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Post-fire residual mechanical properties of concrete made with recycled concrete coarse aggregates

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ABSTRACT

This paper presents results of an experimental study on the residual mechanical performance of concrete produced with recycled coarse aggregates, after being subjected to high temperatures. Four different concrete compositions were prepared: a reference concrete made with natural coarse aggregates and three concrete mixes with replacement rates of 20%, 50% and 100% of natural coarse aggregates by recycled concrete coarse aggregates. Specimens were exposed for a period of 1 h to temperatures of 400 °C, 600 °C and 800 °C, after being heated in accordance with ISO 834 time–temperature curve. After cooling down to ambient temperature, the following basic mechanical properties were then evaluated and compared with reference values obtained prior to thermal exposure: (i) compressive strength; (ii) tensile splitting strength; and (iii) elasticity modulus. Results obtained show that there are no significant differences in the thermal response and post-fire mechanical behaviour of concrete made with recycled coarse aggregates, when compared to conventional concrete.

1. Introduction

Construction industry in general and concrete industry in particular are facing considerable pressure in order to reduce their environmental impact. With this in mind, an effort has been made in order to reduce the consumption of natural resources extracted and, in addition, to establish sustainable alternatives for the management of construction and demolition wastes [1,2].

In this context, extensive research has been carried out over the past two decades on the viability of using recycled aggregates in the production of structural concrete. The main effects of incorporating ceramic [3,4] or concrete [5–9] recycled aggregates, or a combination thereof [10], on the mechanical (short-term) and durability performance of concrete are now reasonably well-known. Accordingly, the European Standard EN 12620 [11] already specifies the requirements that recycled aggregates must comply with in order to be used in the production of structural concrete. In addition, several countries have already released national regulations that define the conditions under which those aggregates can be used [12–15]. Nonetheless, in order to allow for the widespread use of this alternative in the construction industry, there are still some issues that need to be addressed, among which the behaviour of concrete with recycled aggregates when subjected to extreme conditions, namely when exposed to fire.

The present study was developed in order to investigate the residual mechanical performance of concrete made with recycled concrete coarse aggregates (CRCCA) after exposure to high temperatures. The research focused on evaluating the residual mechanical properties of CRCCA, namely the assessment of compressive and splitting tensile strengths and elasticity modulus after fire [16]. For this purpose, four different concrete compositions were produced: a conventional reference concrete (RC) and three concrete mixes with replacement rates of 20%, 50% and 100% of natural coarse aggregates (NCA) by recycled concrete coarse aggregates (RCCA). In addition to the reference temperature (20 °C), those compositions were exposed, for 1 h, to temperatures of 400 °C, 600 °C and 800 °C, after being heated in accordance with the standard curve ISO 834.

2. Literature review and research significance

The behaviour of normal strength conventional concrete under fire, which started to be investigated in the 1920s and has been the object of several studies since then, is now reasonably well understood [17–19]. The deterioration of the mechanical properties of concrete subjected to very high temperature stems from the considerable modifications undergone by its components, namely the physicochemical changes in the cement paste and in the aggregates, coupled with the thermal incompatibility between them [19].

At temperatures of 70–80 °C ettringite dissociates and at about 100 °C the water physically bound in both the aggregates and the cement matrix starts to evaporate, increasing capillary porosity and microcracking — at these relatively low temperatures, concrete may only experience a minor loss of strength. At temperatures ranging from 250 to 300 °C the loss of bound water in the cement matrix

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becomes more prominent — a significant loss of strength is often observed. Up to 600 °C, most aggregates undergo thermal expansion and the consequent internal stresses give rise to extensive cracking — at 600 °C the mechanical performance of concrete is already severely affected. From 600 to 800 °C, carbonates suffer decarbonation — for calcareous aggregates, a considerable contraction may occur (due to the release of carbon dioxide) causing severe microcracking of the cement matrix. Finally, from 800 to 1200 °C, calcareous constituents suffer complete disintegration — concrete becomes a calcinated material [20].

It is now clear that these changes are influenced by a considerable number of environmental factors, among which the temperature level, the heating rate, the stress state and the external sealing and moisture content, the latter having also a remarkable influence on explosive spalling. The mix design, namely the type of aggregate and cement and the interaction between them has also a major influence on the way concrete degrades with temperature [19].

