Properties of a bond between the steel reinforcement and the new generation concretes – a review

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Abstract. The constant development of civil engineering has led to the creation of new generation concretes. Normal concrete is modified in order to provide a plethora of new properties, such as better mechanical characteristics, increased durability and lower environmental impact. This paper presents a literature review of bond behaviour in the new generation concretes, in particular high-performance concrete (HPC), self-compacting concrete (SCC) and high-performance self-compacting concrete (HPSCC). The bond strength test procedures, as well as bond-slip models, are presented. However, these methods are not entirely accurate in the case of new generation concretes due to various modifications of the materials. The scope of the research included, among other things, the influence of material choice on the bond properties. The paper also delineates the impact of compressive strength of the bond properties in the case of new generation concretes. Last but not least, the effect of rebar location is analysed. The study shows several differences in the bond behaviour of the new generation concretes and normal concrete under investigation.

1. Introduction

One of the most crucial and continuously recognised aspects in the reinforced concrete technology is its bond to reinforcing steel. This is a phenomenon that ensures the correct functioning of the reinforced concrete structure. It consists of the mutual transfer of forces between the steel and the concrete. Analysis of bond failure allows determination of changes in deformation of both materials. Therefore, it is an inherent factor taken into account when determining the issues of cracking and deformation of the structure. Constant development of concrete technology enforced research on mechanical properties of new materials, such as new generation concretes, including bond phenomenon.

One of the earliest new generation concretes is high performance concrete (HPC) with not only high compressive strength but also increased resistance to harmful chemical and physical factors, and attractive properties of the fresh concrete mixture. The concrete with utterly different than traditional concrete rheological properties of the fresh mixture is called self-compacting concrete (SCC). The SCC is liquefied concrete, whose specific rheology enables the mould to be completely filled in a correct and properly compacted manner only under its own weight, without any signs of segregation even in the presence of very dense reinforcement. Correctly made hardened self-compacting concrete is characterized by higher integrity, durability and resistance to aggressive factors in comparison to ordinary concrete. There is also a whole group of eco-efficient concretes which are based on a concept of concrete production with fewer resources and less waste and emissions [1]. Smart materials such as
self-sensing concretes have the ability to distinguish concrete conditions and environmental characteristics, which is convenient in construction monitoring. To prolong the durability of structures self-healing concrete is introduced into the industry. The main innovation of self-healing concrete is the ability to heal cracks in it.

Authors of this paper decided to limit the scope of research to high performance concrete (HPC), self-compacting concrete (SCC) and high performance self-compacting concrete (HPSCC), which is based on the technology of the first two.

2. Bond test procedure

There are four different methods, specified in ACI 408 [2] used for an evaluation of bond properties between steel and concrete. Those procedures are called: pull-out, beam end, beam anchorage and beam splice. Moreover, two specific methods are suggested in RILEM regulations [3], namely pull-out and beam test (Figure 1). Due to the high complexity of the bond phenomenon, it is impossible to select a procedure and test element covering the whole issue. Increasingly, it seems necessary to systematize a complex testing methodology, which should include a new standardization of elements [4]. This seems to be even more justified in view of the ever-increasing number of new concretes being introduced into the reality of the building industry. It is suggested that a group of aspects should be identified, for which procedures ought to be selected together with a close correlation between specimens and actual structural elements.

The most pervasive method of bond testing is the pull-out test. It involves testing a reinforcing bar embedded in a cubic or cylindrical specimen. The bond length is relatively short and fixed as five times the rebar diameter. The concrete cover is set as greater than 4 times the rebar diameter, which reflects the dominant mechanism of bond loss due to shear failure. The pull-out test involves applying the tensile force to the reinforcing bar while the concrete block is placed on a fixed bearing. The result of the pull-out test shall be the force and the corresponding slip. The method could be used to formulate and calibrate the bond function, i.e. the relationship between different parameters and the value of the bond stress. During the test, the compressive stress is accumulated in the outer sections of the specimen while the reinforcement is under tension. The method only allows only determining the change in slip value as a function of load.

The second approach recommended by RILEM [3] to evaluate bond strength is a beam test. The method is more adequate for reinforcement exposed to bending. The test specimen consists of two semi-beams linked together by a steel hinge at the top of the beams and by a rebar at the bottom edge of the specimen. The load is applied on the top of the specimen to subject it to the bending. That method ensures a more accurate distribution of stresses in the specimen, because both reinforcement and concrete block are under tension. Similarly to the pull-out test, in the beam test, it is necessary to adopt a deep concrete cover equal to approximately 3-4 times the rebar diameter.
3. Bond-slip model

Attempts to formulate a unified bond function, confirmed by experimental research, which would describe the phenomenon in cracked elements while maintaining an uncomplicated form are still being made. The majority of the suggestions are based on the relationship between bond stress and the displacement of reinforcing bars in relation to concrete. It is assumed that this relationship occurs in any cross-section along the entire length of the bond section. Parameters used in the described dependencies are obtained in tests on models with short sections of bond with the assumption of constant stresses along their whole lengths.

