Performance of concrete with recycled coarse aggregates from the prefabrication industry

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Extended Abstract

Master's Degree in Civil Engineering

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1. INTRODUCTION

As the population grows and improves its lifestyle, a proportional increase of the consumption of natural resources and energy occurs. Therefore, and due to the inevitable consequences of this increase, a change of mentality towards the environment has been witnessed. One of the sectors with greater responsibility in the consumption of natural resources and generation of waste is the construction industry, annually producing approximately 850 Mton of waste in the European Union, equivalent to around 31% of the overall production and the greatest of all sectors (EEA, 2009). The construction and demolition waste (CDW) is undoubtedly one of the main focuses of attention in the search for a sustainable construction.

CDW cause various environmental impacts, such as the use of landfill space, illegal deposits, siltation of rivers and lakes, besides the mere wastage of valuable materials. Notwithstanding these negative aspects, only some countries have high recycling ratios of CDW, such as the Netherlands or Denmark with ratios over 90% (EEA, 2011). On the other hand, there are countries such as Spain and Portugal with still low recycling ratios, around 10% and 5% respectively (Vázquez, 2005; Gonçalves, 2005). This discrepancy may be explained by the existence in the first countries of laws that regulate the reuse and recycling, imposing target ratios, and of stiff fees for dumping in landfills (EEA, 2011).

One of the ways or recovering CDW is turning them into aggregates, which are capable of various applications. The Aggregates European Association estimates an annual production of 3000 Mton of aggregates in the European Union, out of which the recycled aggregates (RA) generated represent around 5% (UEPG, 2013). The same Association refers that the countries that generate more RA are Germany and the United Kingdom, with an annual production of 65 and 59 Mton respectively. However, the country with the highest ratio between the RA's production and the overall aggregates production is the Netherlands (around 25%). As for Portugal, it is estimated that the annual production of this type of aggregates is practically nil.

Since concrete is one of the main constituents of CDW (Oikonomou, 2005; Gonçalves and de Brito, 2010), an important part of RA corresponds to crushed concrete. These RA can result from the demolition of concrete structures or the crushing of precast elements or lab specimens. Presently there are diverse applications of the recycled concrete aggregates (RCA), in foundations, pavements, soils stabilization, and reinforced concrete, among others (Oikonomou, 2005; Hansen, 1992).

Some investigations have been conducted in this field, including some performed in Instituto Superior Técnico: Ferreira *et al.* (2011), Fonseca *et al.* (2011), Amorim *et al.* (2012), Matias *et al.* (2013), Barbudo *et al.* (2013) and others. De Brito and Alves (2010) and de Brito and Robles (2010) performed a literature survey of the researches on recycled aggregate concrete (RAC) at national and international levels, respectively. Gonçalves and de Brito (2010) made an extensive survey of the codes that allow the use of RA in concrete. These studies support the possibilities of applying RCA in the production of new mixes of concrete.

The aim of this research is to study this type of aggregates, in order to contribute to increasing the knowledge on the properties of concrete made with them, more specifically coarse recycled concrete aggregates (CRCA) from the precasting industry. An extensive range of properties was analysed, both in terms of the mechanical performance and of the durability of concrete, unlike most of the studies in the literature that focus only some of them. Another goal was to quantify the benefits from using a superplasticizer (SP) in concrete mixes with full replacement of the coarse primary aggregates (CPA).

This research is part of a project that includes the execution of four full-scale two-storey reinforced concrete structures, made with the mixes analysed in this dissertation. The tests of these structures are the most innovative component of the project, since up to now there are no similar studies of full-scale structures made with RCA. These tests include *in situ* characterization tests of the mixes, non-destructive dynamic characterization of the structures and vertical and horizontal load tests, the second of which up to collapse. The first type of these tests is described in this dissertation and the others, as well as the design of the structures, in the dissertation of the student João Nuno Noronha Ramos Vigário Pacheco.

Since generally concrete produced by precasting is of very good quality, it is expected that its use to obtain coarse aggregates for the production of new concrete has advantages over recycling common concrete. Furthermore, the rejects from precast concrete elements have no contaminants whatsoever, unlike common CDW.

With the results of this study it was possible to prove beyond reasonable doubt the feasibility of safely using aggregates replacement ratios higher than those allowed in existing codes (Gonçalves and de Brito, 2010), as long as the quality of the CRCA is demonstrated. The research is thus a contribution to a wider use of RA, resulting in a decrease of the environmental impacts associated to construction practices.

2. EXPERIMENTAL CAMPAIGN

2.1. Materials

The fine primary aggregate used in all the mixes was silica sand, in two size fractions: fine sand - 0-2 mm - and coarse sand - 0-4 mm. Three types of limestone CPA were used: fine gravel - 4-8 mm -, medium gravel - 6-12 mm - and coarse gravel - 12-20 mm. The RA came from crushed precast concrete elements of strength classes C35/45 and C40/50, designed to support very long beams, and only the coarse fractions (4-10 mm and 10-20 mm) were used. The maximum aggregates' size was 22.4 mm.

Type I cement of class 42.5 R and tap water were used. The brand of the SP was SikaPlast 898, considered a high-performance SP, whose chemical basis is a combination of polycarboxylates modified in an aqueous solution.

2.2. Concrete's composition

In the first experimental stage, a total of 11 concrete mixes were formulated. Besides a reference concrete (RC), there were RAC's with replacement ratios of CPA by CRCA of 10%, 20%, 30%, 40%, 50% and 100% (C10 to C100) and also RAC's with 100% CRCA and SP contents of 0.5%, 1.0%, 1.5% and 2.0% by cement weight (C100SP0.5 to C100SP2.0). The replacement of the aggregates was made in terms of volume, resulting in a lower mass of the incorporated CRCA relative to the CPA. The objective of the SP was to decrease the water content in the RAC and consequently decrease the w/c ratio for the same slump. The objective of this stage was to determine the composition of the mixes to be analysed in the next stage, based on the compressive strength obtained.

Two mixes were pre-set for the second: RC and C100 (with the intent of analysing the full scope of the potential loss of performance in a CRCAC). Two extra mixes were intended: one with the maximum CRCA ratio that did not affect the compressive strength; the other with a minimum SP content that allowed offsetting the compressive strength loss of the C100 relative to the RC. In other words, the mixes targeted were the ones with the closest compressive strength to that of the RC. The objective of the second stage was to evaluate the mechanical and durability performance of these mixes.

The third experimental stage corresponded to the *in situ* characterization of the concrete in full-scale structures. In this stage the same mixes that in the second stage were used.

The RC has a conventional composition, with primary siliceous and limestone aggregates, no admixtures or mineral additions. It was formulated according to the procedures in NP EN 206-1 (2007) and LNEC E 464 (2007), using the Faury's method.

The remaining mixes were based on the RC's composition, *i.e.* the constituents proportions were maintained, as well as the aggregates' size distribution. In the first experimental stage all the CPA and CRCA were sieved and separated by size fraction, with the intent of eliminating any differences in size distribution between the mixes. In the second stage, in order to agree with the procedures used on site to build the full-scale structures mentioned in the introduction, the grading curves of each of the elemental aggregates were used and best fitted to the Faury's reference curve. The mixes' slump was also kept constant, in order to have mixes with the same type of use, and therefore more fairly compared in terms of properties. A target slump range of 120 ± 10 mm was thus defined, within the S3 slump class.

In order to keep the RAC's slump constant, the mixing water compensation method was used, which consists of providing an extra amount of water equivalent to the absorption of the CRCA during mixing, as suggested by Ferreira (2007).

The composition of all mixes analysed is presented in Tables 1, for the first experimental stage, and 2, for the second and third stages.

