Bond behavior of recycled aggregate concrete with corroded and uncorroded reinforcing steel bars

Sònia Garrido Ballart

January 9, 2017

Abstract

Concrete manufacture has a great impact on the environment, as for its production is required a wide range of natural resources. For this reason the recycling of waste concrete from construction is consider as a measure to reduce the environmental impact related to concrete manufacture. Concrete waste recycling allows a more sustainable construction development, environment preservation and green products generation.

To enhance the use of recycled aggregate concrete for structural concrete this paper focuses on the bond capacity between the concrete and the reinforcing steel bars depending on the amount of recycled aggregate used for concrete manufacture. Additionally it was believed interesting to analyze the influence of corroded reinforcing steel bars in the bond behavior of recycled aggregate concrete, since corrosion is one of the most important pathologies suffered by reinforced concrete.

To perform the analysis of the influence of recycled aggregates in the bond capacity of reinforced concrete four doses were designed with different percentages of replacement of natural aggregate by recycled coarse aggregate. With these dosages are made pull – out test with uncorroded bars and pull – out test with three different degrees of corrosion in the reinforcing steel bars.

Comparable bond strength (τ) obtained all the concretes with none corroded bars, however, HR with 100% of recycled aggregates suffered a higher split. After subjecting the concrete specimens to induced corrosion, it was determined that for low corroded specimens, recycled aggregate concretes achieved a higher bond strength performance than conventional concrete, however, for high corrosion levels all concretes obtained minimal bond strength with a similar value.

Key words: aggregates, recycled concrete, corrosion, bond capacity, recycled aggregates.

1 Introduction

Corrosion of the steel bars used as reinforcement in reinforced concrete structures is one of today's main problems. The study of corrosion effects is crucial for a better understanding of the structural behaviour of existing impaired concrete structures. The most severe reinforcement corrosion effect is the change on bond properties between steel and concrete. Moreover, volumetric expansion of corrosion products causes severe problems inducing splitting stresses along corroded reinforcement, with possible resulting damage to the surrounding material. Generally, the splitting stresses are not tolerated by concrete, resulting in cracking and eventually spalling of the cover. As the reinforcement becomes more exposed, the corrosion rate may increase and facilitate the deterioration process.

The bond between concrete and steel is the main affected property caused by corrosion. Steel reinforcement unconfinement due to of concrete cover cracking as well as rust between both materials quickly decreases the bond strength, thus changing the structural behaviour and inducing anchorage failures. Many researchers have studied the effect of the corrosion process on bond deterioration extensively. Several studies have investigated parameters which may influence the bond and anchorage capacity of corroded structures [1–3]. Models studying the interaction between both materials, and numerous experimental studies identifying and studying this phenomena can be found in literature [4–7]. Even though the literature on works covering bond behavior on recycled aggregates is very sparse [8–14].

The increasing amount of construction wastes coming from old and deteriorated structures at its end of service life has a relevant environmental impact in the construction sector due to the economic benefits of using the wastes produced in the form of recycled coarse aggregates (RCA) in the concrete employed in reinforced concrete production. Wastes from older structures yield fragments in which the aggregate is contaminated with various different substances such as gypsum, asphalt, etc. A proper treatment of the recycled aggregate as well as an accurate production process results in recycled aggregate concrete (HR) being a very suitable option to reduce overall cost in the construction sector [15]. Over the past 50 years, the use of RCA has been profoundly studied for concrete production [15–24] and the resulting studies maintain that the major weakness of RCA is its high porosity, which could directly influence a decrease in the compressive strength and durability of concretes produced with those aggregates.

Recent studies have tried to determine the bond between both the recycled aggregate concrete (HR) and the steel with respect to that of conventional concrete (HC) and steel [11]. These studies manifest that a reduction of bond strength could be associated to the amount of recycled aggregate used in the mixture. Several authors [11,13,14] reported reductions of 6-8% up to 30% of bond capacity, nevertheless other researchers work [8] noted differences of approximately 1% between the bond strength of recycled aggregate concrete with respect to that of HC concrete. Although, the reduction in bond strength is strongly related with concrete strength it is also dependent on other parameters such as steel bar rib geometry, and position and orientation of the bars during casting. The amount of concrete covering also has an important influence on this phenomena [8,25–30].

In this research work the influence of the RCA on the bond strength between the recycled aggregate concrete and reinforcement steel either corroded or uncorroded by the use of the direct pull-out tests was analyzed. The obtained results were compared with those obtained from the HC concrete. For that purpose two experimental phases were conducted, during which four different concrete mixtures were cast by replacing 0 (using 100% of natural aggregates, HC concrete), 20% (HR-20), 50% (HR-50) and 100% (HR-100) of natural coarse aggregates by coarse recycled concrete aggregates.

At phase 1, bond behavior from the four types of concretes with steel reinforcement bars of 12 mm was characterized. 100 mm cubic concrete specimens with embedded steel bars were produced in order to carry out the pull-out test. In accordance with [31] specification the uncorroded steel reinforcement bars completely crossed the concrete specimens.

Experimental phase 2 was performed in order to determine the influence of steel corrosion on the bond strength of RA concretes and HC concrete. In this case 10 mm steel bars were used. In order to control corrosion procedure the steel bars were partially embedded in concrete and did not fully cross the entire length of the cubic concrete specimen. Three different corrosion levels were achieved as a result of an applied electrical current on the steel bars in all of the four types of concretes specimens, after which the pull-out test was carried out on each specimen. Concrete initiation cracking and crack length were also measured during the test in order to evaluate the influence of the different percentages of substitute recycled concrete aggregates employed, as well as its higher porosity on the bond capacity due to the steel bars corrosion.