Local bond stress–slip (τ–s) can be established to describe the bond resistance. When the steel bar is in an elastic state, the commonly accepted τ–s relationship from studies [6] is used. This model has been incorporated in Model Code 2010 [5]. There is a distinction made between two bond failure mechanisms – pull-out and splitting – and the location of rebar in "good" or "all other" bond conditions, as well as the restraining conditions – unconfined and stirrups. There are four different phases of the function, depending on the displacement value. A description of the function for pull-out bond failure is given here. At the initial stage according to equation (1), the occurring displacement is less than the limit value of $s_1$.

$$\tau_0 = \tau_{\text{max}} \left( \frac{s}{s_1} \right)^{\alpha}$$

(1)

Further on, until the displacement reaches the second limit value of $s_2$, the function is constant at the value interpreted as the ultimate bond stress $\tau_{\text{max}}$. The ultimate bond stress is described by the characteristic compressive strength obtained for cylindrical specimens. For the displacement between the second and third limit value – $s_3$ – the function has the form given by equation (2).

$$\tau_0 = \tau_{\text{max}} \left( \tau_{\text{max}} - \tau_f \right) \left( \frac{s - s_2}{s_3 - s_2} \right)$$

(2)

In the final stage, the function is constant and has the value of the bond stress $\tau_0$, which results from the friction forces at the concrete-concrete interface, because the displacement of the rebar reaches the equivalent length of the distance between the ribs of the reinforcement - $c_{\text{clear}}$. Achieving such a state...
of failure corresponds to extracting the rebar from the concrete block called a shear failure. All experimental parameters used in the description of the function are listed in Table 1. Moreover, a typical bond-slip relationship according to Model Code 2010 [5] is given in Figure 2.

### Table 1. Parameters defining the mean bond stress-slip relationship for deformed bars according to Model Code 2010 [5]

|                  | Pull-out |                      | Splitting |
|------------------|----------|-----------------------|-----------|
|                  | Good bond conditions | All other bond conditions | Good bond conditions | All other bond conditions |
| \( \tau_{\text{max}} \) | 2.5\( f_{\text{ck}} \) | 1.25\( f_{\text{ck}} \) | 7.0(\( f_{\text{cd}}/20 \))^{0.25} | 8.0(\( f_{\text{cd}}/20 \))^{0.25} | 5.0(\( f_{\text{cd}}/20 \))^{0.25} | 5.5(\( f_{\text{cd}}/20 \))^{0.25} |
| \( s_1 \)       | 1.0 mm   | 1.8 mm                | \( s(\tau_{\text{max}}) \) | \( s(\tau_{\text{max}}) \) | \( s(\tau_{\text{max}}) \) | \( s(\tau_{\text{max}}) \) |
| \( s_2 \)       | 2.0 mm   | 3.6 mm                | \( s_1 \) | \( s_1 \) | \( s_1 \) | \( s_1 \) |
| \( s_3 \)       | \( c_{\text{clear}} \) | \( c_{\text{clear}} \) | 1.2 | \( c_{\text{clear}} \) | 1.2 | \( c_{\text{clear}} \) |
| \( \alpha \)    | 0.4      | 0.4                   | 0.4 | 0.4 | 0.4 | 0.4 |
| \( \tau_f \)    | 0.4\( \tau_{\text{max}} \) | 0.4\( \tau_{\text{max}} \) | 0 | 0.4\( \tau_{\text{max}} \) | 0 | 0.4\( \tau_{\text{max}} \) |

On the basis of equation (1) already defined in prior versions of the Model Code and with their own experimental studies, the researchers [7] determined the function parameters that describe in a very accurate way the bond failure in high-performance concretes. The suggested value of ultimate bond stress is characterized by a linear relationship with the compressive strength of concrete as in equation (3). The parameters \( \alpha \) and \( s_1 \) are equal to the parameters specified in the Model Code 2010 [5] for good bond conditions during pull-out failure.

\[
\tau_{\text{max}} = 0.45 f_{\text{cm}}
\]

![Figure 2. A typical course of the bond-slip function according to Model Code 2010 [5]](image)

### 4. Bond factors

#### 4.1. Effect of composition

This section outlines the effect of modifications to the concrete composition on bond properties. Many studies on bond strength were based on mixtures of varying compositions, which differed in the amount and type of mineral additive, aggregate or superplasticizer.