Table 1 - Composition of the concrete mixes: first experimental stage

Components		**Components mass (kg/m^3)**										
		RC	**C10**	**C20**	**C30**	**C40**	**C50**	**C100**	**C100SP**			
									0.5	**1.0**	**1.5**	**2.0**
Coarse primary aggregates	4 - 5.6 mm	109.3	98.4	87.4	87.4	65.6	54.7	-	-	-	-	-
	5.6 - 8 mm	124.7	112.2	99.7	99.7	74.8	62.3	-	-	-	-	-
	8 - 11.2 mm	126.0	113.4	100.8	100.8	75.6	63.0	-	-	-	-	-
	11.2 - 16 mm	322.5	290.3	258.0	258.0	193.5	161.3	-	-	-	-	-
	16 - 22.4 mm	326.1	293.5	260.9	260.9	195.6	163.0	-	-	-	-	-
Coarse recycled aggregates	4 - 5.6 mm	-	10.0	20.1	20.1	40.1	50.2	100.3	105.7	105.4	105.7	105,9
	5.6 - 8 mm	-	11.4	22.8	22.8	45.6	57.0	114.0	120.1	119.8	120.1	120,4
	8 - 11.2 mm	-	11.6	23.1	23.1	46.3	57.9	115.7	121.9	121.6	121.9	122,2
	11.2 - 16 mm	-	30.0	59.9	59.9	119.8	149.8	299.6	315.6	314.8	315.6	316,3
	16 - 22.4 mm	-	30.3	60.6	60.6	121.1	151.4	302.9	319.1	318.3	319.1	319,8
Coarse sand		543.3	543.3	543.3	543.3	543.3	543.3	543.3	572.3	570.9	572.3	573.7
Fine sand		249.6	249.6	249.6	249.6	249.6	249.6	249.6	263.0	262.3	263.0	263.6
Cement		350.0	350.0	350.0	350.0	350.0	350.0	350.0	350.0	350.0	350.0	350.0
Water		182.0	183.1	184.2	185.1	186.9	189.9	193.3	155.7	157.9	154.4	148.0
Effective w/c ratio		0.52	0.52	0.52	0.52	0.52	0.52	0.52	0.41	0.41	0.40	0.39
Apparent w/c ratio		0.52	0.52	0.53	0.53	0.53	0.54	0.55	0.44	0.45	0.44	0.42
Superplasticizer		0	0	0	0	0	0	0	1.8	3.5	5.3	7.0

Table 2 - Composition of the concrete mixes: second and third experimental stage

Component		**Components mass (kg/m^3)**			
		RC	**C25**	**C100**	**C100SP1.0**
Coarse primary aggregates	Fine gravel	176.0	132.0	-	-
	Brita 1	261.4	196.0	-	-
	Brita 2	666.8	500.1	-	-
Coarse recycled aggregates	AGRB 4-10	-	83.3	333.3	352,0
	AGRB 10-20	-	171.0	683.8	722,3
Coarse sand		448,4	448.8	450.1	475.5
Fine sand		243,4	243.8	245.1	258.8
Cement		350,0	350.0	350.0	350.0
Water		185,5	194.8	206.4	159.2
Effective w/c ratio		0,54	0.54	0.54	0.40
Apparent w/c ratio		0,54	0.55	0.59	0.45
Superplasticizer		0	0	0	3.5

2.3. Tests

The PA (coarse and fine) and the RCA (coarse) used in the various concrete mixes were the object of various standard tests:

- Size grading: standard NP EN 933-1 (2000);
- Particles density and water absorption: standard NP EN 1097-6 (2003);
- Bulk density: standard NP EN 1097-3 (2002);
- Los Angeles wear: standard LNEC E 237 (1970);
- Humidity content: standard NP EN 1097-5 (2011);
- Shape index: standard NP EN 933-4 (2002).

In the first and second experimental stages, the fresh-state concrete tests, workability and density, followed the procedures in NP EN 12350-2 (2009) and NP EN 12350-6 (2009), respectively. In the hardened state, the concrete mixes were subjected to the tests listed in Table 3, in terms of mechanical and durability properties. In the last stage, the tests listed in Table 4 were performed, which represent the *in situ* characterization of full-scale concrete structures.

Table 3 - Tests of hardened concrete (laboratory)

Stage	Test	Standard	Age (days)	Number of specimens	Shape and size (mm)
1st	Compressive strength	NP EN 12390-3 (2011)	7	2	Cubes, 150
			28	4	Cubes, 150
2nd	Compressive strength	NP EN 12390-3 (2011)	7	3	Cubes, 150
			28	5	Cubes, 150
			56	3	Cubes, 150
	Splitting tensile strength	NP EN 12390-6 (2011)	28	3	Cylinders, Φ150 x 300
	Modulus of elasticity	LNEC E 397 (1993)	28	2	Cylinders, Φ150 x 300
	Ultrasound pulse velocity	NP EN 12504-4 (2007)	28	5	Cubes, 150
	Abrasion resistance	DIN 52108 (2007)	91	3	Prisms, 71 x 71 x 50
	Shrinkage	LNEC E 398 (1993)	0-90	2	Prisms, 100 x 100 x 450
	Water absorption by immersion	LNEC E 394 (1993)	28	4	Cubes, 100
	Water absorption by capillary	LNEC E 393 (1993)	28	4	Cylinders, Φ150 x 100
	Carbonation resistance	LNEC E 391 (1993)	7, 28, 56, 90	3	Cylinders, Φ100 x 40
	Chloride penetration resistance	LNEC E 463 (2004)	28, 90	3	Cylinders, Φ100 x 50

Table 4 - *In situ* tests of concrete structures

Test	Standard	Age (days)	Tested elements
Compressive strength of the cube specimens	NP EN 12390-3 (2011)	7	3 cubes per floor, 150 mm
		28	5 cubes per floor, 150 mm
Compressive strength of the concrete cores	NP EN 12504-1 (2009); NP EN 12390-3 (2011)	28	Column - 3 cores; beam - 2 cores; slabs - 2 cores
Ultrasound pulse velocity	NP EN 12504-4 (2007)	28	Columns - 8 measurements per floor; beams - 3 measurements per floor
Surface hardness (Schmidt hammer)	NP EN 12504-2 (2012)	28	Columns - 10 horizontal measurements per floor; Beams - 2 horizontal measurements and 2 descending vertical measurements per floor; Slabs - 3 descending vertical measurements per floor

3. RESULTS AND DISCUSSION

3.1. Aggregates properties

Being the major constituents of concrete in terms of weight and volume, aggregates have influence on its fresh and hardened state properties. Therefore, knowing their properties is extremely important to allow a better characterization of the mixes in which they are incorporated. The results of the tests are presented in Table 5 and the size distribution of the aggregates in Figure 1.

Table 5 - Properties of the fine and coarse aggregates

Property		Fine sand	Coarse sand	Fine gravel	Medium gravel	Coarse gravel	RA 4-10	RA 10-20
Particles density (kg/m^3)	Apparent	2537	2622	2708	2756	2717	2654	2665
	Oven dried	2511	2590	2634	2623	2621	2367	2370
	Saturated surface dry	2522	2602	2661	2671	2657	2475	2481
Bulk density (kg/m^3)		1583	1536	1407	1434	1368	1285	1248
Water absorption, 24 h (%)		0.41	0.46	1.04	1.84	1.34	4.57	4.66
Los Angeles wear (%)		-	-	22.3	27.2	31.5	33.3	41.2
Shape index (%)		-	-	16.4	21.7	14.5	11.5	12.2