2 Materials

2.1 Materials

Type I Portland cement, CEM I 42.5R, was used in concrete mixtures with the characteristic rapid hardened strength of 42.5 MPa. The chemical properties of cement are given in Table 1.

Composition	%
SiO_2	19.16
Fe_2O_3	3.56
Al_2O_3	5.04
CaO	62.9
MgO	1.66
K_2O	0.75
SO_3	3.54
LOI	3.25

Table 1: Chemical properties of cement.

Natural limestone, fine (FA, 0/4 mm) and coarse aggregates (two fractions; CA1, 4/12 mm and CA2, 12/20 mm) were used for concrete production. Physical properties, density and absorption, and the grading distributions are described in table 2 and figure 1, respectively. The properties of the fine and coarse aggregates were determined according to EN specifications. All fractions of natural aggregates satisfy the requirements specified by Spanish Standard of Structural Concrete [32].

RC aggregates were obtained by crushing rejected 40 MPa compressive strength concrete pro-

Coarse aggregates		Fine		Natural limestone	
Tamiz	% Pasa	Tamiz	% Pasa	Tamiz	% Pasa
20	99,21	12,5	100,00	4	99,44
16	82,72	10	96,15	2	75,88
14	62,17	8	67,45	1	38,49
12,5	42,11	5	2,50	0,5	18,38
10	9,73	4	0,31	0,25	7,64
8	0,00	2	0,00	0,125	2,57
5	0,00	1	0,00	0,063	0,12
4	0,00	0,25	0,00	0	0,00

Table 2: Granulometry of the natural aggeregates

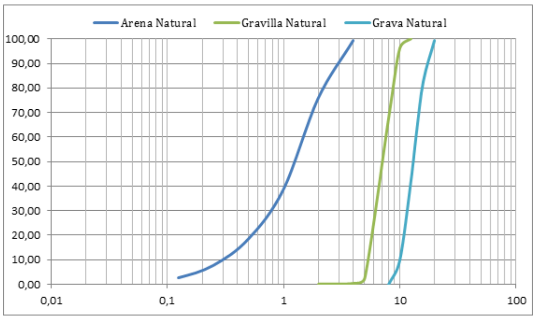


Figure 1: Grading distributions

duced by a precast concrete company. The properties of RCA of density-absorption and grading size are shown in table 3 and figure 1, respectively. The density of RCA was found to be lower than that of the natural aggregates and the absorption capacity higher. The quality of the recycled aggregates obtained from the 40 MPa strength original waste concrete signified that, it was adequate for structural concrete production. The grading was defined by 4/20 mm, it was adequate with respect to Spanish Standard of Structural concrete [32], in its employment for recycled concrete production and it was used in substitution of CA1 and CA2 natural aggregates.

Kg/dm3	GRAVA	GRAVILLA	Arena
Densidad SSS	2.66	2.66	2.63
Desnidad seca	2.65	2.64	2.59
Densidad aparente	2.69	2.70	2.70
Absorción (%)	0.67	0.87	1.70

Table 3: Properties of RCA

At concrete production 4% NaCl in weight of cement was added to the mixture, its aim was the depassivation of the steel inside the concrete and the causing of a conductive medium to facilitate the corrosion procedure. Superplastificizer was also used to provide the desired workability to the mixture.

2.2 Concrete mix proportions

The four mixtures (Conventional concrete, HC; concrete produced with 20% of RCA, HR-20; concrete produced with 50% of RCA, HR-50; concrete produced with 100% of RCA, HR-100) were prepared and cast in the "Structures Technology Laboratory" of the Universitat Politècnica de Catalunya (UPC) – Barcelona Tech (LTE). The replacement of raw coarse aggregates by recycled coarse aggregates was carried out according to volume.

The mix proportion of HC concrete was defined with 300kg of cement and a total water-cement ratio of 0.5 for concretes exposed to a marine and chloride environment described by the Spanish Structural Concrete code [32]. The effective water-cement ratio of HC concrete was determined and maintained constant in all HR concretes, due to the higher water absorption capacity of RCA the total w-c ratio of HR was higher than for HC [21]. The effective w-c ratio on HC concrete was 0.44, this was determined by removing from the total water amount the effective absorption capacity of raw aggregates (the effective absorption capacity was determined by the absorption capacity at 20 min submerged, being defined as 80% and 20% of the total absorption capacity for fine and coarse raw aggregates, respectively).

In order to control the concrete production, recycled coarse aggregates were wetted the day before use by means of a sprinkler system and then covered with a plastic sheet so as to maintain their humidity until used in concrete production. A recommended level of moisture is 80% of their total absorption capacity [21]; however the most important factor was that the aggregates employed were wet in order to reduce their absorption capacity [23]. Due to moderate initial moisture content, recycled aggregate absorb a certain amount of free water and lower the initial w/c ratio in the ITZ, consequently improving the interfacial bond between the aggregates and cement [24]. However, recycled aggregates should not be completely saturated [24], as that would probably result in the failure of an effective interfacial transition zone between both the saturated recycled coarse aggregates and the new cement paste. In line with these recommendations, the compressive strength of concrete made with saturated aggregate was lower than concrete made with air dried aggregates due to the negative effect of the bleeding of the saturated aggregates [20].