Studies on SCC concretes with the addition of fly ash and silica fume [8] proved that regardless of the amount of mineral additive its presence has a positive effect on the bonding phenomenon. According to the research, the optimum amount of silica fume is 10% and of fly ash is 30%, in terms
of bond to reinforcing steel. This conclusion is based on the observation that the greatest bond strengths have been achieved in practice for components made with such additives.

However, the effect of the silica fume content is not unambiguously defined and confirmed in the literature. Studies [9] have shown that the cement replacement with 10% of silica fume in relation to the cement mass resulted in an average bond reduction of 7.5% in comparison to unmodified concretes. Modification of concrete with silica fume causes a change in the mechanism of bond failure associated with splitting of the concrete cover. The smooth concrete surface at the contact with steel reduces adhesion and friction between materials, resulting in a failure surface of steel-concrete contact with the same angle of inclination as the ribs' slope. This mechanism causes a reduction in ultimate bond stresses in comparison to ordinary concretes. Slightly different results concerning the influence of silica fume were obtained in the research [10], which showed that modification of high-performance concrete with silica fume can cause both increase and decrease of bond strength depending on the position of the rebar along the height of the column element. For the upper bars, an average increase in bond strengths of 10 - 16% and 5 - 20% was obtained for 480 mm and 960 mm high elements, respectively. On the other hand, for the lower rebars, an average decrease in bond stress of 5% was noted. In addition, the effect of silica fume was determined by the depth of the concrete cover, which conditions the bond failure mechanism. Tests carried out on self-compacting concretes usually lead to similar or greater bond strengths compared to conventional concretes [11–15]. The improvement in bond performance is explained by the improved homogeneity and quality of SCC. In the study [16] 11 self-compacting concrete mixtures with different water-to-binder ratios and different amounts of mineral additives - in the form of limestone powder or silica fume - and 4 mixtures of normal concrete with a fixed water-to-cement ratio of 0.51 were prepared. The addition of silica fume of 5 - 7.5% in relation to cement mass caused a slight increase of normalised bond stress as compared to unmodified mixes. The greater amount of silica fume reduced the bond. Higher content of superplasticizer in relation to the amount of water resulted in increased bond stresses in SCC mixes, while for ordinary concretes it decreased the bond stresses. It was noted that a scatter of the results of bond tests was significantly smaller in the case of SCC and analogous conclusions were drawn in the studies [17,18]. Investigations on HPSCC [19] indicated that the inclusion of silica fume enhanced the bond conditions, especially in the case of elements with a height of 1600 mm. In general, it may be claimed that with an appropriate dosage of silica fume, its properties will provide an improved bond to steel reinforcement, but this effect is additionally correlated with the depth of the concrete cover.

The partial replacement of cement with fly ash in concrete mixes has a positive effect on bond properties, as was observed in studies [20,21]. The reason for this effect is reduced porosity of mixtures with a high volume of fly ash. Similar conclusions were obtained for SCC mixtures with a significant amount of fly ash [22]. The results indicated that an increase in the content of fly ash improves chemical adhesion between steel and concrete. Mixtures with considerable volumes of fly ash are considered to be highly stable. Elements made with 70% fly ash content in relation to concrete mass exhibited the highest bond strengths compared to elements with 50% and 60% fly ash content.

Either type of coarse aggregate used in concrete mixes and its appropriate composition and quality has an influence on the bond strengths [23,24]. However, the amount of coarse aggregate does not have such a significant impact in the case of bond properties. In the tests [23] on the concrete of the same compressive strength made with basalt and lime aggregate, higher bond strengths by 13% were obtained for the rebars embedded in basalt concrete. The use of basalt aggregate improves the concrete's bending strength, which is reflected in an increase in fracture energy [25]. As a consequence, the decrease in crack propagation in such concrete slows down the mechanism of bond failure by splitting of the concrete cover. In more recent studies [24] on dolomite, gravel and limestone aggregates, the highest values of both compressive strength and ultimate bond stress were obtained for concrete made with dolomite aggregates. With a comparable compressive strength of concrete, rebars
embedded in concrete on limestone aggregate showed higher bond strengths compared to those in gravel concrete.