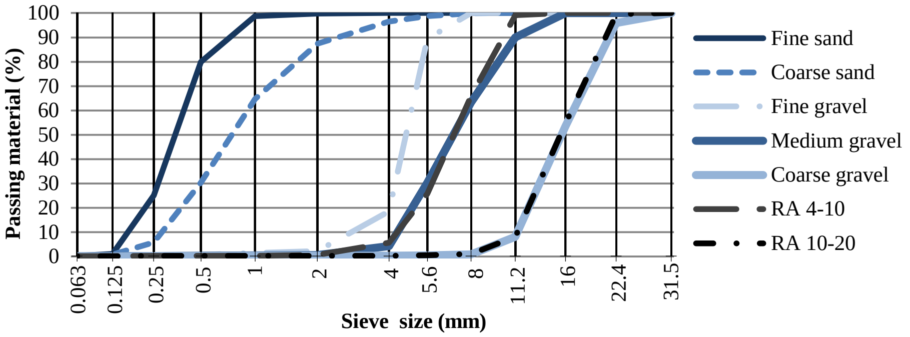

Figure 1 - Aggregates' size distribution

Generally it can be said that the quality of the CRCA was lower than that of the PA (except for the shape index), as a consequence of the mortar adhered to RA. In other words, the particles density and the bulk density of the CRCA are lower than that of the CPA and the water absorption and the wear in the Los Angeles test are higher in the CRCA. However, the loss in the various characteristics analysed was lower than that in previous studies, allowing concluding that the RA used here, coming from precast concrete elements, are of good quality. As a matter of fact, the lower shape index together with the rugosity and porosity of the RA (that contribute to a better bond between them and the cement paste) may lead to an increase of the concrete strength.

3.2. Fresh concrete properties

The workability of the concrete mixes under analysis was fixed *a priori*, *i.e.* in their design, in order to make a more balanced comparison between them. All mixes were to have a slump in the 125 ± 15 mm range, *i.e.* within the S3 slump class (100 mm to 150 mm), and mixes failing to comply with this were rejected and corrected. For the same slump, CRA incorporation did not cause the effective w/c ratio of the mixes to vary, which is explained by their geometric characteristics similar to those of the PA (Matias *et al.*, 2013). The SP's capacity to reduce the w/c ratio ranged from 21% to 26%, *i.e.* there was a significant reduction of the ratio (from 0.52 to 0.41/0.39). However, the influence of the SP content on this capacity is not linear, as Figure 2 demonstrates.

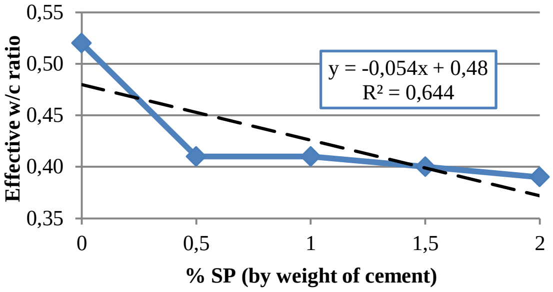

Figure 2 - Correlation between effective water/cement ratio and percentage of SP (by weight of cement)

The density of CRCAC is lower than that of the RC and it decreased linearly as the CPA by CRA replacement ratio increased. The maximum losses were 3.5% and 4.2% (in both cases for full replacement of the CPA), in the first and second stage respectively. Since concrete's density depends directly on its constituents' particles density, this trend was expected. The reduction in w/c ratio caused by the SP addition resulted in an increase of the volume of solid particles (occupying the space previously filled by water) and consequently the density increased. Therefore, the density of the mixes with SP was up to 5.8% higher than that of the corresponding mixes with no SP. These aspects can be established in Table 6.

Table 6 - Density (first and second stages) and compressive strength (first stage)

Concrete mix	Density (kg/m^3)		Compressive strength (MPa) -1^{st} stage	
	1^{st} stage	2^{nd} stage	7 days	28 days
RC	2368	2377	40.9	48.0
C10	2354	-	35.4	46.9
C20	2349	-	38.1	47.7
C25	-	2344	-	-
C30	2342	-	43.2	50.8
C40	2326	-	37.8	48.0
C50	2322	-	40.8	49.5
C100	2284	2278	39.4	50.3
C100SP0.5	2341	-	57.3	63.8
C100SP1.0	2384	2410	58.5	67.6
C100SP1.5	2371	-	57.8	60.8
C100SP2.0	2380	-	59.0	70.3

3.3. Hardened concrete mechanical properties

3.3.1. Compressive strength

3.3.1.1. First stage

The compressive strength of the first experimental stage mixes is presented in Table 6. The ultimate compressive stresses are identical in the CRCAC and RC. At 28 days, the strength of the mixes with 10% and 20% of CRCA was slightly below that of the RC (maximum loss of 2.3%), while the strength of the 30%, 40%, 50% and 100% replacement ratios mixes was higher (increments up to 6%). The maximum strength difference between the C10 and C100 mixes was less than 4 MPa, and it was considered not significant. It is thus not possible to establish any type of correlation between this property and the CRCA content. According to de Brito (2005), similar mechanical strengths between CRCAC and RC can be obtained, for the same aggregates' size distribution and workability, if RCA with current strength and density are used and the concrete strength class is moderate. The identical strength of the concrete mixes may be justified by the shape of the aggregates, their adherence to the cement paste and the quality of the source concrete.

As expected, the strength of the mixes with SP was always significantly higher than that of the remaining ones, but it does not seem to be linearly influenced by the SP content. This increase was caused by the reduction of the effective w/c ratio for the same slump whenever SP was used. There were increments up to 49.6% at 7 days and 39.7% at 28 days, relative to the mixes with 100% CRA without SP.

The compressive strength allowed defining the mixes to be analysed in the second experimental stage. Because the CRCA incorporation had no statistically-significant effect on the compressive strength, a 25% replacement ratio was chosen (C25 mix) because it is the maximum ratio for structural concrete allowed by standard LNEC E 471 (2009) and there was no negative influence of the RA on this property. As for the SP content, 1% of cement weight was chosen because it is used frequently in the industry (and in existing research) and, again, because changes in this content did not have a significant influence on concrete's compressive strength.

3.3.1.2. Second stage

As seen in Figure 3, the results in this stage agree with those from the previous stage.

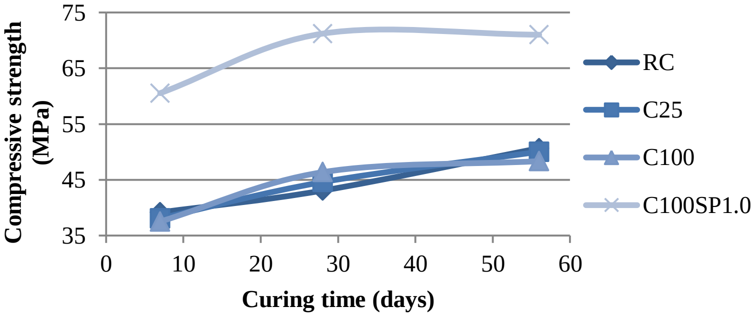

Figure 3 - Evolution of concrete compressive strength over time

The analysis of the evolution curves of the compressive strength over time generally allows splitting the mixes into two groups: RC/C25 and C100/C100SP1.0. The first two mixes registered a strength increase between 28 and 56 days of 17.7% (RC) and 12.2% (C25), and therefore their strength at 56 days may still increase. The strength of the other two mixes stabilized after 28 days with a 4.3% increase (C100) and no increase at all (C100SP1.0; a change of 0.2 MPa may be considered negligible).

It is found that the incorporation of SP resulted in a performance improvement of 61.4% at 7 days (relative to C100) and a reduction of this efficacy at later ages: 53.6% (28 days) and 49.2% (56 days). This confirms the findings of Barbudo *et al.* (2013), *i.e.* the SP accelerates strength development through a faster and more efficient hydration. This can be seen in Figure 4, showing the results of the two experimental stages.