Concrete mix proportions were defined according to their maximum volumetric compaction. This mix proportion for conventional concrete (HC) was defined as 50% of fine aggregates and

50% of coarse aggregate. The distribution of coarse aggregate was 30% CA1 4/12mm and 70% of CA2 12/20mm. The same volume of CA1 and CA2 was replaced by RCA for each recycled aggregate concrete production.

Table 4 shows the mix proportions used. The weight of aggregates is given as their dry weight. The total amount of water was considered, including the 80-90% of humidity of aggregates at concrete production. 4% of NaCl and 1% of superplasticizer were used in all concretes with respect to cement weight.

	HC	HR 20	HR 50	HR 100
Cemento (kg)	300	300	300	300
Agua (kg)	150	133.86	133.86	133.86
Arena (kg)	976	976	976	976
Gravilla (kg)	210	168	105	–
Grava (kg)	765	612	382.5	–
Árido Reciclado (kg)	–	171.018	427.544	855.088
Aditivo (%)	1	1	1	1
a/c total	0.5	0.52965	0.57684	0.64776
a/c efectiva	0.4462	0.4462	0.4462	0.4462
NaCl (% peso de cemento)	4	4	4	4

Table 4: Mix proportion

3 Experimental program and test procedure

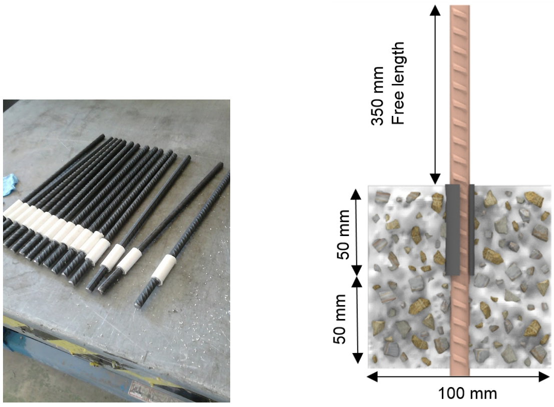
Concrete compressive strength was determined for all types. The bond strength between concrete and steel bars both corroded and uncorroded were analyzed by means of two experimental phases.

3.1 Compression test

The compressive strength of concrete was determined using a compression machine with a loading capacity of 3000 kN. The compressive strength was measured at the age of 28 days following the UNE-EN 12390-3. Three cylindrical specimens (100mm of diameter and 200 mm of length) were used for each type of concrete produced.

3.2 Pull out test

The experimental work presented here focuses on the direct pull out test of natural aggregate concrete and the recycled aggregate concretes, reinforced with steel bars. The underlying purpose of that being to characterize the corresponding bond behaviour. The experimental study was divided in two phases. In experimental phase 1 the test was performed according to the code recommendations [31]. In experimental phase 2, some test specimens were also summited to accelerated corrosion and consequently the embedded bar design was modified with respect to that



(a) Steel bars of 12 mm

(b) Setup for Phase 1

Figure 2: Cubic specimens for Phase 1

criteria, as defined by the code. The pull-out test were carried out through the use of 100x100 mm cubic specimens which had embedded steel in the upper face. For the purpose of this study the top, bottom and sides of the concrete cube shall be known as upper face, bottom face and lateral faces.

3.2.1 Pull out test: Phase 1

The experimental setup was conducted following the code recommendations [31] and using the specific fatigue machine INSTRON 8800 available in LTE.

Steel bars of 12 mm were embedded in the center of the 100x100 mm cubic concrete specimens. The steel bars completely crossed the cube section of 100 mm. A piece of plastic tube was used to debond 50 mm of steel bar from the concrete leaving the other 50 mm to bond with the concrete (see Figure 2b). In total were cast 16 cubic specimens, four for each concrete type.

The total slip between both the concrete and the steel was measured during the test. A LVDT were affixed to the concrete cube at the specimen's bottom in order to register the total rebar slip. Since the total bonded length was less than five times the steel bar diameter, it was possible to assume that the distribution of bond stresses is uniform along the bonded reinforcement section [33,34]. Accordingly, the uniform bond stresses of the specimens can be assessed as such, and the general bond stress could be estimated as:

$$\tau_b = \frac{F_a}{\pi \phi L} \tag{1}$$

where, F_a is the direct applied load to the bar, ϕ is the nominal diameter of the steel bar, and L is the bonded embedded length.

The hydraulic jack clamps were attached to the steel bar end. The load was applied directly

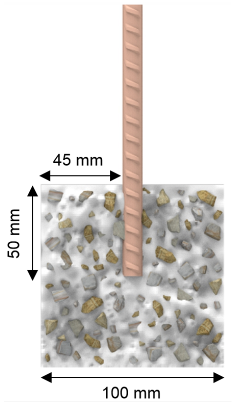


Figure 3: Cubic specimens for Phase 2

to the bar by means of control displacement, in order to achieve results on the pre and post peak behavior up to failure as well as the residual bond capacity. The applied load and the LVDT readings were recorded every half a second by means of a DAQ. The test was conducted applying a constant velocity of 0.2 mm/min. Figure 2 shows the experimental set up.

3.2.2 Pull out test: Phase 2

The phase 2 test specimen design was modified with respect to that defined by code [31] in order to facilitate a simpler and more reliable method of the steel corrosion procedure. 16 cubic specimens of 100 mm were cast for each designed concretes. 10 mm steel bar diameter were used in the experimental phase 2 and the bars, as figure 3 shows, did not fully cross the whole concrete section of 100 mm. The free steel length, for all the specimens, was 350 mm and the bonded section was 50 mm.

