Bond behaviour between recycled aggregate concrete and glass fibre reinforced polymer bars

Baena, M.*, Torres, L., Turon, A., Llorens, M., Barris, C.

Analysis and Advanced Materials for Structural Design (AMADE), Polytechnic School, University of Girona, Campus Montilivi s/n, 17071 Girona, Spain

Abstract

The use of recycled aggregate concrete (RAC) contributes to reducing energy and natural resource consumption in the construction industry. However, incorporating recycled concrete aggregates (RCA) into the concrete production process usually causes some difficulties in controlling fresh and hardened concrete properties. One of the properties susceptible to being affected is bond, which is a requirement for reinforced concrete (RC) structures. Besides, if more sustainable and durable structures are to be had, the benefits of fibre reinforced polymer (FRP) bars should be included to achieve this.

This study evaluates the effect on the bond behaviour between concrete and FRP bars when a percentage of natural coarse aggregates is replaced by recycled concrete coarse aggregates. To that end, a total of 48 pull-out tests were conducted. Three series of concrete mixes (i.e. three different concrete grades) were prepared, each containing four mixes, where the RCA were used at rates of 0%, 20%, 50% and 100% of the coarse aggregate total weight. The study also focuses on the influence of the rebar surface configuration (spirally wounded and ribbed) on FRP-RAC bond strength.

According to the experimental results, no unique pattern for concrete compressive strength variation after RCA has been included can be defined as being valid for all concrete grades. Furthermore, the experimental results showed that both bond development and the deterioration process between the RAC and FRP bars was similar to that between natural aggregate concrete (NAC) and FRP bars.

Keywords: Bond strength, Recycled Aggregate Concrete, FRP, Mechanical testing

*Corresponding author. Tel: +34 972419517; fax: +34 972418098; e-mail: marta.baena@udg.edu (M.Baena)

1. Introduction

The construction industry not only uses large quantities of natural resources, but also disposes of very large quantities of construction and demolition waste as well. The environmental and economic impacts of both these practices are considerable. Numerous policies aimed at increasing reuse and recycling are being promoted by many governments. The use of recycled aggregate concrete (RAC) is one way to reduce not only energy consumption, but also that of the available natural resources, thus solving some of the problems in construction engineering. However, there is a widespread reluctance to use recycled concrete as an aggregate in new concrete which results from the limited information available on the topic.

Employing demolition materials as a source of aggregates to produce new concrete may pose workability problems. The main problem with using recycled aggregates in structural concrete is their high water absorption capacity. The recycled aggregate (RA) is composed of natural aggregate (NA) bonded with cement mortar. This cementitious paste gives the recycled concrete aggregates (RCA) a rougher, lighter and more porous structure, thus decreasing their particles' density and increasing their water absorption capacity with respect to NA [1,2,3,4]. This leads to difficulties in controlling the properties of fresh concrete and consequently influences the strength and durability of hardened concrete [5]. One way to reduce the absorption capacity of RCA is to increase the amount of mixing water [6,7,8]. Alternatively, several authors pre-soaked RCA before use, keeping mixing water constant [9,10,11,12].

Several studies have looked into the impact using recycled aggregates has on the properties of hardened concrete. The differences in the sources and quality of the original concrete, their different crushing process and the RCA selection procedure, along with the variable percentage of NA being substituted by RCA used in the different studies are some of the reasons for the large variability in the experimental results obtained and which are not always consensual. Several studies report that when RCA is incorporated there is a decrease in the mechanical properties of concrete (in particular in the compressive strength, the splitting tensile strength and the Young's modulus) [13,14], while others obtained slight increases in the concrete strength when incorporating either recycled concrete coarse aggregate (RCCA) or recycled concrete fine aggregate (RCFA) [15,16]. Further studies define a replacement rate threshold value beyond which mechanical properties of hardened concrete decrease and below which mechanical properties increase [12,17,18]. In a closer analysis of the effect of substituting NA with RCA on the mechanical properties of hardened concrete, other studies differentiate between higher and lower compressive strength levels [9,19]; in these latter studies, the planes where failure takes place and the effect of the higher roughness of RCA are presented as determining factors.

One of the most important requirements in reinforced concrete (RC) constructions is the bond between the concrete and the reinforcement. Therefore, evaluating that bond behaviour between the reinforcement and the RAC is an essential requirement when employing RAC in RC structures. Investigations into the effect of RCA on bond strength with reinforcing steel are very limited [4,17,19,20,21], and the results and conclusions are not always consensual, even when confirmed and highly accepted aspects are checked. It is widely accepted, for instance, that the bond strength between natural aggregate concrete (NAC) and steel rebars is related to the square root of the concrete compressive strength ($f_c^{0.5}$). When this issue is studied in the case of RAC, different conclusions can be found in the literature. The now widely-accepted trend was confirmed in [4,17], where a decrease in the bond strength of RAC of differing percentages or RCA, that resembled the decrease in concrete compressive strength, was reported. Butler and co-workers [19] however, conclude that there is a weak relationship between the bond strength and the splitting tensile strength of concrete, and propose that bond strength is more dependent on the crushing resistance of RCA, thus highlighting the importance of knowing both the source and the characteristics of these RCA. Taking this one step further, results in [21] indicate that concrete compressive strength and the bond strength of RAC were affected by the aggregate size, with the smaller size of coarse aggregate gaining the advantage in the case of both mechanical properties. However, with the same maximum aggregate size, the compressive strength of the RAC decreased as the RCA replacement ratio increased, and the bond strength for 0% RCA replacement ratio was always higher than that of 100% RCA replacement ratio. In terms of normalized bond strength (i.e. bond strength divided by $f_c^{0.5}$), the authors observed reverse tendencies according to the maximum aggregate size: the normalized bond strength of RCA with a maximum aggregate size of 20 mm showed a tendency to increase in proportion to the RCA replacement ratio, while the normalized bond strength of RCA with a maximum aggregate size of 25 mm gradually decreased. This is in agreement with [22], where a 12.5 mm maximum aggregate size was used and the authors observed an increase in the normalized bond strength as the RCA replacement ratio was increased. However, the results in [21,22] contradict [4] where a 16 mm maximum aggregate size was used and the authors observed a decrease in the normalized bond strength as the replacement ratio of RCA was increased.

Another widely-accepted statement is that bond mechanisms acting between plain reinforcement and NAC differ from those acting between deformed rebars and NAC. As a result, the bond strength subsequently developed also varies. Xiao and Falkner [20] analysed the differences between the bond of plain and deformed steel bars when RAC was used, and confirmed that the bond strength of the deformed bars almost doubled that of the plain bars. According to the results reported, similar values of bond strength were obtained for deformed bars; irrespective of the RCA replacement percentage. However, a different trend was observed for plain bars, whose bond strength value decreased according to the rising RCA replacement percentage.

Given the inconsistencies in the experimental results, the database needs to be broadened so as to expand practical use of RAC in modern civil infrastructures. In addition, if more sustainable and durable structures are desired, the benefits of non-metallic reinforcement should be included, and therefore research into the combination of RAC and fibre reinforced polymer (FRP) rebars should be addressed. To the best of the authors' knowledge, no previous study on the bond behaviour between RAC and FRP reinforcement has ever been conducted. Therefore, within this new research field, it would seem reasonable to take the well-established knowledge on the combination of NAC and FRP as the starting point from which to evolve [23,24,25,26].