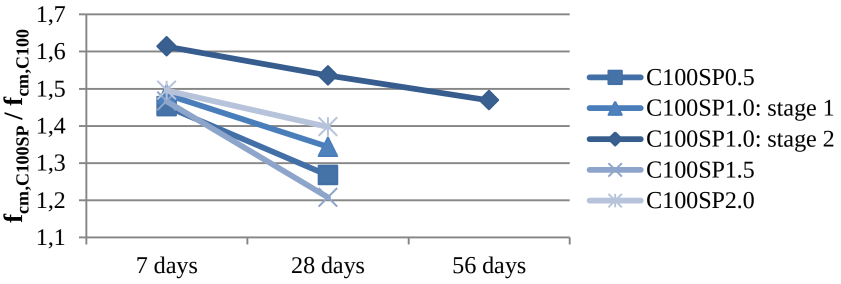

Figure 4 - Concrete compressive strength versus SP incorporation

3.3.2. Splitting tensile strength

The splitting tensile strength of all mixes is presented in Table 7.

Table 7 - Test results of the splitting tensile strength

Concrete	Splitting tensile strength (MPa)	D$_{RC}$ (%)	D$_{C100}$ (%)
RC	3.1	-	-
C25	2.9	-4.2	-
C100	3.1	0.0	-
C100SP1.0	4.3	39.0	39.1

Both mixes with CRCA and no SP had strength values similar to that of the RC, *i.e.* no trend was found linking this property with the replacement ratio of CPA by CRCA. The mix with 100% of CRCA had the same ultimate tensile stress as the RC and the mix with 25% of CRCA registered a value 4% lower. This loss was well below the one seen in previous researches, such as Matias *et al.* (2013) and Rao *et al.* (2011), where the CRCAC had losses of 16% and 24% respectively. This diluted reduction and the fact that it is not linearly related with the replacement ratio may be justified by the better bond between the CRCA and the cement paste, as a consequence of their greater rugosity and porosity (as referred for compressive strength). Malesev *et al.* (2010) refer that tensile strength is affected mostly by the RCA's quality and not their quantity, which agrees with the findings of our study. It is therefore concluded that the good quality CRCA used here had no significant influence on this property.

When SP was incorporated in concrete its tensile strength increased by approximately 39%. It is thus concluded that the use of SP is beneficial for this property, considerably increasing it and more than offsetting the (in this case negligible) negative effect of the CRCA. These trends agree with those detected by Barbudo *et al.* (2013).

3.3.3. Modulus of elasticity

As expected, the modulus of elasticity was lower in the RAC, proportionally to the replacement ratio of CPA by CRCA (Table 8). A maximum reduction of 11% was registered, for full aggregates' replacement. The capacity of the aggregates to resist deformations is controlled by their stiffness and influenced by their porosity. The RA's are therefore more deformable because of the high porosity of the adhered cement paste. As the modulus of elasticity depends on the deformability of all its constituents, it is natural that it is lower in the CRCAC than in the RC (de Brito, 2005; Hansen, 1992). In fact, it is found there is a practically linear relationship ($R^2 = 0.91$) between the relative modulus of elasticity and the replacement ratio of CPA by CRCA (Figure 5).

Table 8 - Test results of the modulus of elasticity

Concrete	Modulus of elasticity (GPa)	D$_{RC}$ (%)	D$_{C100}$ (%)
RC	36.2	-	-
C25	34.1	-5.9	-
C100	32.1	-11.3	-
C100SP1.0	39.5	9.2	23.0

These results agree with various researches (Malasev *et al.*, 2010; Fonseca *et al.*, 2011; Rao *et al.*, 2011), where there is a clear descending trend of the modulus of elasticity as the CRCA content increases. However, the performance loss in our study is lower than in the others, which ranged between 14% and 34%, confirming once again the good quality of our CRCA (as seen in the aggregates' tests).

The mix with 100% of CRCA and 1% of SP has a higher modulus of elasticity than all the other mixes. Unlike in the Barbudo *et al.* (2013) work, the performance loss due to the use of CRCA was more than offset by the use of SP. The increments were of 9.2% and 23.0% relative to the RC and the C100, respectively.

3.3.4. Ultrasound pulse velocity

Table 9 shows the results of the ultrasound pulse velocity tests.

Table 9 - Test results of the ultrasound pulse velocity

Concrete	Ultrasound pulse velocity (km/s)	D_{RC} (%)	D_{C100} (%)
RC	4.71	-	-
C25	4.65	-1.4	-
C100	4.42	-6.2	-
C100SP1.0	4.70	-0.3	6.2

Since the ultrasound pulse velocity is an indirect measure of concrete's porosity, it was expected that it would be lower in the RAC than in the RC, because of the higher porosity of the adhered cement paste in the RA. The trend was confirmed, with a maximum loss of around 6%, for a mix with full replacement of CPA by CRCA. Even though small this velocity decrease agrees with those obtained by Kou *et al.* (2012) and Rao *et al.* (2011), i.e. 8% and 10% respectively. Figure 5 shows a strong relationship, with a correlation coefficient practically equal to 1, between the ultrasound pulse velocity and the CRCA incorporation ratio.

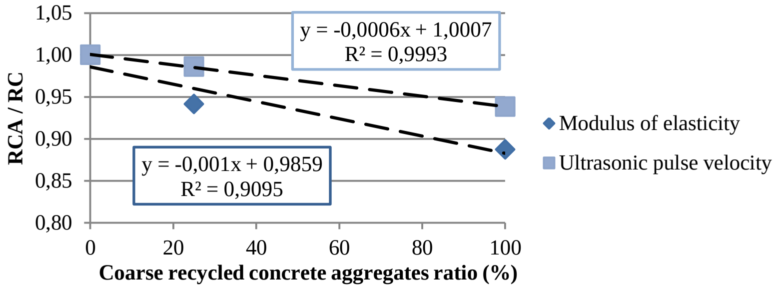

Figure 5 - Influence of the incorporation of CRA on the modulus of elasticity and ultrasonic pulse velocity

The addition of SP led, as expected, to a decrease of the RA negative effect, practically offsetting the velocity loss relative to the RC. In other words, an almost identical ultrasound pulse velocity was obtained in the C100SP1.0 and RC mixes. This is due essentially to the water reducing capacity of the SP used. The increase of ultrasound pulse velocity due to the decrease of w/c ratio is confirmed by the Ravindrajah *et al.* (1988) study.

3.3.5. Abrasion resistance

Table 10 clearly shows that the abrasion resistance increases with the incorporation of CRCA in concrete. In other words, lower wear values were obtained in the CRCAC than in the RC. However, it was not possible to establish a clear correlation between abrasion resistance and CRCA incorporation ratio, as suggested by de Brito (2010). The loss of thickness in the CRCAC was lower by approximately 6% than that of the RC. This can be justified by the better adherence of the cement paste to the RA, because of their greater rugosity and porosity, as referred by Leite (2001) and Poon *et al.* (2004). Because the cement paste is more prone to suffer wear than the PA, it is just natural that the mixes that have a better bond between the paste and the aggregates have lower abrasion wear.

The results obtained in this research agree with those of previous researches, which suggest a better performance of CRCAC in terms of this property (Fonseca *et al.*, 2011; Matias *et al.*, 2013).

Table 10 - Test results of the abrasion resistance

Concrete	Abrasion wear (mm)	D_{RC} (%)	D_{C100} (%)
RC	4.21	-	-
C25	3.94	-6.5	-
C100	3.96	-5.9	-
C100SP1.0	3.30	-21.6	-16.6

Since the CRCAC have better performance in this property compared to the RC, the use of SP in their production is not relevant for abrasion wear. Mix C100SP1.0 showed decreases of 21.6% and 16.6%, relative to the RC and C100 respectively, further improving the performance of the CRCAC in this property. A similar conclusion was drawn by Barbudo *et al.* (2013) and Pereira *et al.* (2012) for CRCA and fine RCA respectively.

3.4. Hardened concrete durability properties

3.4.1. Shrinkage

The shrinkage results are exposed in Figure 6. As expected, since the test was performed inside a room under controlled conditions (temperatures of 20 ± 2 °C and relative humidity of 50 ± 5%), the shrinkage deformations increased over time for all mixes.