The bars were embedded from the center of the upper surface of the cubic specimens to a length which was equivalent to five times the steel bar diameter. The moulds used were specially designed in order to ensure that the bars were fixed at the same position in all the specimens. Figure 3 describes the test specimens and the established setup. The embedded bar was covered by 45 mm of concrete in all directions. This configuration ensured the same corrosion level guaranteed in all the embedded section of the steel bar.

In this case the hydraulic jack clamps also were attached to the steel bar end in order to pull the bar out of the concrete. The tests were conducted by means of displacement control, of 0.2 mm/min, until failure or until the residual bond load was reached.

The load was applied directly to the bar via a hydraulic jack controlled by a load cell. The total displacement as well as the applied load were registered. Each measurement was registered and stored every half a second by means of a DAQ. The registering of the real slip between both materials was impossible to carry out as the bar did not completely cross the specimens. Thus the displacement values registered included all the other external deformations such as slip in the clamps, neoprene compression, concrete deformation, free length steel bar deformation, etc. Consequently the slip values obtained in phase 1 and phase 2 were not director comparable. The max was the only comparable data.

3.3 Accelerated corrosion

3.3.1 Corrosion method

Corrosion of steel reinforcement was induced by means of the passing of an electrical current. Following Faraday's law, Equation 2, it is possible to determine the weight loss of steel at any given time through corrosion, due to the applied intensity of the electrical current I(t), together with the diameter and exposed length of bar.

$$E = \frac{m_{Fe} \int I dt}{VF} \tag{2}$$

Where, m_{Fe} is the atomic mass, V is the steel Valencia that is equal to two and F is the Faraday's constant. The applied intensity being a determined, fixed, known factor remained constant. Consequently it was possible to rewrite Faraday's law as Equation 3, in order to determine the weight loss of steel.

$$\delta m = \frac{m_{Fe} I t}{VF} \tag{3}$$

Many researchers have claimed [35,36] that the use of current densities below 200 μA/cm2 for accelerated corrosion tests causes a similar (5-10% difference) loss of weight in the steel to weight loss of bars estimated by Faraday's law.

Thus, it is possible to accurately estimate the corrosion level achieved by applying electrical current density values below this threshold. Furthermore, current densities above this threshold cause earlier cracking as well as notable difference with the real corrosion procedure and resulting corrosion products. It is known that the bond between steel and concrete is affected by the corrosion rate [37]. In the present work, 1% loss of weight of steel bar by corrosion procedure, at 10 days was estimated and consequently 130 μA/cm^2 of electrical current density was applied. The same electrical current density was also maintained over a period of 20 and 30 days in order to produce 2% and 3% of weigh loss of steel bar due to the corrosion procedure.

In order to produce accelerated corrosion on embedded steel bars by means of passing an electrical current, the steel bars must be depassivated. In this experimental study, as mentioned above, sodium chloride was added to the concrete mixing water in order to remove the passive layer by means of chloride attack.

3.3.2 Corrosion procedure

The specimens produced in phase 2 were placed into plastic recipients and approximately 90% of their volume immersed (90 mm) in water, which contained 5% of NaCl solution (electrolyte). A copper plate was located under the concrete specimens, see figure 4. The copper plate was used as a cathode during the corrosion process and the steel bar was used as anode. The steel bar and copper plate were connected to the power supply equipment, which provided the specified current as defined earlier. A different number of specimens was connected in series guaranteeing the same intensity (see Figure 5).

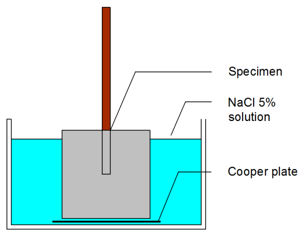


Figure 4: Corrosion procedure for specimens of Phase 2

A naked eye, visual control to define the induced corrosion cracking state was carried out in order to measure the crack occurrence (mainly for first crack). Cracks along the surface were measured every day until the time exposure was reached.

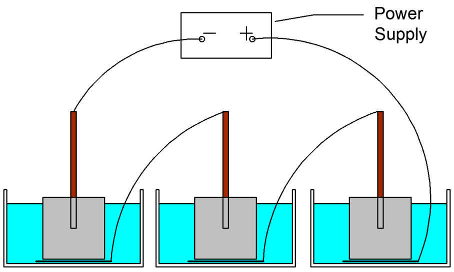


Figure 5: Specimens connected in series to guarantee the same intensity

Three specimens of each type of concrete (HC, HR-20, HR-50 and HR-100) were exposed to the corrosion process at three different ages, 10 days (1% estimated corrosion level), 20 days (2% estimated corrosion level) and 30 days (3% estimated corrosion level). All the specimens were tested by employing the pull-out test described previously (phase 2).

4 Test results

4.1 Concrete properties

Table 5 describes the compressive strength at 28 days of curing and the standard deviation of the values for all types of concretes produced.

Concrete	Compressive strength (MPa)	
	Mean	σ
HC	51.2	0.92
HR 20	48.28	0.66
HR 50	47.8	0.21
HR 100	49.95	3.05

Table 5: Compressive strength

As expected there were no significant drop in the compressive strength of the HR one to the adequate compressive strength (40MPa) of the parent concrete. The weak point of medium–high strength concretes made with coarse recycled aggregates can be determined by the strength of the recycled concrete and its attached mortar [15,21].