This paper investigates bond behaviour between FRP bars and concrete with different replacement ratios (0%, 20%, 50% and 100%) for the recycled coarse aggregates which were applied to three different concrete grades (low, medium and high). The study also focuses on the influence of the rebar surface configuration (spirally wounded and ribbed) on FRP-RAC bond strength. The replacement ratio of RCA was termed as the recycled aggregate replacement percentage to the total coarse aggregates by weight.

2. Experimental programme

2.1. Materials

The RAC mixes were made up of cement, water, a natural fine aggregate, a natural coarse aggregate, a recycled coarse aggregate and an additive. CEM I 52.5R cement, in accordance with the European standard EN 197-1: 2011 [27], was used in this study.

A commercial viscosity modifier and underwater admixture was used to improve workability and ensure compliance with the requirements set out by the Spanish Code on Structural Concrete (EHE-08) [28] for low water-to-cement ratio concretes.

The coarse aggregates used in this study are both natural aggregates (NA) and recycled concrete aggregates (RCA). The NA were obtained from a local quarry and one fraction size (5-15 mm) was used. The RCA were produced at a local construction and demolition waste treatment and recovery plant. The properties of the old concrete are unknown. The RCA size fraction used was also 5-15 mm. Two sizes of natural fine aggregates were used in this study: 0-2 and 0-4 mm. Table 1 summarizes the physical and mechanical properties, as well as standards used to determine the properties of the aggregates. At present, there is no standard test procedure for determining the amount of adhered mortar on recycled concrete aggregates. However, recycled aggregate was analysed in accordance with the European Standard EN 933-11:2009 [29] and Table 2 presents its constituents. The gradations and particle size distributions of the NA and RCA analysed in accordance with the European Standard EN 933-1:2012 [30] are presented in Table 3 and Fig. 1. The properties were all ascertained at the CECAM (Centre of Construction Studies and Materials Analysis) laboratories.

Property	Standard	Aggregate			
		NA 0/2	NA 0/4	NA 5/15	RCA 5/15
Maximum grain size (mm)	EN 933-1 [30]	2	4	15	15
Fineness modulus	EN 13139 [31]	3.39	4.31	5.04	4.84
Apparent density, ρ_a (kg/m^3)	EN 1097-6 [32]	2400	2620	2720	2670
After oven-drying density, ρ_{rd} (kg/m^3)	EN 1097-6 [32]	2210	2530	2680	2370
Saturated surface density, ρ_{ssd} (kg/m^3)	EN 1097-6 [32]	2290	2570	2700	2490
Water absorption (%)	EN 1097-6 [32]	3.7	1.6	0.6	4.9
Sand equivalent (%)	EN 933-8 [33]	60	80	-	-
Los Angeles test value (%)	EN 1097-2 [34]	-	-	22	28
Flakiness index (%)	EN 933-3 [35]	-	-	11	7

Table 1. Physical and mechanical properties of the aggregates.

Aggregate	*Rc* (%)	*Ru* (%)	*Rb* (%)	*Rg* (%)	*Ra* (%)	*X* (%)
RCA	53	45	0.7	0	1.2	0

Table 2. Composition of recycled concrete aggregate. *Rc* = concrete, mortar and natural aggregates with mortar attached; *Ru* = unbound natural aggregates without mortar attached; *Rb* = ceramics (brick, tiles etc); *Rg* = glass; *Ra* = asphalt; *X* = other impurities (wood, plastic, metals).

Coarse aggregate type	Sieve size (mm)										
	0.063	4	5.6	6.3	8	10	11.2	12.5	14	16	
NA 5/15	0.8	1	5	11	34	64	82	98	100	100	
RCA 5/15	0.4	2	3	15	44	73	84	96	99	100	
Fine aggregate type	**Sieve size (mm)**										
	0.063	0.125	0.25	0.5	1	2	4	5.6	6.3	8	16
NA 0/4	3.2	6	16	31	51	71	93	99	99	100	100
NA 0/2	3	11	28	48	74	97	100	100	100	100	100

Table 3. Aggregate gradations (cumulative percentage passing (%)).

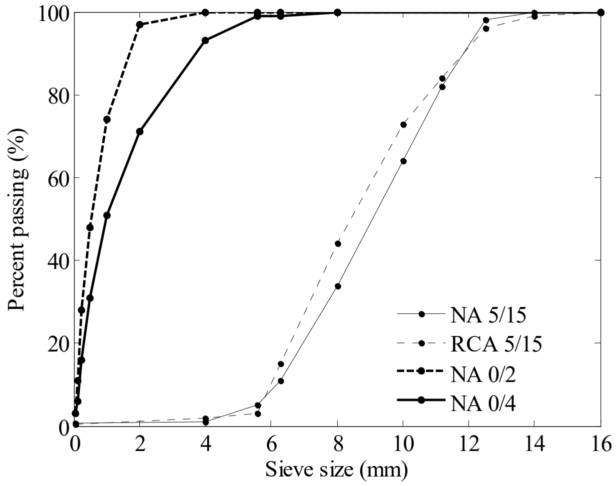

Figure 1. Particle size distribution of the aggregates.

The size distribution curves of NA 5/15 and RCA are similar, indicating that the gradation/granulometry of the final mixture would not be greatly affected by the replacement with RCA. This is also depicted in the similar values for the fineness modulus of both the coarse aggregates (presented in Table 1). Not unexpectedly, the density of the RCA is lower than that of the natural coarse aggregate (NA 5/15); this difference is explained by the greater porosity of recycled aggregate due to the presence of adhered mortar [1-4]. Recycled aggregates also have a higher absorption capacity, when compared to natural aggregates, which can be detrimental to the workability of the concrete mix [2]. Along these lines, water absorption is limited to a maximum of 7% for RCA by the Spanish Code on Structural Concrete (EHE-08) [28] and consequently the water absorption value for the recycled aggregate used in this study complies with the specifications defined in the EHE-2008 [28]. Moreover, the external morphology of the aggregates (presence of sharp edges, angular outlines, variable shapes and a more or less flat surface) is also of importance as it can lead to a reduction in the quality of concrete in terms of strength and durability. It is common practice to determine the shape of the coarse aggregates by calculating the flakiness index. Flakiness indexes for both the NA 5/15 and the RCA used in this study do not exceed the 35% threshold set by the EHE-08 [28]. The study of the physical and mechanical properties of aggregates is completed by determining the resistance to fragmentation. The EHE-08 [28] recommends using the Los Angeles test to assess the resistance of coarse aggregates to erosion caused through abrasion, wear, and impact. The Los Angeles test sets a value of 40% as the uppermost limit. Due to the low presence of ceramic (visible to the naked eye) in the coarse aggregates used in this study (see Table 2 and Fig. 2), including RCA would not exacerbate resistance to corrosion. It would appear from the physical and mechanical characterization of coarse aggregates, that both the NA and RCA used in this experimental study are suitable for concrete manufacturing.