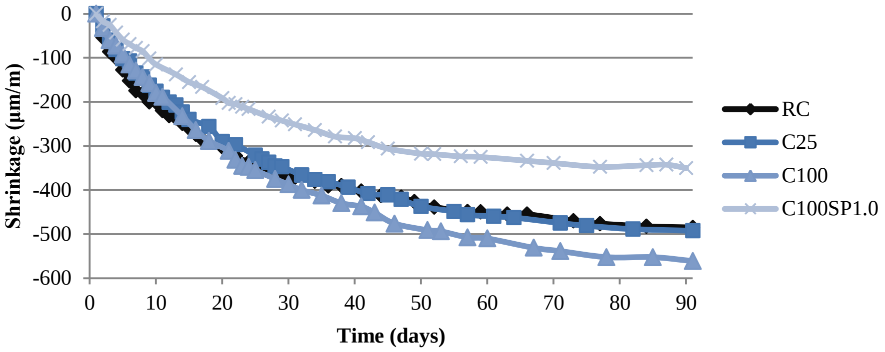

Figure 6 - Evolution of shrinkage deformation over time

The incorporation of CRCA had a detrimental effect on concrete's performance. At 91 days increases (relative to the RC) of 1.5% and 15.7% were observed for the 25% and 100% replacement ratios respectively. This can be justified by two factors: less stiffness (as found out for the modulus of elasticity of concrete) and greater water content of the CRCAC. The first factor is explained by the lower stiffness of the RCAC relative to the CPA, conferred by the adhered cement paste, which is translated into a decrease of the restriction to deformations. On the other hand, the water added to compensate for the higher absorption of the RA leads to a higher content of water available to evaporate in the RAC, and therefore an increase of the shrinkage deformations.

Figure 7 confirms the influence of the incorporation of RCRCA on this property, with a practically linear relationship between the two factors ($R^2 = 0.977$). This is corroborated by the Limbachiya *et al.* (2000) and Matias *et al.* (2013) works.

According to Coutinho and Gonçalves (1994), SP influence the shrinkage deformations when used to increase the workability for the same w/c ratio, causing an increase in shrinkage. When used to obtain higher strength maintaining the workability and decreasing the w/c ratio, they cause a decrease in shrinkage. Since the SP use in this study corresponds to the second case, lower shrinkage deformations were expected in the mix with SP (since there is less water available to evaporate) than in the analogous mix without SP. This was confirmed by the results obtained, with decreases of 27.8% and 37.6% relative to the RC and C100 respectively. The beneficial effect of SP is therefore confirmed, if used to reduce the w/c ratio, resulting in a more than full compensation of the negative effects of the use of CRCA.

3.4.2. Water absorption by immersion

As seen in Table 11, the replacement of CPA by CRCA causes an increase of the water absorption by immersion, and the use of SP in the mix with 100% of CRCA was beneficial for this property.

There were increases of water absorption of 9.1% and 26.9% for the 25% and 100% CRCA incorporation ratios respectively. This is justified by the greater water content in the RCA (necessary to keep constant the workability but increasing the porosity) and high water absorption capacity of the CRAC, since this property depends on the water within the concrete and the pores structures of the aggregates (Coutinho and Gonçalves, 1994). The maximum increase found here is slightly below those obtained by other researchers, *i.e.* values between 29% and 44% (Gonçalves et al, 2004; Malesev *et al.*, 2010; Rao *et al.*, 2011; Matias *et al.*, 2013).

Table 11 - Test results of the water absorption by immersion

Concrete	Water absorption by immersion (%)	D_{RC} (%)	D_{C100} (%)
RC	14.21	-	-
C25	15.51	9.1	-
C100	18.04	26.9	-
C100SP1.0	13.73	-3.4	-23.9

Figure 7 allows analysing the water absorption by immersion evolution with the aggregates' replacement ratio. The high value of the correlation coefficient ($R^2 = 0.991$) of the linear regression proves the linear relationship between these two factors.

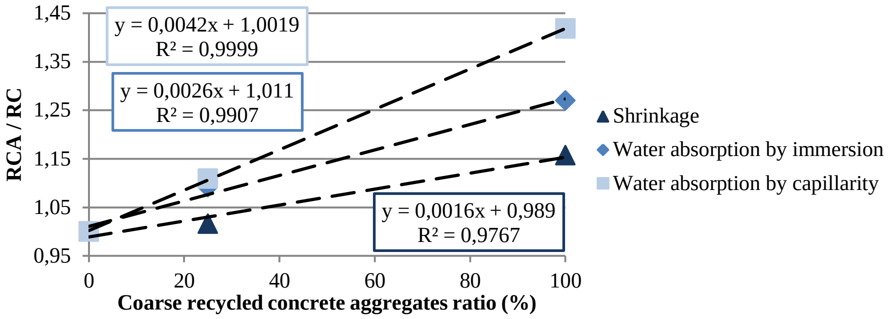

Figure 7 - Influence of the incorporation of CRA on shrinkage, water absorption by immersion and by capillary

Due to the w/c ratio reduction and consequent decrease of the voids index caused by the incorporation of SP in concrete, a water absorption drop occurred. There were reductions of 3.4% and 23.9% relative to the RC and C100 respectively. It is thus concluded that the use of SP is beneficial in terms of this property, resulting in a complete remission of the effect of the CRCA.

3.4.3. Water absorption by capillarity

Table 12 shows the results of the capillary water absorption tests (capillarity absorption at 72 h) and Figure 8 provides the capillary absorption coefficients. Figure 7 shows the linear regression of the capillary water absorption relative to the replacement ratio of CPA by CRCA, with a correlation coefficient almost unitary ($R^2 = 0.999$). It is clear that the capillary absorption increases linearly with the replacement ratio as did the water absorption by immersion. This increase is essentially due to the factors mentioned in the previous section (increase of the water content to compensate for the loss of workability and greater porosity of the CRCAC).

Table 12 - Test results of the water absorption by capillarity

Concrete	Capillary water absorption (g/mm^2)					
	3 hours	6 hours	24 hours	72 hours	$D_{RC,\,72\,h}$ (%)	$D_{C100,\,72\,h}$ (%)
RC	1.80E-03	2.33E-03	3.95E-03	5.51E-03	-	-
C25	1.96E-03	2.53E-03	4.37E-03	6.11E-03	10.9	-
C100	2.58E-03	3.31E-03	5.63E-03	7.81E-03	41.8	-
C100SP1.0	1.25E-03	1.62E-03	2.58E-03	3.58E-03	-34.9	-54.1

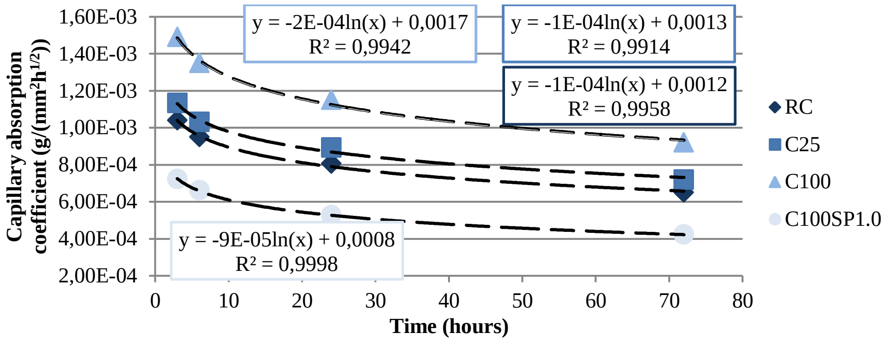

Figure 8 - Evolution of capillary absorption coefficients over time

The addition of SP to the mix led to a decrease of the effect of the RA, completely offsetting the greater capillary absorption of the mixes with CRCA. In other words, the capillary water absorption of the C100SP1.0 mix was lower (around 35%) than that of the RC. Relative to an analogous mix without SP (C100), the reduction was of approximately 54%.