In what concerns the behaviour of CRCCA at very high temperatures, very few studies are reported in the literature. If, *a priori*, no remarkable changes are to be expected when compared to conventional concrete, the different constitution of CRCCA (namely, the higher porosity, due to the hardened cement paste adhered to the RCCA, the existence of two aggregate–mortar interfaces) and its possible influence on the residual mechanical performance under fire are worth investigating.

One of the few studies on the post-fire mechanical performance of recycled concrete is the one of Zega and Di Maio [21]. The authors compared the post-fire compressive strength and elasticity modulus of conventional concrete (RC) made with natural coarse aggregate (granitic crushed stone), comprising different water/cement ratios (0.40, 0.55 and 0.70), with that of CRCCA of similar characteristics, incorporating 75% by volume of RCCA. These latter aggregates were obtained from crushing waste concrete produced with granitic crushed stone, but with different strength levels (which were not reported). Cylindrical specimens (150 × 300 mm) were heated in an electric oven, according to a non-standardised heating curve, up to a temperature of 500 °C for periods of around 1 h and 4 h (after which maximum temperatures in the centre of the specimens were about 220–250 C and 450 °C, respectively) and then cooled down slowly. Although, in general, the post-fire performance of RCCA and RC was similar, in terms of both compressive strength and elasticity modulus, for a w/c ratio of 0.40 and 1 h of thermal exposure, the CRCCA presented better mechanical performance than the RC. The authors attributed this result to the similar thermal expansion coefficients between the aggregates and the mortar at the interface of CRCCA, which would reduce the significance of micro- and macro-cracks during the compression process.

In a further study, Zega and Di Maio [22] performed similar tests but now on concrete produced with natural and recycled coarse aggregates of different origins, namely granitic crushed stone, siliceous gravel and quartzitic crushed stone. In these experiments, the authors confirmed the better post-fire mechanical performance of CRCCA, particularly for the lowest w/c ratio (0.40) and when produced with quartzitic aggregates.

Xiao and Zhang [23] evaluated the residual compressive strength of CRCCA with different replacement rates (0%, 30%, 50%, 70% and 100%) of natural siliceous coarse aggregates by recycled siliceous coarse concrete aggregates, obtained from the demolition of an abandoned airport runaway. Cubic specimens (150 mm) were heated according to ISO 834 standard curve up to predefined temperature levels (200 °C to 800 C, in steps of 100 C), which were kept constant for over 2 h and, finally, the oven door was opened and the specimens were cooled down to room temperature inside the oven. The authors concluded that for a replacement rate of 30% the residual compressive strength of CRCCA was lower than that of RC. However, for replacement rates not less than 50% the residual compressive strength of CRCCA became considerably higher than that of RC. In addition, for these higher replacement rates the residual compressive strength noticeably increased for temperatures between 300 C and 500 C — the authors did not provide any explanation for this atypical result.

The study reported here was developed in order to clarify the residual mechanical performance of CRCCA after exposure to high temperatures. The research focused on evaluating the residual post-fire mechanical properties of CRCCA, namely the compressive strength, the splitting tensile strength (not addressed in the above mentioned investigations) and the elasticity modulus. All concrete mixes were produced with limestone aggregates, which had not been tested in previously related studies.

3. Experimental programme

3.1. General methodology

The aggregates used in the concrete compositions produced in the present study were identical to those used in the parent concrete (PC), so the only difference between them was the adhered hardened cement paste in the RCCA. Accordingly, the PC used in this study ($f_{cm,28}^{cube} = 36.3 \pm 1.50$ MPa) was manufactured in a concrete plant and cast in the laboratory using a set of plywood moulds. After 30 days, it was demoulded and crushed using a stone crusher.

The separation of coarse aggregates (natural and recycled) in size types aimed at obtaining a better fit to Faury's theoretical curve and eliminating potentially entropic parameters in the comparison between the different compositions produced. All concrete compositions, with either natural or recycled aggregates, were produced maintaining the same exact grading curves.