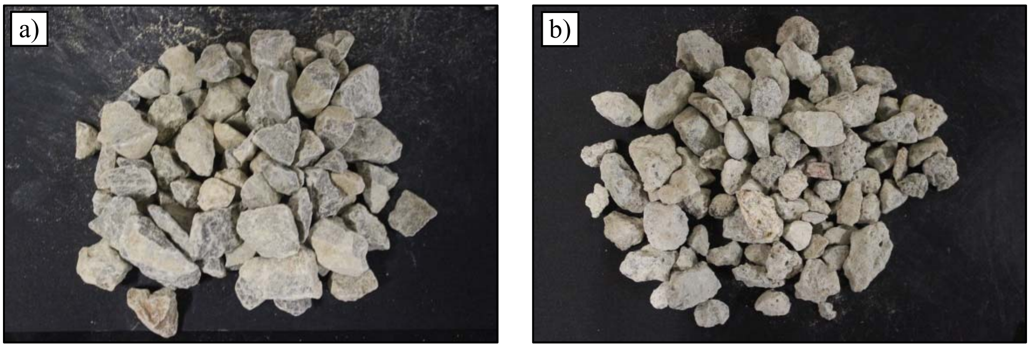

Figure 2. Photos of a) natural coarse aggregates (NA 5/15) and b) recycled concrete aggregates (RCA 5/15).

Two types of Glass Fibre Reinforced Polymer (GFRP) bars with different surface configurations were used. The Type A bar had a helical wrapping surface and some sand coating, whilst the Type B bar had a ribbed surface. In both cases, a nominal diameter of 16mm was considered. Normalized tests, according to CSA S806-12 [36], were conducted to obtain their cross-sectional area. The mean values of tensile strength, f_{fu}, and modulus of elasticity, E_f, were obtained from uniaxial tension tests according to ASTM D7205 [37] and are shown in Table 4, along with the corresponding nominal values as given by the manufacturers. The surface texture of the GFRP bars is illustrated in Fig. 3.

Type	Surface treatment	Diameter, d_b (mm)	Tensile Strength, f_{fu} (MPa)	Modulus of Elasticity, E_f (GPa)
A	HW, SC	15.875 (5/8 in)	910 (655)	49.2 (40.8)
B	GR	16	1313 (1000)	70.1 (60)

Table 4. GFRP properties. HW = helical wrapping; SC = sand coating; GR = grooves. Values provided by manufacturers in brackets.

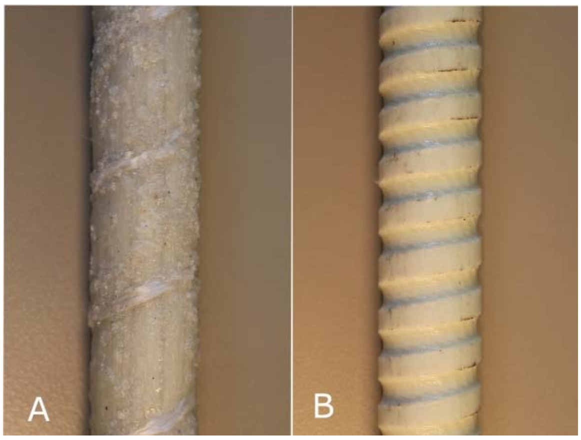

Figure 3. Surface treatment of the GFRP bars.

2.2. Concrete mix proportions

A total of twelve different mixes were employed to examine the influence of RCA incorporation on the bond behaviour between GFRP bars and RAC.

Three series of concrete mixes were prepared and each series contained four mixes, where the RCA were used at rates of 0%, 20%, 50% and 100% of the coarse aggregates total weight. The three mixes with 0% of replacement were needed to benchmark the results. These "control" concrete mixes, presented in Table 5, are commonly used by ready-mix producers in Spain and were selected to have different 28-day target strengths: (a) Mix 1, referred to as C20, has a target cylinder strength f_c=20 MPa, (b) Mix 2, referred to as C30, has a target cylinder strength f_c=30 MPa and (c) Mix 3, referred to as C50, has a target cylinder strength f_c=50 MPa.

The use of the underwater admixture additive enabled NA to be replaced with RCA, resulting in a similar workability (measured following the European Standard EN 12350-2:2009 [38]) and without increasing the water/cement ratio. Only in the case of Mix 1, with low percentage of additive, was it necessary to slightly increase the water and cement content (in different proportions, thus increasing the total w/c ratio) to preserve workability. The proportions of the concrete mixes used in this study are listed in Table 5.

Series	RCA replacement rate (%)	W/C	Unit weight (kg/m^3)							Slump (cm)
			C	W	NA 5/15	RCA 5/15	NA 0/2	NA 0/4	AD	
Mix 1, C20	0	0.77	160.4	123	980	0	180	800	2.23	7.0
	20	0.86	170	145.67	784	196	180	800	2.2	7.0
	50	0.79	205.42	162.43	490	490	180	800	2.23	7.5
	100	0.82	201.24	165.43	0	980	180	800	2.63	7.5
Mix 2, C30	0	0.60	275	165	960	0	180	725	3.3	8.0
	20	0.60	275	165	768	192	180	725	3.3	7.5
	50	0.60	275	165	480	480	180	725	3.3	8.0
	100	0.60	275	167	0	960	180	725	3.3	8.0
Mix 3, C50	0	0.42	450	190.66	1050	0	60	580	6.75	10.0
	20	0.42	450	190.66	840	210	60	580	6.75	10.5
	50	0.42	450	190.66	525	525	60	580	6.75	10.5
	100	0.43	450	192.33	0	1050	60	580	6.75	11.0

Table 5. Concrete mix proportions. W/C = water-to-cement ratio; C = cement; W = water; AD = admixture.

2.3. Curing and material test method for concrete

For each mix, three cylindrical specimens 150 mm diameter and 300 mm height and three 150 mm cubic specimens were cast in steel moulds and transferred to a curing room until de-moulding 24 hours later. The material characterization specimens were then de-moulded, marked and once again transferred to the curing room which was set at 20±2ºC with approximately 95% humidity. The cylinders were used to determine the tensile splitting strength (f_{ct}), and the cubic specimens were used to determine the compressive strength ($f_{c,cub}$) at 28 days.

2.4. Details of pull-out specimens

The pull-out specimens were fabricated according to the ACI 440.3R-04 [39] standards and a 200 mm cubic mould was used to manufacture them. The bond length, l_b, was five times the rebar diameter, this short anchored length allowing the assumption of a uniform distribution of bond stress. The bond length was marked with PVC pipes and placed at the bottom of the concrete cube (see Fig. 4a). The concrete was then poured with the GFRP bars in a vertical position inside the mould, in the middle of the specimen. After moulding, the specimens were transferred to a curing room for 24 hours, along with the concrete characterization specimens. Thereafter, the pull-out specimens were de-moulded, marked and transferred once again to the curing room at 20±2 ºC and about 95% humidity, until testing at 28 days.

Two pull-out specimens per each concrete series, recycled aggregate percentage (0%, 20%, 50% and 100%), and GFRP bar types (A and B) were manufactured, thus giving a total of 48 pull-out specimens. The elements tested were identified as Cx-R-G-N, with Cx standing for the type of concrete (C20, C30 C50), R for the RCA rate (0%, 20%, 50%, 100%), G for the type of GFRP bar (A or B), and N for the identification of identical specimens.

2.5. Test set-up

The pull-out test set-up is shown in Fig. 4b. The tests were performed in accordance with ACI 440.3R-04 [39], using a servo-hydraulic testing machine with a capacity of 600 kN. Displacement control was selected to capture post-peak behaviour. The load was applied to the reinforcement bar at a rate of 0.02 mm/s and measured with the electronic load cell of the testing machine. The loaded and unloaded end slips were measured with four linear variable differential transformers (LVDTs). An automatic data acquisition system was used to record the data.