The reductions in this test (35%) are significantly higher than those in the absorption by immersion test (3.4%). This can be justified by the fact that the capillary absorption is more influenced by the quality of the cement paste, contrarily to the absorption by immersion that is more affected by the CRCA content (Gonçalves *et al.*, 2004). Therefore, the performance improvement resulting from the w/c reduction due to the SP (and consequent increase of the cement paste quality) is more important for the capillary absorption.

A logarithmic regression is the best adjusted to the progress over time of the capillary absorption coefficients found, providing for all mixes excellent correlations (coefficients practically equal to 1). As expected, it is found that the capillary absorption coefficient decreases more rapidly in the first hours of the test, tending to stabilize over time, *i.e.* the absorption occurs faster in the first hours of contact of the specimens with water. The similar pattern of the regressions for all mixes indicates that the evolution of the capillary absorption over time is identical in the CRCAC and BR mixes, and the same can be said for the mix with SP. This agrees with the Wirquin *et al.* (2000) study, reported by Levy and Helene (2004).

3.4.4. Carbonation resistance

Table 13 shows the carbonation depths measured in the various mixes and test ages.

Table 13 - Test results of the carbonation resistance

Concrete	Carbonation depth (mm)											
	7 days	D_{RC} (%)	D_{C100} (%)	28 days	D_{RC} (%)	D_{C100} (%)	56 days	D_{RC} (%)	D_{C100} (%)	91 days	D_{RC} (%)	D_{C100} (%)
RC	3.00	-	-	4.51	-	-	8.27	-	-	10.44	-	-
C25	3.16	5.4	-	5.29	17.4	-	8.99	8.8	-	11.21	7.4	-
C100	3.68	22.7	-	5.67	25.8	-	8.95	8.3	-	10.61	1.7	-
C100SP1.0	1.14	-61.8	-68.9	1.89	-58.2	-66.8	3.38	-59.1	-62.2	4.21	-59.6	-60.3

The CRCAC registered carbonation depths at all ages similar but slightly higher than those of the RC. The differences measured ranged between 0.2 mm and 1.2 mm, and therefore they may simply result from the test results' scatter. It is thus concluded that the CRCA had no significant influence on the concrete (accelerated) carbonation resistance, similarly to the compressive strength.

The similar performance of the RAC and RC is due essentially to the very good quality of the CRCA used in this study, sourced from high-strength precast concrete elements. As evidenced in section 4.1, the CRCA have better characteristics than those of similar aggregates in previous studies, especially at the level of particles density, bulk density, Los Angeles wear (the 4-10 mm fraction) and water absorption.

This (lack of) trend contradicts most of the researches concerning CRCA, such as Amorim *et al.* (2012) and Kou and Poon (2012), in their experimental campaigns, and Xiao *et al.* (2012), in their literature survey. However, Levy (2001) also got identical carbonation depths in CRCAC and RC. However, this author used higher cement content in the CRCAC, which might have influenced this property.

The results of the C100SP1.0 mix allow concluding that the use of SP improves the carbonation performance of concrete, since it leads to a reduction of the effective w/c ratio, which results in a decrease of the porosity and CO_2 permeability. The use of SP resulted in improvements between 58% and 62% relative to the RC and between 60% and 69% relative to the C100. Relative to the CRCAC with no SP, the efficacy of the SP decrease over time, since the improvement of 69% at 7 days decreased to 60% at 91 days. These results confirmed what was referred in the section relative to compressive strength, *i.e.* the SP has an effect of accelerating concrete hardening and has a more significant influence in the first days of curing.

3.4.5. Chloride penetration resistance

This property is quantified by the chloride ions diffusion coefficient, as seen in Table 14. The results from two testing ages are coherent, demonstrating no clear influence of the CRCA content. Variations (relative to the RC) of approximately +7% and -3% were observed for the mixes with 25% and 100% of CRCA incorporation respectively. These small differences between mixes may be simply due to the inherent scatter of results of this test. The results agree with those from carbonation resistance, where differences between 2% and 7% were obtained at 91 days. It is thus concluded that this specific type of RCA, from high-strength concrete elements, does not significantly affect this property.

Table 14 - Test results of the chloride penetration resistance

Concrete	Chloride diffusion coefficient (x10^{-12} m^2/s)					
	28 days	D_{RC} (%)	D_{C100} (%)	91 days	D_{RC} (%)	D_{C100} (%)
RC	16.46	-	-	14.13	-	-
C25	17.67	7.3	-	15.14	7.1	-
C100	15.95	-3.1	-	13.77	-2.6	-
C100SP1.0	7.97	-51.6	-50.1	7.31	-48.3	-46.9

Most researches relative to the incorporation of CRCA in concrete, *e.g.* Rao *et al.* (2011), Amorim *et al.* (2012) and Kou and Poon (2012), suggest that there is a chloride penetration resistance loss. However, Limbachiya *et al.* (2000) registered similar performances of the CRCAC, possibly because they also used RCA from precast concrete elements.

As for carbonation resistance, it is concluded that the use of SP results in a performance improvement relative to chloride penetration resistance. The production of the C100SP1.0 mix intended to evaluate whether it would be possible to offset the effect of CRCA; however, because the use of this particular type of RCA did not result in a loss of chloride penetration resistance, the SP-related improvement relative to the RC was very high. The C100SP1.0 mix showed reductions of the chloride ions diffusion coefficient between 47% at 91 days and 52% at 28 days.

3.5. *In situ* concrete characterization of full-scale structures

3.5.1. Compressive strength of the cube specimens

Table 15 shows the compressive strength results at both 7 and 28 days.

Table 15 - Compressive strength of the cube specimens

Concrete	$f_{cm\ 7}$ (MPa)	D_{RC} (%)	D_{C100} (%)	$f_{cm\ 28}$ (MPa)	D_{RC} (%)	D_{C100} (%)
RC	27.7	-	-	31.3	-	-
C25	25.6	-7.5	-	28.8	-7.9	-
C100	26.5	-4.5	-	32.5	4.2	-
C100SP1.0	50.3	81.5	90.0	57.9	85.1	77.6

It was found that the RCAC had no significant influence on the compressive strength of concrete. Despite a 7-day reduction of 7.5% witnessed in the C25 mix, the C100 mix only shows a 4.5% decrease (these reductions are relative to the RC's compressive strength). These results are not conclusive (from a statistical point of view) and can be due to execution conditions and testing variability. At 28 days, the variations registered were -7.9% and +4.2%, respectively for the C25 and the C100 mixes. No correlation between RCAC and compressive strength was established, given the inconclusive results obtained. Some explanations for this fact can be seen in subchapter 4.3.1, which explains the results obtained in the laboratory tests.

As expected, the mix with SP incorporation had the highest compressive strength. This was due to the decrease in the w/c ratio, allowed by the use of this product. Comparing C100 and C100SP, the use of SP increased the compressive strength by 90% and 78%, respectively in specimens tested at 7 and 28 days. It was noted, once again, the higher effectiveness of SP at lower ages.

3.5.2. Compressive strength of the concrete cores

After converting the results of the concrete cores to equivalent cube values, the differences between structures were evaluated. Tables 16 and 17 show the results obtained. In agreement with the standard cubes' results, it was observed that the compressive strength was unaffected by RCA incorporation. The maximum difference between mixes without SP was lower than 3 MPa, i.e. it was not significant. Additionally, the standard deviation bars shown in Figure 4 overlap the values of C25 and RC. The maximum variation, between RCAC mixes and RC, of the ultimate compression was 5.8%.