4.2 Pull-out test results. Type of concrete influence

4.2.1 Pull out test results: Phase 1

The results of direct pull out test in terms of τ_{max} (maximum bond capacity) for the specimens produced at phase 1 are shown in table 6. The value of τ_{max} was very similar in all the concretes with lower than 10% variation with respect to the maximum bond capacity of HC. That variation was in accordance with the compressive strength obtained by HR and HC concretes, in accordance with other researchers [8,38].

Concrete	Phase 1			
	Experimental	Kim and Yun	Yanciner et al	Moldel code. ‘10
HC	28.64	13.27	24.37	17.88
HR 20	29.69	13.26	23.31	17.37
HR 50	27.14	13.24	23.19	17.28
HR 100	30.31	13.20	23.92	17.66

Table 6: Pull - out test results for Phase 1 compered with the three mathematical models

The LVDT placed underneath the specimen was able to monitor and register the real direct slip between the concrete and steel bar. Figure 6 shows the direct bond-slip curves of the four types of concretes. It shows that the maximum slip registered was higher in HR concretes and it increased when higher percentages of recycled aggregates were used.

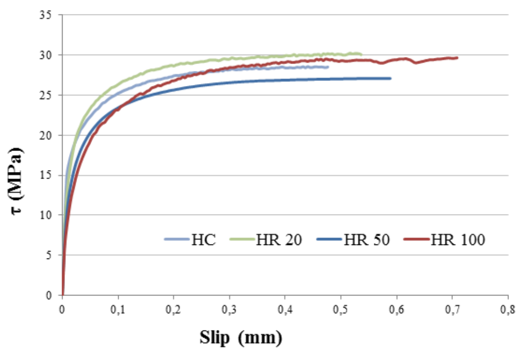


Figure 6: τ - slip for the four types of concrete of Phase 1

Figure 7 shows the average values of the instant τ with respect to τ_{max} ratio for each type of concrete according to the instant slip respect to slipmax ratio. The HC, RCA-20, RCA-50 concretes showed almost the same stiffness with smooth variations in the curves. However RCA-100 presented higher slips for the same τ_{max} values in spite of its very high bond capacity. There was a considerable reduction in stiffness, more than 30%, in the HR-100 concrete when compared to other concretes. A behavior fact which has also been documented by other authors [12]. It is also known that the modulus of elasticity is reduced with the employment of RCA for concrete production [16, 21] and it is coherent with the behavior observed in figure 6. The higher porosity registered for RCA concretes could perfectly explain this behavior in concretes produced with 100% of RCA.

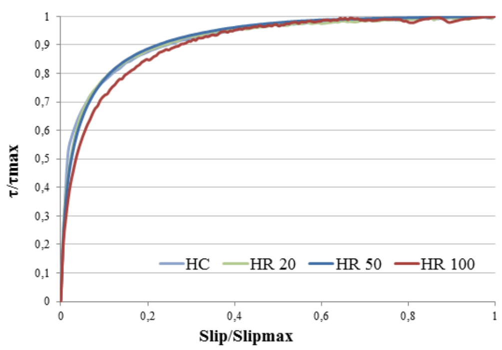


Figure 7: Unitary curves τ/τ_{max} respect to $slip/slip_{max}$ for the four types of concrete of Phase 1

4.2.2 Ultimate bond strength (τ_{max}) estimation models

It is possible to find in the literature few mathematical models which estimate/ predict the bond strength value for HC [30, 39]. As expected, fewer models are designed to determine the bond strength for HR concretes [9,11]. In this section are described the main models and the variables for different types of concrete.

In accordance with bond strength prediction of conventional concrete, the values obtained by Model Code [39] depends on the compressive strength of concrete and the model defined by Yalciner et al. [30] takes into account the compressive strenth of concrete and the ratio between the concrete cover and stell bar diameter for pull-out test setup. The model defined by Kim & Yun [9] for HR concretes, takes into account the compressive strength of the concrete, the ratio between the concrete cover and stell bar diameter for pull-out test setup and the RCA precentage in the concrete by a reduction coeffient.

Table 6 shows the obtained bond strength values of the four types of concretes according to the three mathematical models defined previously byModel Code [39], Kim &Yun [9] and the model defined by Yalciner et al. [30]. The bond strength values predicted by Kim& Yun model [9] understimated the real experimental values obtained by Recycled aggregates cocnretes because it considere by a coefficne that the presence of RCA reduces the bond strength of the concrete.

As it is shown the appliance of those models in the case of similar concrete compressive strength of HR with that of HC concretes results in a big split in between the theoretical and the experimental values. However, a model for HC concretes presented in [30], that only take the concrete compressive strength value as a material properties input thrown very good approximations with the experimental values for all kind of concretes. That fact drives again to the previously mentioned hypothesis, which the concrete compressive strength is the most influential value on.

One important problem related with those ones, is that them don't include important defining variables that characterize the problem. The main problem is that all those models have as main hypothesis a drop of concrete compression strength for HR.

4.2.3 Pull out test results: Phase 2

The τ_{max} value for the four types of concrete specimens, with uncorroded bars, are shown in table 7. max value for recycled aggregate concretes was similar to that obtained for HC concrete.% HR-20 concrete presented a slightly higher τ_{max}, when HR-50 suffered a lower reduction than 10% with respect to the value of conventional concrete. The RCA-100 obtained a similar value to that of HC concrete. These values were in accordance with those obtained in Phase 1 and also in accordance with the compressive strength results obtained by all the concretes.