Since RCCA have higher water absorption than NCA, in order to keep the effective water/cement ratio (w/c) constant in the different compositions, a methodology was used to ensure compensation during the mixing process. Based on previous experiments carried out by Ferreira et al. [24], it was concluded that the best way to achieve such goal is to add directly to the mix an additional amount of water equivalent to the absorption expected from RCCA during the mixing process, and to create conditions for its absorption.

3.2. Concrete design

In the production of RC natural limestone aggregates were used of the same nature as those of the PC. The fine aggregates ($D < 4$ mm) used were natural fine and coarse sand. The following types of coarse aggregates ($D \geq 4$ mm) were used: granule ($4 \leq D < 5.6$ mm), fine gravel ($5.6 \leq D < 11.2$ mm) and coarse gravel ($11.2 \leq D \leq 25.4$ mm). All coarse aggregates were obtained from rock crushing.

In order to define the concrete composition, a methodology was followed, which allows evaluating the properties of concrete incorporating RCCA. The mix proportions of the produced concrete were designed using Faury's method and the formulation suggested by Gomes [10].

Regarding workability, a slump of 80 ± 10 mm (measured with the Abrams cone) was defined. Maintaining constant this property was considered to be crucial for a consistent comparison of the other properties of the different concrete compositions. A cement type CEM II 42.5 R was used and the maximum particle size was 25.4 mm.

Four different concrete compositions were produced with varying replacement rates of NCA by RCCA — they were designated as RC (reference concrete, with no replacement of NCA by RCCA), C20 (concrete with 20% replacement of NCA by RCCA), C50 (50% replacement) and C100 (100% replacement). The composition of all mixes is shown in Table 1. The apparent w/c ratio concerns the total amount of water introduced in the mix, taking into account the expected absorption by RCCA. On the other hand, the effective w/c

Table 1
Composition of the mixes [kg/m^3].

Component/property		Density [kg/m^3]	Concrete mix			
			RC	C20	C50	C100
Natural coarse aggregates	4–5.6 mm	2600.00	103.712	82.969	51.856	0.000
	5.6–8 mm		117.832	94.266	58.916	0.000
	8–11.2 mm		119.133	95.307	59.567	0.000
	11.2–16 mm		236.807	189.445	118.403	0.000
	16–25.4 mm		407.993	326.394	203.996	0.000
Recycled coarse aggregates	4–5.6 mm	2400.00	0.000	19.147	47.867	95.734
	5.6–8 mm		0.000	21.754	54.384	108.768
	8–11.2 mm		0.000	21.994	54.985	109.969
	11.2–16 mm		0.000	43.718	109.295	218.591
	16–25.4 mm		0.000	75.322	188.304	376.609
Fine sand		2600.00	185.306	185.306	185.306	185.306
Coarse sand		2400.00	478.666	478.666	478.666	478.666
Cement		3100.00	445.474	445.474	445.474	445.474
Water		1000.00	191.554	196.515	203.957	216.360
Apparent w/c ratio		–	0.43	0.44	0.46	0.49
Effective w/c ratio		–	0.43			
Fresh density [kg/m^3]			2413.5 ± 16.1	2392.3 ± 10.9	2355.0 ± 5.9	2299.8 ± 5.1
Slump [mm]			89 ± 2.8	91 ± 6.0	88 ± 7.4	82 ± 4

ratio refers to the effective amount of water available for the hydration process of the cement paste.

3.3. Specimen preparation

For each mix and type of thermal exposure (c.f. Section 3.6.) 5 cubic specimens (150 mm) were cast for compressive strength tests and 3 cylindrical specimens (ϕ 150 × 300 mm) were cast for splitting tensile strength and elasticity modulus tests. After 24 h ± 4 h specimens were placed in a curing chamber (temperature set at 20 °C and relative humidity at 100%) for 28 days. Subsequently specimens were moved to a dry chamber (20 ± 2 °C temperature and 50 ± 5% relative humidity) where they were kept for 21 days in order to dry, because fire exposure of specimens in a saturated state or with high moisture content would most likely lead to excessive spalling. Specimens were exposed to heat at age 49 days and, after cooling down to ambient temperature, 4 days later they were finally subjected to mechanical testing.