Table 16 - Estimated *in situ* cube strength of cores (columns, beams and slabs)

Concrete	**$f_{cm,\ in\ situ\ cubes}$ (MPa)**								
	Columns	**D_{RC} (%)**	**D_{C100} (%)**	**Beams**	**D_{RC} (%)**	**D_{C100} (%)**	**Slabs**	**D_{RC} (%)**	**D_{C100} (%)**
RC	-	-	-	27.6	-	-	30.7	-	-
C25	27.5	-	-	27.6	-0.2	-	30.6	-0.6	-
C100	32.6	-	-	30.7	11.1	-	30.0	-2.4	-
C100SP1.0	57.2	-	75.5	53.4	93.4	74.1	52.4	70.5	74.7

Table 17 - Estimated *in situ* cube strength of cores (average)

Concrete	**$f_{cm,\ in\ situ\ cubes}$ (MPa)**	**D_{RC} (%)**	**D_{C100} (%)**
RC	29.7	-	-
C25	28.5	-3.9	-
C100	31.4	5.8	-
C100SP1.0	54.8	84.4	74.3

The analysis of the results in terms of structural elements shows that the slabs had roughly the same compressive strength (maximum difference of 2.4% between C100 and RC). The beams showed a higher difference (11%, between the same concrete mixes).

The differences between the various concrete elements were small, with no structural element with a notably different strength from the others. The maximum difference between elements for the same concrete mix was 11%, also the maximum observed between different elements and different concrete mixes - this reinforces the idea that the incorporation of these RCAC has no significant influence on this property.

The results of the cores taken from C100SP1.0 indicate that the use of SP increases the compressive strength of RCAC, in agreement with the results of the standard cubes. This admixture originated an average compressive strength gain of 74%, of C100SP1.0 relative to C100 - the qualitative and quantitative influence of this product did not depend on the structural elements compared.

Figure 5 shows the trend between compressive strength obtained in cubic specimens (28 days) and concrete cores. The high correlation index ($R^2 = 0.999$) proves the congruence between tests. The values obtained from the concrete cores are slightly lower than the ones concerning cubic specimens, with small differences between 1% and 5%.

3.5.3. Ultrasound pulse velocity

Table 18 concerns the results of the ultrasound pulse velocity test.

Table 18 - Test results of the ultrasound pulse velocity

Concrete	**Ultrasound pulse velocity (km/s)**								
	Columns	**D_{RC} (%)**	**D_{C100} (%)**	**Beams**	**D_{RC} (%)**	**D_{C100} (%)**	**Average**	**D_{RC} (%)**	**D_{C100} (%)**
RC	4.18	-	-	4.03	-	-	4.13	-	-
C25	4.07	-2.7	-	4.05	0.4	-	4.06	-1.8	-
C100	3.93	-5.9	-	3.88	-3.8	-	3.90	-5.4	-
C100SP1.0	4.37	4.7	11.3	4.38	8.6	12.9	4.38	6.0	12.1

The UPV was lower in RCAC structures, proportionally to the RCA's incorporation ratios. By comparing the average values obtained, there was a maximum reduction of 5.4%, corresponding to total replacement of the coarse fraction of the aggregates. A linear correlation between these two properties was calculated and the correlation index obtained was 0.994 (Figure 9).

Both beams and columns exhibited a decrease in UPV due to RA incorporation. The average decrease in this parameter for a total replacement of the coarse fraction of the aggregates was of 3.8% (beams) and 5.9% (columns). Generally, it was found that columns have a slightly higher UPV than beams, thus it can be stated that these elements tend to have a lower void ratio and a higher quality.

The use of SP more than offset the loss in UPV (relative to RC): the concrete mix, C100SP1.0, had the highest UPV values (a 6% increase relative to RC and a 12% increase relative to C100 were witnessed). This is mostly related to the role of the SP as a water reducing agent: since the w/c ratio decreased, the volume of solid particles of this concrete mix increases and the void ratio decreases.

3.5.4. Surface hardness (Schmidt hammer)

As shown in Tables 19 and 20, concerning horizontal and vertical descending measurements respectively, the RCAC tend to have a higher rebound number than the RC mix. There were increments up to 4.2% and 13.3% in vertical descending and horizontal measurements, respectively.

Table 19 - Test results of the surface hardness (columns, beams and slabs)

Concrete	**Rebound Number - horizontal**						**Rebound Number - vertical descending**					
	Columns	**D_{RC} (%)**	**D_{C100} (%)**	**Beams**	**D_{RC} (%)**	**D_{C100} (%)**	**Beams**	**D_{RC} (%)**	**D_{C100} (%)**	**Slabs**	**D_{RC} (%)**	**D_{C100} (%)**
RC	30	-	-	27	-	-	22	-	-	24	-	-
C25	32.5	8.3	-	31.5	16.7	-	24	9.1	-	20.5	-14.6	-
C100	34	13.3	-	34	25.9	-	25	13.6	-	25	4.2	-
C100SP1.0	42	40.0	23.5	42	55.6	23.5	35	59.1	40.0	38	58.3	52.0

Table 20 - Test results of the surface hardness (average)

Concrete	**Rebound Number - horizontal**	**D_{RC} (%)**	**D_{C100} (%)**	**Rebound Number - vertical descending**	**D_{RC} (%)**	**D_{C100} (%)**
RC	30.0	-	-	24.0	-	-
C25	32.5	8.3	-	22.5	-6.3	-
C100	34.0	13.3	-	25.0	4.2	-
C100SP1.0	42.0	40.0	23.5	37.0	54.2	48.0

Almost all the correlations obtained for the various structural elements confirm that there is a clear influence of RCA incorporation on the surface hardness of the mixes produced (Figure 10). Only the linear regression determined for the slabs had an inadequate correlation factor - this is mostly due to the results of the C25 slab (-14.6% relative to RC) that disagree with the values of the remaining mixes, with increases in surface hardness between 8.3% and 16.7%. The columns had a rebound number slightly higher than the beams, in agreement with the trends of the other tests performed.

The results of this study, with higher rebound numbers associated with RCA use, agree with the findings of Al Mufti and Fried (2012). These researchers justified their results with higher water absorption and superficial texture of the RAC mixes, caused by RA incorporation, as causes for a harder surface. As stated by Leite (2001) and Poon *et al.* (2004), these two factors can lead up to an improvement of the cementitious matrix/aggregate binding. The high quality of the RCA used in this study is also another explanation for an increase in rebound number associated with RCA incorporation.

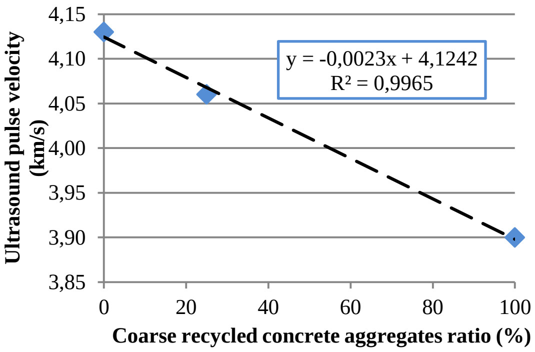

Figure 9 - Influence of the incorporation of CRA on ultrasound pulse velocity

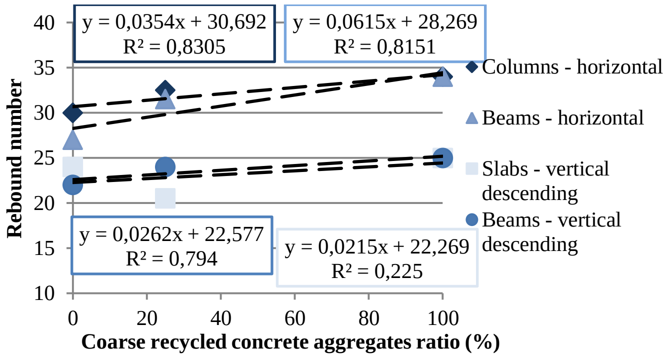

Figure 10 - Influence of the incorporation of CRA on rebound number (columns, beams and slabs)

In agreement with the other properties analysed, the use of SP results in a harder surface; therefore, C100SP1.0 had the highest average rebound numbers (relative to C100, the results were 23.5% higher for horizontal measurements and 48.0% higher for vertical descending measurements).