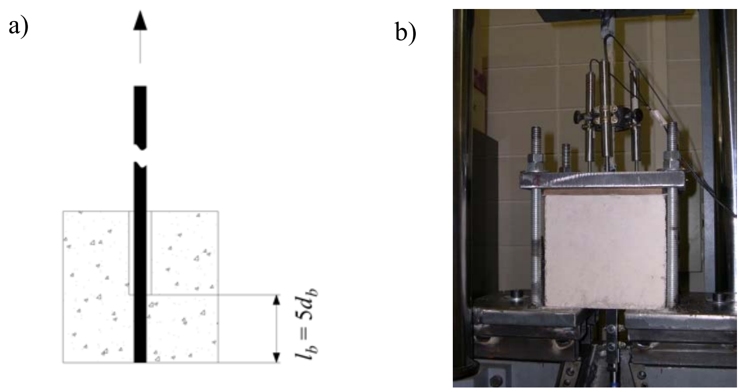

Figure 4. Pull-out test a) specimens, b) set-up.

3. Test results

3.1. Compressive strength

The compressive strengths of the concrete mixes at 28 days are shown in Fig. 5 and Table 6. Different trends in the evolution of compressive strength can be found when natural aggregate is replaced with RCA, and this is in contrast with the trends in the literature that reported a decrease in compressive strength when natural aggregate is replaced with RCA [21,40]. To explain the effect that RCA has on concrete compressive strength, the water-to-cement ratio of the concrete mixtures and the failure planes of hardened concrete must be considered.

According to the proportions of the concrete mixes presented in Table 5, the water-to-cement ratio was not kept constant in the Mix 1 (C20) series for the sake of workability. Concrete mixtures with larger water-to-cement ratio are largely referred to in the literature review as resulting in lower mortar strengths, which then manifests itself as a reduction in concrete compressive strength when failure occurs through the mortar phase. Visual inspection of the failure plane for the Mix 1 specimens in this experimental study confirmed that failure occurred mainly through the mortar phase, thus explaining the loss of compressive strength. Special attention should be given to mix C20-100 when compared to the other mixtures of the same Mix 1 series. In this case, the decrease in compressive strength (due to a larger water-to-cement ratio) is mostly offset by the effect of the higher roughness of the RCA (compared with the smoother surface texture of the NA). As suggested by [9,19,41] it appears that the aggregate-mortar bond strength between the new mortar and the RCA is greater than the aggregate-mortar bond strength between new mortar and the natural aggregate, due to both the presence of non-hydrated cement particles in the RAC and their higher roughness.

No influence of water-to-cement ratio (and therefore no influence of mortar phase strength) can be expected in the analysis of the effect of RCA on concrete compressive strength in the Mix 2 (C30) and Mix 3 (C50) series, as the water-to-cement ratio was kept constant, irrespective of the percentage of RCA replacement. Therefore, to explain the effect that RCA has on concrete compressive strength, attention should be focussed on the failure planes of hardened concrete.

Failure planes can be classified as being mainly around or mainly through the coarse aggregate [19]. Failure planes occurring around the aggregate indicate that the mortar-aggregate interface is the limiting strength factor. It should be noted that concretes made up of recycled aggregates have two interfacial zones (IZ): one formed in the recycled aggregate (bond between gravel and old mortar) and the other newly created between the recycled aggregate (including old mortar) and the new cement paste. Therefore, for the recycled aggregate concrete (RAC) with failure occurring around the coarse aggregate, either the old or the new IZ is the limiting strength factor. Failure planes occurring through the coarse aggregate, on the other hand, indicate that it is the strength of the coarse aggregate itself that is the limiting strength factor. Again, in the recycled aggregate concrete, the limiting strength factor can be either the old or the new coarse aggregate strength.

Visual inspection of the failure plane for Mix 2 (C30) specimens confirmed that for RCA replacement ratios equal or less than 50%, failure was occurring mostly around the aggregates, whilst for 100% RCA replacement, failure occurred mainly through the aggregates. According to the concrete compressive strengths presented in Table 6, it can be concluded that for RCA replacement up to 50%, the IZ seems to be the limiting strength factor, and therefore concrete compressive strength increases due to the higher roughness of the RCA. In contrast, for 100% RCA replacement (C30-100), the original natural aggregate strength (in the RCA) was the limiting strength factor, rather than the mortar-aggregate bond.

Visual inspection was also applied to characterise Mix 3 (C50). In this case, the predominant failure plane observed was through the aggregates, thus confirming the quality of the new mortar phase. The initial benefit provided by 20% RCA replacement, probably attributable to their higher roughness, is followed by a loss in concrete compressive strength when the replacement rate is increased, this signalling that the original natural aggregate strength (in the RCA) is the limiting strength factor.

Mix design	Compressive strength, $f_{c,cub}$ (MPa)	Tensile splitting strength, f_{ct} (MPa)
C20-0	27.46	1.95
C20-20	22.02	1.93
C20-50	25.45	1.97
C20-100	26.79	2.37
C30-0	34.10	2.33
C30-20	35.76	2.48
C30-50	38.82	2.98
C30-100	38.37	2.31
C50-0	54.28	3.50
C50-20	60.65	3.40
C50-50	58.42	3.23
C50-100	50.59	3.55

Table 6. Properties of hardened concrete.

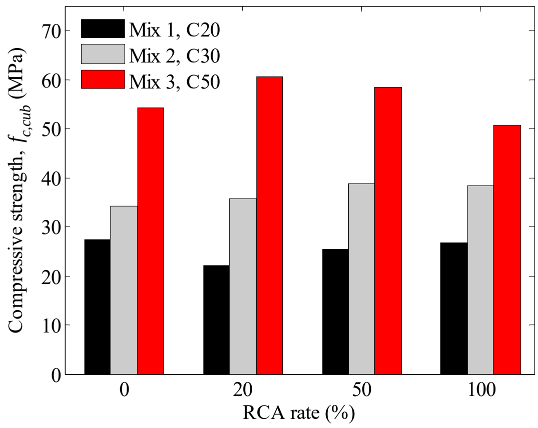

Figure 5. Effect of recycled aggregate content on concrete compressive strength (for different concrete grades).

3.2. Bond stress versus slip curves

The relationship between the bond stress and slip between the rebar and the concrete is used to analyse bond behaviour. The bond stress can be determined from the tensile pull-out load applied during the test. Although stress distribution is not constant along a bond length, the short bond length assumed in this experimental programme (i.e. bond length equal to five times the rebar diameter) allows the usual assumption in pull-out test results that bond stress is uniformly distributed. Therefore, an average bond stress can be defined as:

$$\tau_{av} = \frac{P}{\pi d_b l_b} \tag{1}$$

where P is the tensile load applied to the reinforcing bar, d_b is the rebar diameter and l_b is the bond length. At each load level, the slip at the loaded end is computed as the average of the slip values measured by the top 3 LVDTs minus the elongation of the FRP bar in the length between the top surface of bonded length and the point of attachment of the measuring device on the FRP bars [39]. The unloaded end slip is obtained from the bottom LVDT.

Representative specimen curves are shown in Fig. 6 to illustrate the bond stress-slip relationship obtained for the different specimens. Experimental results for specimens with the type A bar are shown in Figs. 6a, 6c and 6e, with every subfigure being representative of concrete Mixes 1 (C20), 2 (C30) and 3 (C50) specimens, respectively. Likewise with Figs. 6b, 6d and 6f, where these subfigures represent the experimental results for the B type bar specimens. In every subfigure (Fig. 6a-6f) a representative specimen of each RCA replacement rate (0%, 20%, 50% and 100%) is presented.

