f_{ck} \geq 20$	μ $f_{ck} \geq 35$	κ	α	β
High-pressure water blasted	$R \geq 3$ mm						
Sand blasted	$R \geq 0.5$ mm						
Smooth							

The design value of shear stress that can be taken up, τ_{Rd} , has a top limit in order to avoid failure of the concrete diagonal strut. β results from the angle of the diagonal strut towards the joint and, furthermore, restricts the maximum percentage of reinforcement to at about 2%.

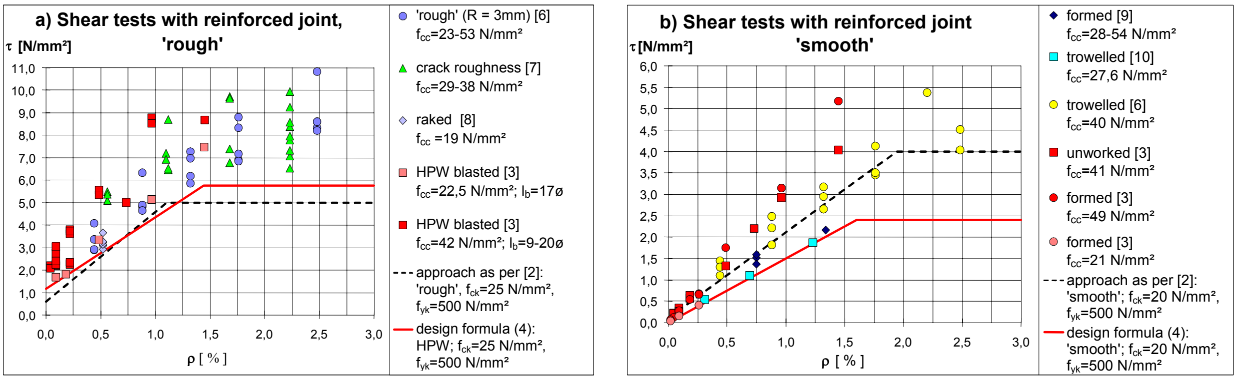

Fig. 6 Comparison of design formula (4) with test results on rough joints (a) and smooth joints (b)

Fig. 6a compares the results from the described tests with HPW-blasted surfaces and other test results from literature with the design formula (4) and the approach according to EC2 [2]. The values calculated in accordance with the design approaches are regarded without partial safety factors, i.e. on a characteristic level. It was verified that the new design approach provides a good lower limit for the test results. On the other hand, considering the test results with smooth joints, the formula according to EC2 [2] gives higher values on the unsafe side (Fig. 6b). On evaluating the formula (4), design charts for different strengths of concrete and joint roughness can be produced [11]. Generally it can be seen that the new approach delivers a better, i.e. more realistic approach to the actual behaviour than given in EC2 [2], thus ensuring a consistent level of safety.

Conclusion and outlook

Using the new design approach to shear joints, the shear resistance can be determined as a function of the surface roughness and the degree of joint reinforcement as well as any existing external normal forces. The concept has already been taken into in the national Austrian standard for reinforced-concrete design [12] and is used successfully in practice (example see Fig.7)

Fig. 7 Composite bridge strengthened by concrete overlay

Where its use in bridge construction is concerned, the loadbearing behaviour under dynamic loading with a high number of load cycles is of additional interest. As initial preliminary tests with high-pressure water blasted joint surfaces with broken bond show, a decrease in the resistance to about half of the static loadbearing capacity must be taken into account under fatigue loading [13]. Further research into this subject is being carried out at present time.

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