Three additional cubic specimens (150 mm) of each mix were produced for subsequent determination of the average value of compressive strength at the age of 28 days (cured in the wet chamber).

3.4. Tests on aggregates

The following tests were carried out on the aggregates: (i) size grading analysis (according to EN 933-1 [25] and EN 933-2 [26] standards); (ii) particle density and water absorption (EN 1097-6 [27]); (iii) water content after drying in a ventilated oven (EN 1097-5 [28]); (iv) loose bulk density (EN 1097-3 [29]); (v) Los Angeles abrasion (LNEC E-237 [30]); and (vi) shape index (EN 933-4 [31]). Table 2 shows the most relevant results obtained in those tests and Fig. 1 illustrates the grading curves of all types of aggregates used in the experiments.

Table 2
Aggregates test results.

Property	Fine sand	Coarse sand	Granule	Fine gravel	Coarse gravel	RCCA
Particle dry density [kg/m^3]	2.61	2.40	2.39	2.44	2.60	2.25
Particle saturated surface-dried density [kg/m^3]	2.62	2.42	2.42	2.48	2.63	2.40
Water absorption [%]	0.2	0.8	1.3	1.3	1.0	6.7
Loose bulk density [kg/m^3]	1.41	1.54	1.47	1.44	1.42	1.28
Water content [%]	–			2.6		
Shape index [%]	–	–	17.5	18.3	14.0	22.3
Los Angeles coefficient [%]	–	–	22.8	28.6	30.7	41.5

As expected, RCCA's densities are lower than those of the NA, due to the higher porosity and lower density of RCCA's adhered mortar. The results also show that RCCA's water absorption is about 6 times higher than that of the NCA, due to the high porosity and water absorption presented by the fraction of adhered mortar of RCCA.

Additionally, RCCA's shape index is about 34% higher than that of the NCA, which means that RCCA are sharper than NCA. RCCA's abrasion loss is about 52% higher than that of the NCA, mostly due to the lower resistance of the cement paste adhered to RCCA.

The evaluation of the water absorption of RCCA, when immersed in water, allowed predicting the behaviour of these aggregates during mixing thus determining the amount of additional water to be introduced in the mix. Fig. 2 shows the evolution of the RCCA's water absorption as a function of time.

The absorption curve reveals that the water absorption of RCCA occurs essentially during the initial instants, reaching about 80% of its absorption potential after only 5 min of immersion. After this period, the water absorption's increase is much slower, tending to a value of 84% after 30 min.

3.5. Tests on fresh concrete

The slump of the fresh concrete mixes was determined by means of the Abrams cone test (EN 12350-2 [32]), and so was their bulk density using EN 12350-6 [33] — results are presented in Table 1. With

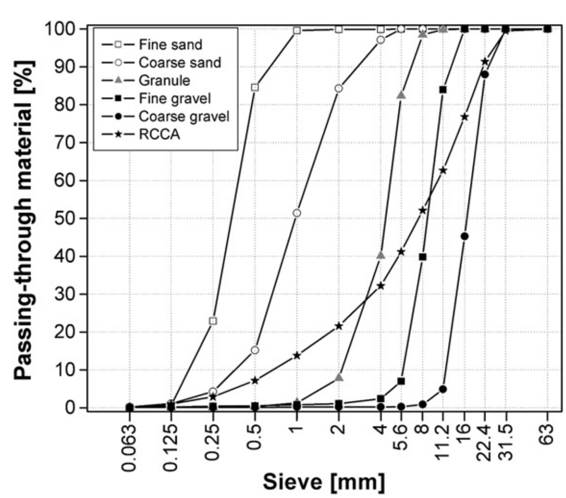

Fig. 1. Grading curves of the aggregates used in the concrete mixes.