Ravindrajah *et al.* (1988) studied an exponential regression between the rebound number and compressive strength for conventional concrete. In this investigation the same kind of regression was tested and good correlation indexes were obtained. However, the regression that had the highest indexes (always above 0.90) was linear. Both regressions can be observed in Figures 11 and 12, for cubic and core specimens, respectively.

The surface hardness test had the highest scatter of all tests in the campaign, because of the lower precision of this kind of test, which also only evaluates the superficial concrete - about 3 cm deep, according to Júlio *et al.* (2004).

3.5.5. Comparison between in situ and laboratory results

The comparison between laboratory and *in situ* results is the theme of Table 3.

Table 21 - Comparison between *in situ* and laboratory tests

Concrete	**Cube strength - 7 days (MPa)**			**Cube strength - 28 days (MPa)**			**Estimated cube strength of cores (MPa)**			**Ultrasound pulse velocity (km/s)**		
	Lab.	***In situ***	**D *In situ* / Lab. (%)**	**Lab.**	***In situ***	**D *In situ* / Lab. (%)**	**Lab.**	***In situ***	**D *In situ* / Lab. (%)**	**Lab.**	***In situ***	**D *In situ* / Lab. (%)**
RC	39.2	27.7	-29.3	43.1	31.3	-27.3	43.1	29.7	-31.1	4.71	4.13	-12.4
C25	38.1	25.6	-32.8	44.6	28.8	-35.4	44.6	28.5	-36.0	4.65	4.06	-12.8
C100	37.5	26.5	-29.5	46.4	32.6	-29.7	46.4	31.4	-32.3	4.42	3.90	-11.8
C100SP1.0	60.5	50.3	-17.0	71.2	57.9	-18.7	71.2	54.8	-23.1	4.70	4.38	-6.9

On one hand, it is observed that the difference between RCAC's and RC's results is similar (and negligible) for both testing (and execution) environments; on the other hand, it is understandable that the *in situ* performance is poorer than the performance associated with laboratory execution.

Nevertheless, both environments suggest that these RA have no significant influence on the compressive strength of the mixes produced, and that they only show a slight decrease of UPV associated with RA incorporation. The difference between *in situ* and laboratory results is mostly justified by the different curing conditions.

All the test results show that the difference between *in situ* and laboratory results was the lowest in the mix with SP. This is justified by the different atmospheric conditions during the execution and curing of the structures (and cubic specimens). The RC, C25 and C100 structures were executed during July and August, whilst the C100SP1.0 structure was cast in October.

Concrete Society (1976) recommends an estimation factor of 1.3 as a means of converting *in situ* test results to results in specimens subjected to a standardized curing process, to account for better (wet) curing conditions of the latter. Only the structure C100SP1.0 complied with this figure. The remaining structures showed a higher difference between laboratory and *in situ* compressive strengths (1.45, 1.56 and 1.48, respectively for the RC, C25, C100 mixes), which suggests a curing process worse than expected by this regulation.

Bungey *et al.* (2006) mention that the difference between *in situ* and laboratory compressive strengths ranges between 25% and 35%. On the other hand, Petersons (1964) refers values approximately between 10% and 30%. Ergün and Kürklü (2012) reached similar conclusions in their study. So, the reductions obtained here (between *in situ* and laboratory compressive strengths) agree with these authors' conclusions.

Specifically in terms of compressive strength, it was found that, between 7 and 28 days, the evolution of concrete strength is independent of the concrete mixes (the 7 day strength was 91%, 85%, 81% and 85% of the 28-day strength, respectively for the RC, C25, C100 e C100SP1.0 mixes) - the laboratory results of the 7-day strength were, respectively, 89%, 89%, 81% and 87% of the 28-day strength.

Finally, Figures 11 (compressive strength of cube specimens), 12 (estimated cube strength of cores) and 13 (UPV) present the correlation between in situ and laboratory results. The compressive strength results originated excellent correlation factors (higher than 0.98), unlike UPV results (0.60). However, if the C100SP1.0 mix is not accounted for (dark point in figure), the UPV correlation index is increases to 0.989. The need to disregard this concrete mix for this purpose was anticipated, since in lab conditions the C100SP1.0 had an UPV value similar to the one of RC, whilst in *in situ* conditions this mix had a higher UPV velocity relative to RC. It was considered that the behaviour of the concrete mixes produced in laboratory conditions and in an *in situ* environment was analogue.

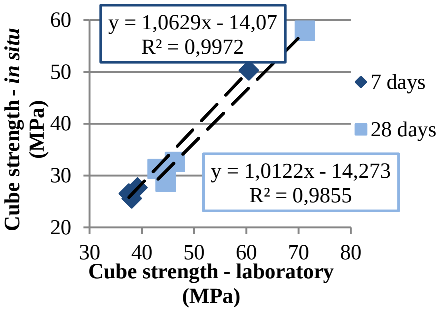

Figure 11 - Correlation between *in situ* (cube strength) and laboratory compressive strength

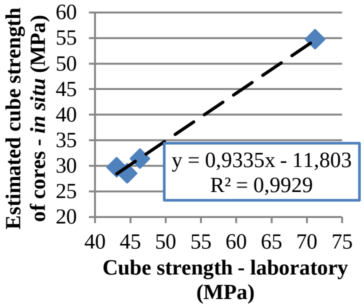

Figure 12 - Correlation between *in situ* (estimated cube strength of cores) and laboratory compressive strength

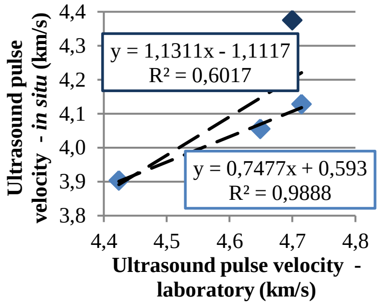

Figure 13 - Correlation between *in situ* and laboratory ultrasound pulse velocity

4. CONCLUSION

This experimental campaign's objective was to evaluate the effect of incorporating recycled concrete aggregates from precast concrete elements on new concrete's properties. The following conclusions were drawn:

1. The aggregates from rejects of the precasting industry are of excellent quality and their performance in terms of the various properties analysed is better than that of current concrete recycled aggregates reported in most of the researches in the literature;
2. Therefore, the normal trends of mechanical and durability performance of the recycled aggregates concrete relative to conventional concrete are totally or partially mitigated, even though not reversed;
3. The comparison between *in situ* and laboratory results proved to be conclusive; all tests showed a clear trend of lower values associated with *in situ* tests. Regardless, the RCAC influence was not affected by this fact, with both experimental campaigns leading to the same conclusions;
4. Coarse recycled aggregates from precast concrete element are capable of being used to produce new concrete (up to 100% content) without losses in terms of most properties. For water absorption by immersion and by capillarity to remain unaffected the incorporation ratio should be limited to 25%;
5. It is possible to increase the present ratio limits of incorporation of this type of aggregates in codes of various countries, if the quality of their source is demonstrated in advance (such as in precast elements);
6. The use of a superplasticizer in concrete is beneficial for all properties analysed. There was a performance improvement, both in mechanical and durability terms, offsetting any negative effect of the recycled concrete aggregates. Therefore, the use of superplasticizers in recycled aggregates concrete is highly recommended, leading to excellent performances.

The coarse recycled aggregates generated from precast elements and the use of superplasticizers open new perspectives for recycled aggregates concrete, enabling the production of high-performance concrete.

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