The global behaviour of the bond stress-slip relationships is characterised by an initial increase in bond stress with little slip, which is usually referred to as the micro-slip stage. This stage is followed by the internal cracking stage, when the load increases towards a critical value and the free end of the rebar begins to slip, thus demonstrating that the adhesion force at the anchorage has been all but exhausted. After this point, the rate of slip begins to increase until the maximum bond stress is attained. At the descending stage, the stress declines rapidly and the slip increases, with bond being attributed to the bearing and friction between the rebar and concrete.

The analysis of the above-mentioned curves indicates that no great differences are to be found in the bond stress-slip responses shown in every subfigure. This means that the inclusion of recycled concrete aggregate does not modify the bond mechanisms and therefore the bond development and deterioration process between the recycled aggregate concrete and the FRP bars is similar to that between natural aggregate concrete and FRP bars.

Moreover, small differences can be found between bond stress-slip curves for the A and B type bars. Although surface treatments of these two bars are different (helical wrapping surface vs. ribbed surface), both bars would be classified as deformed bars (as neither of them belong to the plain bar group), and therefore no great differences in the post-peak bond mechanism could be expected. In this sense, for both types of bars, the crushed concrete sticking to the front of the lugs exerts a wedging action; as a consequence the surrounding concrete exerts a confinement action on the rebar, and therefore bearing resistance is the bond mechanism being activated. It can be seen that for similar conditions, peak values are higher for the B type bars and the slope beyond the peak is lower for the A type bars. This is a consequence of the failure mode being more alike to wear and friction for the A type bar and to the shearing of the lugs in the case of the B type bar [23].

Finally, experimental results confirm that concrete strength has a significant influence on bearing resistance, which increases with increasing concrete compressive strength. Therefore, the larger the concrete grade, the larger the bearing resistance is and the less the abruptness in the decay of post-peak bond stresses is.

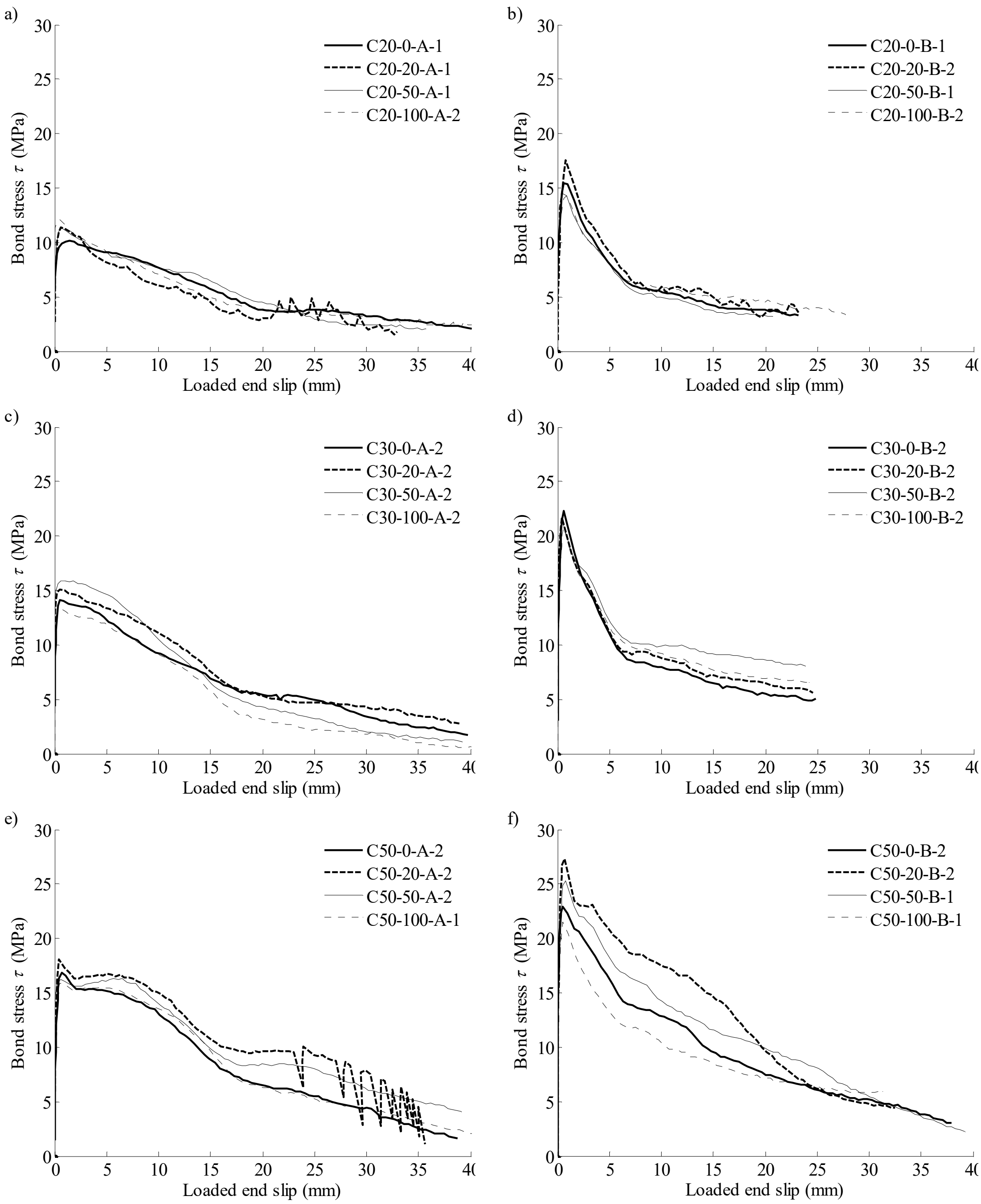

Figure 6. Representative test curves of bond stress versus slip: a) C20-A; b) C20-B; c) C30-A; d) C30-B; e) C50-A; f) C50-B.

3.3. Bond strength

A summary of the experimental results obtained from pull-out tests are given in Table 7, for the A type bar, and in Table 8, for the B type bar. In these tables, f_c is the concrete compressive strength, P_{max} is the peak load (i.e. maximum load attained in the bond test), τ_{av} is the bond strength and τ_{av}^* is a normalised bond strength that accounts for the effect of the concrete strength, defined as:

$$\tau_{av}^* = \frac{\tau_{av}}{\sqrt{f_c}} \quad (2)$$

The mean values of bond strength, τ_{av}, and normalised bond strength, τ_{av}^*, of nominally identical specimens are also reported. It should be mentioned that no results are able to be presented for specimen C50-20-B-1 due to technical problems with the data acquisition system during the test. All specimens failed in pull-out mode and no cracks were observed at the end of the tests. Visual inspection of the surface of the reinforcing bars after the pull-out test confirms differences in how this surface was damaged according to the grade of concrete. Low surface damage was observed in bars of series C20, with most of the cases having some sticked concrete. Moving to series C30, a more worn surface was observed for bars type A (with some concrete sticking to the bar surface, and some bar fibres sticking to concrete); in the case of bar type B, damage of the bar surface consisted mainly in wear, with loss of bar ribs arising on occasions. Finally, damage was largely concentrated on bar surface (and not in concrete surface) for specimens in series C50; in this sense, it is worth mentioning that for bar type B damage mostly occurred by shearing of the bar surface rib.