Concrete	Phase 1 Experimental	Phase 2 Experimental
HC	28.64	34.36
HR 20	29.69	35.78
HR 50	27.14	32.57
HR 100	30.31	35.63

Table 7: Experimental values without corrosion for Phase 1 and 2

Figure 8 shows the ratio between the τ_{max} (phase 1), τ_{max} (phase 2) and the compressive strength for the three RA concretes produced with respect to the same values of HC concrete. As illustrated, the results obtained by recycled aggregate concretes were similar to those obtained by conventional concrete. The reduction of any of the properties was lower than 10% with respect to the values of the latter mentioned. The compressive strengths of the HR were lower than that of HC concrete however, the max value for recycled aggregate concretes reached or exceeded that of conventional concretes except for the concrete produced with 50% of recycled aggregates. In all cases, all the registered variations were attributed to the inherent high dispersion of the pull-out tests.

Figure 8: Ratio between the τ_{max} and compressive strenght of Phase 1 and 2

Figure 9 shows the influence of compressive strength on τ_{max} values. It was determined by the ratio of $\tau_{max-HR}/\tau_{max-HC}$ with respect to f_{c-HR}/f_{c-HC}. All the values were approximately 1 due to the similar compressive strength of all the concretes studied.

Figure 9: Influence of compressive strength on τ_{max} values

4.3 Induced cracking due to corrosion procedure

One of the main purposes of the present work was to describe the behavioral differences of specific studied concretes under corrosion phenomenon. Cracks produced by corrosion are a major problem during the concrete deterioration process, therefore the first crack was produced within an experimental determined parameter. It was necessary to carry out a constant visual inspection to determine the exact time of the first cracking of each type of concrete during the electrical current exposition period. The average value readings on all the different specimens tested are presented in table 8.

Type of concrete	HC	HR 20	HR 50	HR 100
First crack (days)	4.75	6.15	7.66	6.55
Ratio of time (%)	0	29.47	61.26	37.89

Table 8: Time for the first crack for every type of concrete

It was noted that the HR concrete needed approximately 30% more exposure time than conventional concrete for the first cracking to appear. A similar behavior was described in other works for HC concretes cast with different w/c ratios. According to Yalciner et al. [30] the concretes produced with high water/cement ratio needed more time for the first cracking to appear than that on the concretes with low water-cement ratio due to lower porosity of later ones. The high porosity of concrete have similar adequate influence on the happening of the first cracking in all tested concretes.

The recycled aggregate concretes had a higher capacity of absorbing the internal stresses caused by the corrosion products because of their natural high porosity. A higher corrosion level was needed in order to achieve the same external cracking for HR. It was noted that the HR-50 concrete needed a much longer exposure time than that of the conventional concrete for the first cracking to appear, however this concrete had the lowest max value probably because of higher internal damage with respect to the other HR specimens.

(a) (b) (c)

Figure 10: cracks in the upper feces prolonged to the lateral faces during the corrosion procedure

A description of the crack pattern caused by corrosion cracking is described in table 9. The first cracks appeared on the upper face of the concrete cubic specimens, in which the steel bar was embedded. The cracks became prolonged on the lateral faces due to the increase of corrosion level. Table 9 describes the total amount of cracks that appeared: the numeration of the upper face cracks describes the order of occurrence, whilst the lateral face crack's numeration corresponds to the number of the upper face crack prolongation. The max of all the corroded samples is also described in the same table 9.

It must be considered as remarkable that there was no clear relationship between the values obtained by the visual crack measurements and the max achieved for each concrete by means of the direct pull out tests (phase 2). A similar behavior was stated by Yalciner et al. [30], where the bond strength capacity was not attributed to the number of cracks in conventional concretes. That phenomenon could most probably be attributed to the different internal damage on each specimen. Due to the impossibility of observing the internal damage via a visual inspection a post pull-out test inspection was performed on the specimens.

(a) HR - 20 corrosion level 1% (b) HC corrosion level 3% (c) HR - 100 corrosion level 3%

Figure 11: Internal damage for different corrosion levels

Nearly all the specimens showed 2 or 3 upper surface cracks which later became prolonged on the lateral faces. However, with respect to the HR-50 concrete it was noted that after 30 days of being induced with electrical current (of 130 μA/cm2) it only suffered one crack on its upper surface, which later became prolonged on one of the lateral faces. However, the pull-out test revealed serious internal damage (see figure 10), in accordance with the above mentioned hypothesis. Break out corrosion products capacity throughout porous concrete leads to crack pattern change in addition to avoid corrosion products go out faster from the concrete pours mesh. Figure 10 depicts other specimens with differing forms of internal damage, revealing different internal crack formations.

(a) (b)

Figure 12: Steel bars after the corrosion procedure and the pull - out test.

4.4 Corrosion level influence on bond slip relationships

Table 10 describes the results of pull-out tests carried out with uncorroded and corroded steel bars (with 1%, 2% and 3% estimated corrosion level). The standard deviation of the results is also described.