4.2. Effect of compressive strength
It is commonly known that the compressive strength of hardened concrete influences the steel–concrete bond. A brief analysis of this effect leads to a conclusion that increase of compressive strength generates an increase of bond strength. Initially, the ultimate bond stress was linearly related to the tensile strength of concrete or its estimator in the form of the square root of the concrete compressive strength \( f_{c}^{1/2} \) [26]. However, this does not translate to the behaviour of HPC. It was proven in the test [27] in which it was noted that for high strength concretes, normalization of the bond stresses to \( f_{c}^{1/2} \) causes their decrease with an increase of compressive strength. The explanation for this phenomenon can be derived from the altered distribution of bond stress along the length of the bar in high strength concrete. Many researchers investigated this issue and suggested taking the ultimate bond stress values at the level of the cube root of the concrete compressive strength \( f_{c}^{1/3} \) [28] or forth root \( f_{c}^{1/4} \) in the case of elements without stirrups or transverse reinforcement [23]. For confined elements, a better match is a three quarters power of compressive strength \( f_{c}^{3/4} \) [29]. The findings of research [28,30] confirmed that the growth of bond stress is slower than the growth of compressive strength.

4.3. Effect of rebar properties
The basic geometrical parameters of reinforcing bars include the height, spacing and width of ribs, the angle of inclination of the rib surface to the centreline of the rebar and the diameter of the rebar. In the international literature, the \( f_{r} \) index has been introduced, which is the ratio of the ribs' surface, projected perpendicularly to the rebar's centreline, to the nominal cross-sectional area. In the research [31] 14 specially prepared rebar profiles and one commercial type with fixed diameter and variable \( f_{r} \) index were used. The test elements were made of concrete of compressive strength equal to 40, 80 and 120 MPa. It was noted that the influence of the compressive strength of concrete on the bond stress depends on the type of ribbing. The lower the \( f_{r} \) index, the less visible was the effect of compressive strength on average bond stress. Studies on ordinary and high strength concrete [32] showed that an increase in relative rib area from 0.04 to 0.10 resulted in a 40% increase in bond strength.

In general, it can be stated that the smaller the diameter of the rebar is, the higher the bond stress values will be obtained [11, 24, 33–36]. The main reason for this dependency is considered to be the phenomenon of concrete shrinkage. The problem was raised in research carried out on beam elements made of ordinary and high-performance concrete on rebars of diameters \( \phi 10, \phi 16 \) and \( \phi 25 \), both plain and ribbed [35]. It was reported that a larger increase in bond strength when comparing high-performance concretes and normal concrete was obtained for smaller reinforcement diameters rather than for larger diameters. The higher bond stress in high-performance concretes is caused by its higher tensile strength. In SCC an increase in bond strength for smaller diameters has also been observed [11, 24]. The study [11] also noted that regardless of the reinforcement diameter, SCC exhibited higher bond strengths than normal concretes. Tests on self-compacting high strength concretes showed similar results [36], namely the highest values of ultimate bond stress were obtained for the rebars of the smallest diameter. The difference between the bond test results recorded between SCC and NC decreased as the reinforcement diameter increased. Conversely, different findings appeared in the literature [37], where an increase in the ultimate bond stress together with an increase in the reinforcement diameter was shown.

4.4. Effect of rebar location
In this section, the effect of the location of the reinforcing bar is discussed. It is understood both positions along the height and length of the test element. Its orientation in relation to the casting direction is also considered.
4.4.1. Rebar location along the height. The deterioration of the bond quality conditions along the heights of the elements is caused by a specific form of segregation related to the release of free water and is known in the literature as the top-bar effect. The phenomenon has been taken into account in the international standard regulations [38–40] and the decrease in bond strength is compensated by an appropriate rise in development length. Many studies conducted on new generation concretes [12, 14, 27, 41–44] proved that the top-bar effect is observed, although it is significantly reduced. The experiment [43] on 6 high performance concrete mixtures with varying contents of silica fume and consistencies showed that for elements with a height of less than 600 mm, the difference between the bond stress in the bottom and the top part of the elements was not significant. For specimens with a height of 480 mm, an average reduction in bond strength below 10% was noted. A more significant reduction of 17 - 35% was observed for elements with a height of 960 mm. On the contrary, in tests [12] for similar elements (1100 mm high) made of ordinary concrete, the average reduction in bond stresses was over 50%. In addition, columns made of SCC were examined there. In the case of concretes with a compressive strength of 40 MPa, the highest bond stress reduction was at a comparable level of 35% and 39% for conventional and self-compacting concrete, respectively. Stronger discrepancies were found for concretes with a compressive strength of 25 MPa, for which the observed decrease in bond stress was approximately 34% for SCC and approximately 52% for ordinary concrete. Tests [14] performed on column specimens of 1500 m height made of ordinary and self-compacting concrete with variable water-to-cement ratio showed a decrease in bond strength along the heights of the elements. The reduction of the ultimate bond stress for self-compacting concretes ranged between 32 - 55% and between 60 - 74% for ordinary concretes. In general, the top-bar effect is less significant in SCC than in normal concretes. Interestingly, the experiment [45] showed that the cement replacement with silica fume to the self-compacting concrete mix in the amount of 8.9% - 10.6% in relation to the cement mass practically eliminates the reduction of bond stress along the height of the element.