Specimen	$f_{c,cub}$ (MPa)	Maximum load, P_{max} (kN)	Bond strength, τ_{av} (MPa)	Mean bond strength (MPa)	Normalised bond strength, τ_{av}^* (MPa$^{0.5}$)	Mean normalised bond strength (MPa$^{0.5}$)
C20-0-A-1	27.46	41.28	10.26	11.57	1.96	2.21
C20-0-A-2	27.46	51.75	12.87		2.46	
C20-20-A-1	22.02	46.66	11.60	10.26	2.47	2.19
C20-20-A-2	22.02	35.89	8.93		1.90	
C20-50-A-1	25.45	46.36	11.53	11.16	2.29	2.21
C20-50-A-2	25.45	43.37	10.79		2.14	
C20-100-A-1	26.79	39.10	9.72	11.00	1.88	2.12
C20-100-A-2	26.79	49.33	12.27		2.37	
C30-0-A-1	34.10	53.84	13.39	13.87	2.29	2.38
C30-0-A-2	34.10	57.73	14.36		2.46	
C30-20-A-1	35.76	55.93	13.91	14.58	2.33	2.44
C30-20-A-2	35.76	61.35	15.26		2.55	
C30-50-A-1	38.82	63.76	15.85	16.00	2.54	2.57
C30-50-A-2	38.82	64.91	16.14		2.59	
C30-100-A-1	38.37	61.49	15.29	14.37	2.47	2.32
C30-100-A-2	38.37	54.10	13.45		2.17	
C50-0-A-1	54.28	62.22	15.47	16.22	2.10	2.20
C50-0-A-2	54.28	68.22	16.97		2.30	
C50-20-A-1	60.65	68.20	16.96	17.57	2.18	2.26
C50-20-A-2	60.65	73.15	18.19		2.34	
C50-50-A-1	58.42	80.31	19.97	18.21	2.61	2.38
C50-50-A-2	58.42	66.14	16.45		2.15	
C50-100-A-1	50.59	64.91	16.14	14.54	2.27	2.04
C50-100-A-2	50.59	52.05	12.94		1.82	

Table 7. Summary of bond strength for specimens reinforced with spirally wounded bar (type A).

A comparison between the experimental results for the A type bar and the B type bar depict that, although bond after peak load relied on bearing resistance and friction for both types of bar, larger bond strengths were in fact obtained for ribbed bars, irrespective of the concrete grade or the RCA percentage rate (see Fig. 7). This is in accordance with Xiao and Falkner [20], who reported the same trend for bond behaviour between steel reinforcement and RAC.

Moreover, the state of the art largely accepts that bond strength between natural aggregate concrete and reinforcement is related to concrete compressive strength, with this interrelation being more pronounced for deformed and/or ribbed bars. In the case of recycled aggregate concrete, this is also confirmed by the experimental results of this experimental programme (see Fig. 8), which show that larger bond strengths are obtained for increasing concrete grades, irrespective of the RCA replacement rate or the bar type.

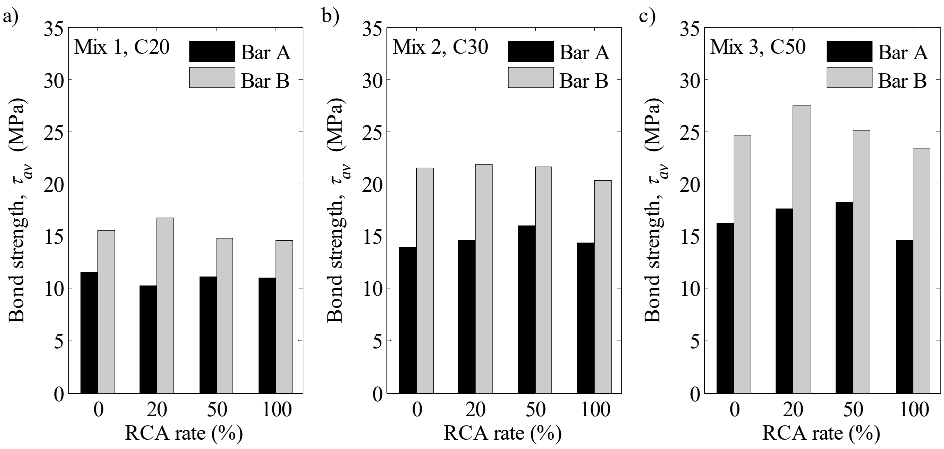

Figure 7. Comparison between mean bond strength for different bar types for concrete: a) Mix 1, C20; b) Mix 2, C30; c) Mix 3, C50.

Specimen	$f_{c,cub}$ (MPa)	Maximum load, P_{max} (kN)	Bond strength, τ_{av} (MPa)	Mean bond strength (MPa)	Normalised bond strength, τ_{av}^* (MPa$^{0.5}$)	Mean normalised bond strength (MPa$^{0.5}$)
C20-0-B-1	27.46	63.11	15.69	15.55	3.00	2.97
C20-0-B-2	27.46	61.92	15.40		2.94	
C20-20-B-1	22.02	62.81	15.62	16.74	3.33	3.57
C20-20-B-2	22.02	71.79	17.85		3.80	
C20-50-B-1	25.45	59.52	14.80	14.84	2.93	2.94
C20-50-B-2	25.45	59.82	14.88		2.95	
C20-100-B-1	26.79	59.50	14.80	14.61	2.86	2.82
C20-100-B-2	26.79	57.97	14.41		2.78	
C30-0-B-1	34.10	83.51	20.77	21.56	3.56	3.69
C30-0-B-2	34.10	89.86	22.35		3.83	
C30-20-B-1	35.76	87.80	21.83	21.83	3.65	3.65
C30-20-B-2	35.76	87.73	21.82		3.65	
C30-50-B-1	38.82	87.04	21.65	21.61	3.47	3.47
C30-50-B-2	38.82	86.74	21.57		3.46	
C30-100-B-1	38.37	78.28	19.47	20.35	3.14	3.28
C30-100-B-2	38.37	85.36	21.23		3.43	
C50-0-B-1	54.28	105.89	26.33	24.70	3.57	3.35
C50-0-B-2	54.28	92.72	23.06		3.13	
C50-20-B-1	60.65	-	-	27.49	-	3.53
C50-20-B-2	60.65	110.53	27.49		3.53	
C50-50-B-1	58.42	102.33	25.45	25.11	3.33	3.28
C50-50-B-2	58.42	99.60	24.77		3.24	
C50-100-B-1	50.59	87.23	21.69	23.30	3.05	3.28
C50-100-B-2	50.59	100.20	24.92		3.50	

Table 8. Summary of bond strength for specimens reinforced with ribbed bar (type B).