As previously mentioned, HC concrete and HR concretes had similar bond strength when they were not exposed to the corrosion effect. However the bond strength for HR after 10 days of being submitted to an electrical current was higher than that of the HC concrete and this difference was higher when a higher amount of RCA was employed in concrete production. Consequently the concretes produced with a higher amount of RCA presented better behavior in terms of bond capacity under initial corrosion procedures. Basically, that trend was followed for the three tested corrosion levels, nevertheless, this better behavior was significantly decreased as the specimen became more corroded, achieving similar max values after 3% of corrosion level Figure 11 describes the reduction (in %) of max of each concrete at a different cor-

Concrete	Corrosion lever	N° of cracks	Cracks upper faces (cms)			Cracks lateral faces (cms)			τ_{max},(MPa)
			1	2	3	1	2	3	
HC	1%	3	6.0	5.0	4.5	-	3.0	5.5	5.53
		3	3.0	5.0	5.0	-	4.0	6.5	5.47
	2%	3	6.0	5.0	4.5	-	3.0	5.5	4.09
		3	3.0	5.0	5.0	-	4.0	6.5	5.37
	3%	2	4.5	4.5	-	8.0	9.0	-	3.47
		2	4.0	4.5	-	9.0	8.5	-	4.13
		3	4.0	4.5	4.0	10.0	8.5	5.5	4.39
HR 20	1%	3	2.5	5.5	4.5	-	-	2.0	6.72
		2	4.4	4.5	-	1.0	5.5	-	7.13
	2%	2	4.0	4.5		7.0	5.0		6.61
		3	5.0	4.5	5.0	5.0	6.5	7.0	4.92
		3	6.0	5.0	4.0	4.5	3.5	8.0	5.28
	3%	2	4.5	4.5	-	4.0	7.0	-	5.81
		2	5.0	4.0	-	7.5	3.0	-	5.31
		2	5.0	4.5	-	5.5	6.0	-	4.32
HR 50	1%	1	7.0	-	-	-	-	-	6.91
		2	4.0	4.0	-	1.5	6.0	-	6.24
	2%	3	4.0	5.5	6.0	4.0	5.0	-	5.20
		3	4.5	4.5	4.0	8.0	1.0	9.5	5.65
		3	5.0	4.5	5.0	8.0	8.0	6.0	6.30
	3%	1	5.5	-	-	6.0	-	-	5.04
		1	4.5	-	-	8.5	-	-	3.53
HR 100	1%	3	4.7	4.0	4.0	-	-	-	8.40
		2	5.5	4.0	-	5.5	5.5	-	7.67
	2%	3	4.0	4.5	5.0	3.0	8.0	4.5	6.58
		3	4.2	5.5	4.0	1.0	4.0	4.5	6.30
		3	6.0	5.5	4.5	4.5	6.5	8.0	5.49
	3%	2	4.5	4.5	-	7.0	9.0	-	3.90
		3	4.0	6.5	6.0	8.5	6.0	6.0	4.50

Table 9: Evolution of the cracks for the four types of concrete due to the corrosion of the bars during the corrosion procedure.

Concerte	Corrosion – Meanτ_{max} (MPa)							
	0 días		1%		2%		3%	
	Mean	Std	Mean	Std	Mean	Std	Mean	Std
HC	32.38	1.75	5.36	0.04	4.73	0.90	4.00	0.47
HR 20	34.94	3.13	6.93	0.29	5.65	0.73	4.81	0.69
HR 50	28.77	0.21	6.57	0.47	5.72	0.55	4.29	1.07
HR 100	31.84	3.63	8.03	0.52	6.12	0.57	4.20	0.43

Table 10: My caption

rosion level with respect to its max value when the steel was uncorroded. In general term it was possible to observe a better performance of RCA concretes which improved in concretes produced with high percentages of RCA at low corrosion level. However at high corrosion level, the bond capacity was quite similar in all concretes. It is also relevant to note that concrete produced with more RCA, produced a higher drop of max.

Figure 13 shows the ratio of τ_{max} of recycled aggregate concretes with respect to that of conventional concrete at different corrosion levels. It can be asserted that max was better in all of the HR concretes than in the Conventional Concrete, independent of the percentages of recycled aggregates used for concrete production and the corrosion level of the steel bars.

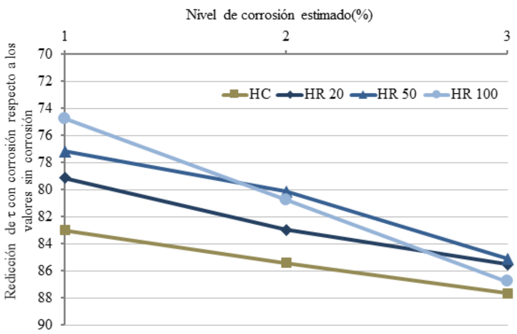


Figure 13: Reduction, in % of τ_{max} for every concrete with different corrosion levels with respect τ_{max} without corrosion.

Increases of up to 40% of bond capacity were registered for recycled aggregate concretes, with very low corrosion levels. The concrete produced with 100% of recycled aggregate achieved the highest max value (see figure 12). The improvement of performance for higher corrosion levels showed increases below those of 10% with which could be defined as being similar in behavior to all of the corroded specimens.

Table 11 describes the bond-displacement behavior of all the tested corroded specimens (2-4 samples were used in each type of concretes). As mentioned above, the displacement showed corresponds to a total displacement which includes slip in the clamps, free steel bar length deformation, etc., as well as the slip between both materials. This is due to configuration of the pull out

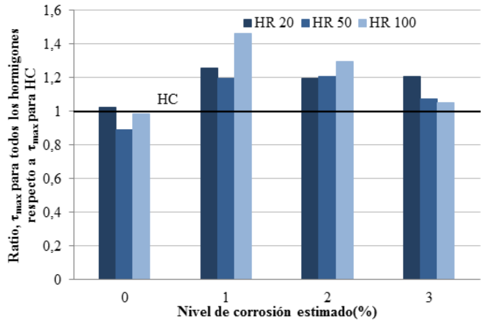


Figure 14: Ratio of τ_{max} for every concrete of Phase 2 with respect to τ_{max} for HC.

test in Phase 2. However, it is possible to observe the specimen's general behavior in terms of τ-displacement. The concretes produced employing a higher amount of RCA a higher total deformation suffered, accord with the results described in Phase 1 where the RCA-100 showed stiffer bond-slip behavior than the other specimens.