4.4.2. Rebar location along the length. The influence of the reinforcement distances from the casting point on the bond properties is not sufficiently recognized in the literature. In general, it can be observed that the bond conditions drop at a greater distance from the casting point [15,45–47], especially in the case of lower slump flows of fresh concrete mix. The phenomenon of a decrease in the height of the concrete surface was observed [47] for a distance of up to 2.20 m from the casting point. As a result, the reinforcement bars were insufficiently covered. An interesting observation was made in the study of distances up to 1.40 m from the casting point [48] where for the rebars located in the middle of the height higher ultimate bond stress was obtained as compared to the outer rebars (lower and top ones). It was concluded that this tendency is the result of good homogeneity of the mixture in the middle part of the elements. What is more, along the length of the element, an average drop in normalised bond stresses of approximately 11% was observed. Nonetheless, there are studies where no significant loss of bond strength at a distance of 1.60 m from the casting point was observed [15,45]. For smaller distances up to 0.90 m no changes in bond along the length of the specimen were noted [46]. Tests carried out on 1.60 m long high performance self-compacting concretes elements [49] indicated an average decrease in bond strength of 13.5% and 20% for normalised ultimate and average bond stress, respectively. However, no significant changes were observed in the top-bar effect in relation to the distance from the casting point.

4.4.3. Rebar orientation to the casting direction. The reinforcement can be placed perpendicularly or parallel to the direction of concreting. In the tests [12], elements made of normal and self-compacting concrete with predicted compressive strength of 25 and 40 MPa were prepared. Plain and ribbed reinforcing bars were used. The orientation of the reinforcement was more important in the case of lower strength concretes. For elements made of low strength concrete, the bond stress reduction was 25% for perpendicular bars compared to parallel ones. On the other hand, for the bond strengths of rebars embedded in high strength concrete, they did not significantly differ. Similar results were
observed in the studies [50] carried out on ordinary, HPC and HPSCC. Regardless of the compressive strength and method of compaction, rebars arranged parallel to the direction of concreting obtained higher bond stresses in comparison to those located perpendicularly. Nevertheless, the phenomenon was less perceptible for HPC and HPSCC concretes. The average reduction of bond stresses for HPC and HPSCC was 7 and 9%, respectively, and for ordinary concrete was 18%. Inferior bond properties of reinforcing bars arranged perpendicularly to the direction of concreting are caused by settlement of the mixture under the rebar, which negatively affects the quality of the reinforcement cover.

5. Conclusions

This paper reviews the phenomenon of the bond strength of the reinforcement steel in the new generation concretes, such as HPC, SCC and HPSCC. The research in this field, carried out by many authors, has led to the following conclusions:

1) Overall, a modification of the composition of the new generation concretes has a positive effect on the bond between the concrete and the reinforcing steel. In the case of replacing a part of cement with silica fume, depending on the thickness of the concrete cover and the replacement amount, the bond strengths are higher compared to those in conventional concretes. In the case of a fly ash, the higher additive content results in higher bond stresses. The positive influence of mineral additives on the bond is related to their tightening properties.

2) The bond between the steel and new generation concretes is also influenced by the type and composition of the aggregate. In general, it can be assumed that the higher the aggregate strength, the higher the bond strength, hence, concretes with basalt and dolomite aggregate perform better than those with traditional gravel.

3) The bond stress increases with the compressive strength of concrete. However, it increases disproportionately to the compressive strength increase. This is caused by the different distribution of the bond stress along the length of a rebar in the high strength concretes.

4) As in conventional concretes, the geometry of the reinforcement also influences the bond phenomenon. An increase of the relative rib area causes an improvement of the bond strength. It is assumed that the smaller the diameter of the reinforcement is, the higher the bond stress will be.

5) The top-bar effect is widely recognised in the literature and acknowledged in the international regulations in force. In general, it can be stated that in the new generation concretes, a unification of the bond strength along the heights of the elements occurs. This is linked to the tightness and better homogeneity of the structure of the new generation concretes.

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