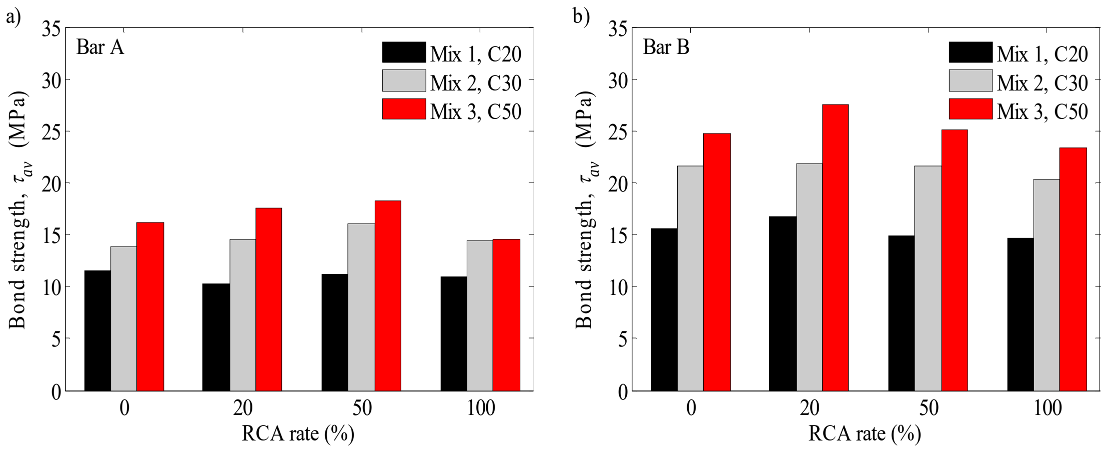

Figure 8. Comparison between mean bond strength for different concrete grades for bar: a) type A, and b) type B.

3.4. Influence of RCA on bond strength

In this section, the experimental results on bond strength from the three different target concrete compressive strengths (C20, C30 and C50) are analysed to determine whether the impact of the RCA replacement rate (0%, 20%, 50% and 100%) on bond strength varies according to the grade of concrete.

The influence of RCA on the bond strength of steel RC has mainly been studied using a fixed target value of concrete compressive strength [17, 19, 21, 22] and few studies have incorporated different grades of concrete strengths [4].

A literature review on bond between NAC and FRP reinforcement reveals that the bond strength between FRP bars and concrete with a compressive strength approximately greater than 30 MPa does not greatly depend on the value of the concrete strength, but rather on the rebar's properties and surface configuration. However, different dependencies are observed for concrete grades around 15 MPa, when bond is affected much more by the concrete grade [23,24]. Experimental results in this study confirm that this same trend also applies to bond between RAC and FRP reinforcement.

In the present experimental programme, specimens casted with the Mix 1 concrete mixture (with a target concrete compressive strength of 20 MPa) are said to represent low concrete grade specimens. Therefore, concrete compressive strength is expected to be a determinant in the analysis of the influence of RCA on their bond strength. As reported in Section 3.1, the decrease in the concrete compressive strength in the Mix 1 series is not directly related to a higher RCA replacement rate, but rather to a higher water-to-cement ratio (for replacement rates equal to 0%, 20% and 50%) and a combination of the water-to-cement ratio and higher roughness of RCA (for a replacement rate of 100%). Bearing this in mind, whatever the origin/cause of the variation in the compressive strength of the concrete, variations in bond strength for both bar types (A and B) resemble those of the compressive strength (see Table 9).

Specimens cast from the Mix 2 concrete mixture (with a concrete compressive strength target of 30 MPa) are said to represent medium concrete grade specimens. In this case, the failure and bond strength are said to be not only dependent on concrete properties, but also on a combination of hardened concrete properties and the surface configuration of the reinforcement. Experimental data is thus analysed separately for A and B type bars. The bond result analysis for the A type bar reveals that for the RCA replacement ratios of 20% and 50%, variations in the bond strength in percentage terms resembles those in the compressive strength of concrete. The effect of replacing 100% of the NA with RCA shows a different tendency; in this case, although the compressive strength of the concrete is increased (with respect to the replacement ratio of 0%), a smaller percentage of increase in the bond strength is found, probably due to the change in the concrete failure plane (analysed in Section 3.1). This is a sign of bond being a consequence of the combination of this higher compressive strength and the bar surface configuration. Results on specimens that combine this same concrete mixture (Mix 2, C30) and the type B bar, show that the RCA replacement ratio, and as a consequence the compressive strength of the concrete, may not be a highly determining factor. The increase in the compressive strength of the concrete produced by the inclusion of RCA does not imply any great changes in bond strength (see Table 9). In this case, the ribbed surface configuration of the bar means the bond is more dependent on the shear strength of both the bar and the concrete ribs. This is in accordance with [20], whose experimental programme covered RCA replacement ratios of 0%, 50% and 100% with concrete grades similar to those in Mix 2 in this study.

Experimental results for the highest concrete grade specimens (specimens with the Mix 3 concrete mixture and with a target concrete compressive strength of 50 MPa) indicate that bond strength increases for RCA replacement ratios equal to 20% and 50% and decreases for a total replacement (r=100%), irrespective of the type of reinforcement bar. This is a sign of the bond being greatly influenced by the concrete compressive strength, with the bar's surface configuration producing a low impact. It should be mentioned, however, that the percentage variations in compressive strength and the bond strength are different.

RCA replacement rate, r (%)	Mix 1, C20			Mix 2, C30			Mix 3, C50		
	$f_{c,cub}$ (MPa)	$\tau_{av}/\tau_{av,\,r=0}$		$f_{c,cub}$ (MPa)	$\tau_{av}/\tau_{av,\,r=0}$		$f_{c,cub}$ (MPa)	$\tau_{av}/\tau_{av,\,r=0}$	
		bar A	bar B		bar A	bar B		bar A	bar B
0	27.46	1.00	1.00	34.10	1.00	1.00	54.28	1.00	1.00
20	22.02	0.89	1.08	35.76	1.05	1.01	60.65	1.08	1.11
50	25.45	0.96	0.95	38.82	1.15	1.00	58.42	1.12	1.02
100	26.79	0.97	0.95	38.37	1.04	0.94	50.59	0.90	0.94

Table 9. Effect of RCA replacement rate on bond strength.

3.5. Relative normalized bond strength

The analysis of the experimental results presented until now, clearly suggest that when talking about a relationship between the variations in bond strength due to the inclusion of RCA, one cannot forget the variations in bond strength due to variations in the concrete's compressive strength, these being an inherent effect of including RCA.

For the effect of the inclusion of RCA on bond strength to be analysed in isolation and without taking into account the effect of changes in the compressive strength of the concrete, the normalised bond strength of the pull-out tests in this experimental programme (presented in Tables 7 and 8) is used to calculate the percentage change from the baseline value produced by the replacement of RCA (i.e. $\tau_{av}^{*}/\tau_{av}^{*}{}_{,\,r=0}$), the results of which are presented in Table 10.

A general result of this experimental programme which is applicable to any concrete grade and/or reinforcement surface configuration, is that it is the total replacement of RCA (r=100%) that produces a decrease in normalized bond strength. This general result aside, the experimental results are analysed separately for the different concrete grades.

Concrete compressive strength is usually reported as being a determinant on bond strength in low concrete grade specimens (Mix 1). When the effect of concrete compressive strength is removed, the experimental results show that in the case of the same RCA replacement ratio similar variations in the normalized bond strength between the two types of FRP bars and concrete are found, but with the exception of the 20% RCA replacement rate combined with the type B bar. These results confirm the low impact of bar surface's configuration.

In contrast, bond in the medium concrete grade (Mix 2) is reported to be dependent on the combination of hardened concrete's properties and the surface configuration of the reinforcement. Once the effect of the concrete's properties is removed, experimental results show that in deformed bars rather than in ribbed bars, replacing RCA causes either higher increases or smaller reductions in the normalized bond strength.