The observed trend describes a decreasing bond capacity which was directly related to the extent of corrosion products through the specimens. In contrast, the higher the amount of RCA employed for concrete production the higher the bond capacity achieved at 1% and 2% of corrosion level. However at 3% of corrosion level, HR-50 and HR-100 concretes achieved similar bond capacity to HC concrete.

According to the results obtained, the HR concretes produced with three different percentages of RCA presented a higher bond strength at lower corrosion levels than Conventional Concrete with respect to the corrosion damage propagation of the previously depassivated reinforced bar. Other researchers [39] claimed the reduction of performance of HR in the whole service life, when chloride diffusion was also considered. In that case the performance of HR was worse due to higher chloride diffusion than HC concrete. However, due to the addition of chloride in the mixtures used in this research work and the depassivation of the reinforcement bars the chloride diffusion was not taken into account. HR Concrete achieved 40% higher bond strength than HC concrete, thus clearly showing better performance at low corrosion level. At high corrosion level, all the concretes tend to have a similar bond strength, however, HR concrete always has higher values, approximately 10% higher than HC concrete.

The high porosity of RCA was probably the reason for the adequate bond behavior of recycled aggregate concrete, as they were able to absorb the corrosion products. This was not the case with respect to high corrosion level, as the

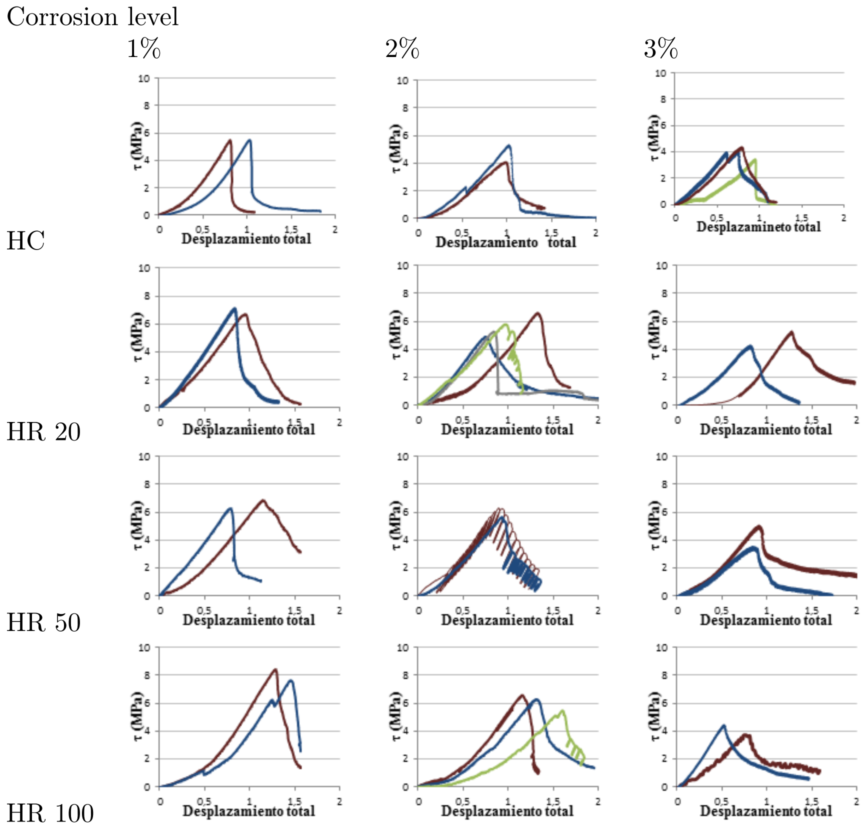


Table 11: τ (MPa) - total displacement (mm) for all the specimens

absorption capacity had already reached it maximum limit and consequently no more corrosion products could be absorbed.

5 Conclusions

Based on the results of the study, the following conclusions can be drawn:

1. The bond behaviour is strongly dependent on compressive strength values. The recycled aggregate concretes which had a similar compressive strength to that of conventional concrete obtained similar or better bond strength.

2. Recycled aggregate concretes produced employing up to 50% of coarse recycled aggregates achieved similar slip and bond strength to those of conventional concrete. However the recycled aggregate concrete produced with 100% of RCA suffered an important stiffness drop.

3. On reaching the same corrosion level it was noted that the initial cracking (visually detectable) of recycled aggregate concretes occurred later than conventional concrete. Furthermore there is no determined relationship between the amount of cracks produced with RA replacement and the corrosion level, nor is there a relationship between the amount of cracks and maximum bond strength.

4. The employment of a higher amount of RCA leads to better bond strength performance at very low corrosion levels. A higher corrosion level caused similar behavior on HR and HC concretes.

5. An increase of the corrosion damage propagation phase in the structural service life of HR concrete is because of its capacity to reduce longitudinal cracking and production of better bond strength performance for low corrosion levels.

It would be advantageous to produce a further work based on an analysis of slower corrosion rates in order to accurately compare the presented conclusions with those of naturally corroded concrete specimens. The resulting values obtained from HR, which would have been subjected to corrosion over long periods of time, simulating closer to natural corrosion procedure, are expected to be defined as behaving even better than those obtained in this research work, consequently improving their adequate property with respect to that of HC concrete.

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