RCA replacement rate, r (%)	Mix 1, C20			Mix 2, C30			Mix 3, C50		
	$f_{c,cub}$ (MPa)	$\tau_{av}^{*}/\tau_{av}^{*}{}_{,r=0}$		$f_{c,cub}$ (MPa)	$\tau_{av}^{*}/\tau_{av}^{*}{}_{,r=0}$		$f_{c,cub}$ (MPa)	$\tau_{av}^{*}/\tau_{av}^{*}{}_{,r=0}$	
		bar A	bar B		bar A	bar B		bar A	bar B
0	27,46	1,00	1,00	34,10	1,00	1,00	54,28	1,00	1,00
20	22,02	0,99	1,20	35,76	1,03	0,99	60,65	1,03	1,05
50	25,45	1,00	0,99	38,82	1,08	0,94	58,42	1,08	0,98
100	26,79	0,96	0,95	38,37	0,98	0,89	50,59	0,93	0,98

Table 10. Effect of RCA replacement rate on normalized bond strength.

The literature on bond between NAC and steel or FRP reinforcement, remarks that bond strength is a function of the square root of the concrete strength [42-44]. Therefore, a number of researchers have proposed equations that represent the bond between the reinforcing bars and the concrete in which the compressive strength of concrete is a factor involved. Some of these expressions are presented next and checked to confirm their applicability to bond between RAC and FRP.

CEB-FIP Model-Code [42] distinguishes between two different situations when defining the bond strength between ribbed bars and concrete: unconfined concrete (failure through concrete splitting) and confined concrete (failure through pull-out). As all the specimens in this experimental programme failed in pull-out mode, only the equations for confined concrete are considered in this study:

Good bond conditions: $$\tau_{max} = 2.5\sqrt{f_c} \quad (3)$$

All other bond conditions: $$\tau_{max} = 1.25\sqrt{f_c} \quad (4)$$

Orangun and co-workers [43] proposed the following formula:

$$\tau = 0.083045\sqrt{f_c}\left[1.2 + 3\frac{C}{d_b} + 50\frac{d_b}{l_b}\right] \quad (5)$$

where C is the minimum value between C_s and C_b, where C_s=min(1/2 of clear spacing, side cover) and C_b=cover. Based on the previous expression and an expanded experimental data base, Darwin and co-workers [44] propose an alternative equation as follows:

$$\tau = 0.083045\sqrt{f_c}\left[\left(1.06 + 2.12\frac{C}{d_b}\right)\left(0{,}92 + 0{,}08\frac{C_{max}}{C_{min}}\right) + 75\frac{d_b}{l_b}\right] \quad (6)$$

where C_{min}=min(C_x, C_y, C_s/2), C_{max}=max[min(C_x, C_s/2), C_y] in which C_x is the side cover, C_y is the bottom cover and C_s is the spacing between the bars.

The results from this experimental programme are compared with predictions from the above-mentioned expressions in Fig. 9. It should be noted that the predictions from Eqs. 5 and 6 have almost overlapped. This is due to the bond test set-up used in this study, namely a pull-out test with a single bar centred in a cubic concrete block.

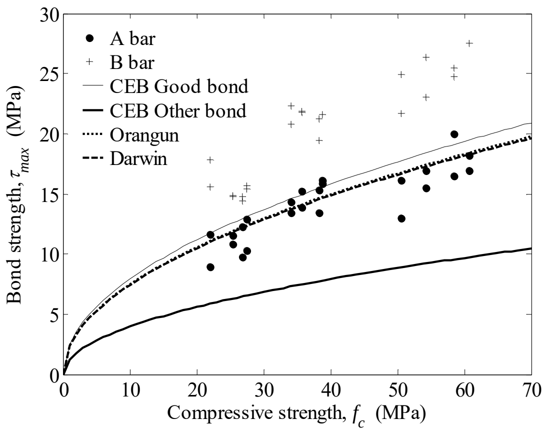

Figure 9. Variation of the bond strength with the concrete compressive strength.

Although Eqs. 3 and 4 were originally defined for ribbed bars, the bond strengths in the specimens cast with ribbed bars (type B) are underpredicted by the CEB-FIP proposal, probably due to the conservative nature of the code. When the experimental results on the bond tests on the deformed bars are considered (type A), satisfactory predictions are obtained thanks to a combination of the conservative character of the code and the customary lower bond performance of deformed bars.

Furthermore, although originally developed for steel reinforcement, the predictions using Eqs. 5 and 6 fall within the point cloud of the experimental results, thus demonstrating their capability to be applied to both NAC and RAC bonded to steel or FRP reinforcement.

4. Conclusions

In this paper, the effect of including recycled coarse aggregates on the properties of hardened concrete and bond behaviour between FRP bars and concrete was investigated. From the results and the discussion the following conclusions have been drawn:

- The physical and mechanical properties of the recycled coarse aggregates used in this study comply with the specifications prescribed in standards for its structural use, and therefore are suitable for concrete manufacturing.

- No unique pattern for the compressive strength variation of concrete, due to the inclusion of recycled coarse aggregate, can be defined as being valid for the three concrete grades (low, medium and high). This is because of the many factors involved, such as the addition of new interfacial zones and aggregates whose origin and properties are usually unknown.

- In the case of low grade concrete (i.e. C20 in this study, with a target of f_c=20 MPa), adding recycled coarse aggregates lead to a decrease in the concrete's compressive strength, no matter what RCA replacement ratio is considered. The variation in the water-to-cement ratio applied to the mix proportions for workability, together with failure occurring through the mortar phase, are the cause of the loss.

- In the case of medium grade concrete (i.e. C30 in this study, with target of f_c=30 MPa), the benefits in compressive strength resulting from RCA inclusion are limited to an RCA replacement ratio equal or less than 50%. For higher replacement rates, the failure plane changes to occur exclusively through the aggregates and the increase in the concrete's compressive strength slows down.

- In the case of high grade concrete (i.e. C50 in this study, with a target of f_c=50 MPa), no clear tendency can be defined. More precisely, the concrete compressive strength clearly increases for the initial RCA replacement ratio of 20%; however, a higher replacement ratio mitigates the growth and the total replacement of the aggregates impacts negatively on the compressive strength.

- Substituting natural aggregate with a recycled concrete aggregate causes no significant change in the bond-slip curves, irrespective of the concrete grade, bar type or RCA replacement ratio considered. Therefore, the bond development and deterioration process between recycled aggregate concrete and FRP bars is similar to that between natural aggregate concrete and FRP bars.

- As in bond between natural aggregate concrete and FRP bars, greater bond strengths are obtained in the recycled aggregate concrete for the increasing concrete grades. Similarly, ribbed bars showed greater bond capacities than deformed ones, providing enough confinement is guaranteed.

- The bond strength in specimens belonging to the lowest concrete grade category (i.e. specimens cast from Mix 1) is greatly affected by the compressive strength of the concrete but with no impact from the bar surface configuration. Therefore, replacing RCA produces no significant variation in normalized bond strength, irrespective of the reinforcing bar type.

- The bond strength in specimens belonging to the medium concrete grade category (i.e. specimens cast from Mix 2) is dependent on the combination of hardened concrete properties and surface configuration of the reinforcement. If the effect of the concrete's compressive strength is removed, greater benefits from replacing RCA are obtained for the normalized bond strength of deformed bars, when compared to that of ribbed bars.

- Satisfactory bond strength predictions by the CEB-FIP model code and other expressions available in the literature were obtained, thus confirming their applicability for bonding between RAC and FRP.

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