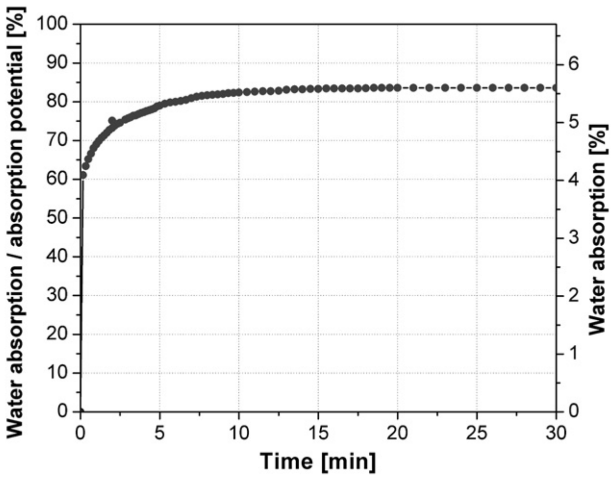

Fig. 2. RCCA's water absorption vs. time.

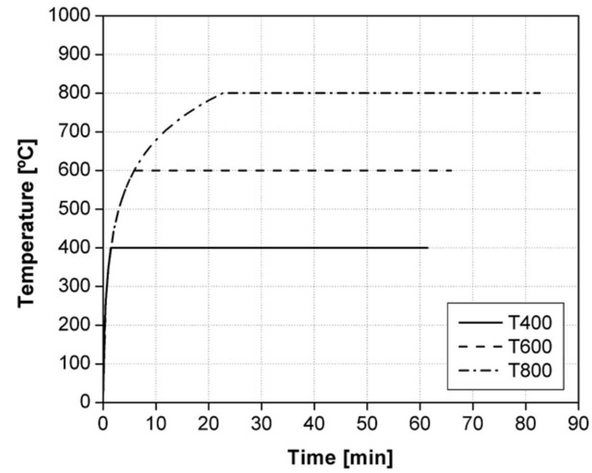

Fig. 3. ISO 834 time–temperature curves for different temperatures of thermal exposure.

only a few exceptions, RCCA's slump remained within the predefined range of 80 ± 10 mm, a requirement considered essential for a valid comparison of the remaining concrete properties. As expected, the fresh density of CRCCA decreases linearly with increasing replacement rates of NCA by RCCA — this trend is due to the density difference between those coarse aggregates.

3.6. Thermal exposure

Specimens from all types of concrete (RC, C20, C50 and C100), besides being tested at ambient temperature (about 20 °C), were subjected to the following three temperatures for a period of 1 h: 400 °C, 600 °C and 800 °C. Specimens were heated in a vertical oven (external dimensions of 1.35 m long × 1.20 m wide × 2.10 m high) that is fired by 6 gas burners controlled by a computer, which reads the oven temperature from 3 internal thermocouples and is able to adjust the burners' intensity in order to follow, as close as possible, a predefined time–temperature curve. Heating was performed with the fire exposure defined in the ISO 834 standard [34], also referred in Eurocode 1 — part 1.2 [35],

$$T(t) = 20 + 345 \times \ log_{10}(8 \times t + 1), \tag{1}$$

where:

- T — oven temperature (°C);
- t — time (min).

The nominal heating time–temperature curves used (T400, T600 and T800) for each exposure temperature (400 °C, 600 °C and 800 °C, respectively) are illustrated in Fig. 3. Cooling of the specimens took place slowly inside the closed oven until temperatures decreased below 120 °C ± 20 °C. Afterwards, the oven door was opened and specimens were taken to the laboratory, where they remained at ambient temperature until tested.

Due to the considerable amount of material to be heated and the limited dimensions of the oven, each thermal exposure had to be carried out in two stages: specimens from concrete mixes RC and C20 were heated in a first batch, and specimens from mixes C50 and C100 were heated in a second batch.

In order to measure the temperature evolution inside the concrete specimens, during casting, three thermocouples type K were placed in one cubic specimen (150 mm) of each concrete composition at depths of 10 mm (T1), 37.5 mm (T2) and 75 mm (T3).

3.7. Tests on hardened concrete after thermal exposure

In order to evaluate the post-fire residual mechanical properties of concrete, the following tests were carried out in specimens cooled at ambient temperature: (i) compressive strength (according to EN 12390-3 [36]); (ii) splitting tensile strength (EN 12390-6 [37]); and (iii) elasticity modulus (LNEC E-397 [38], roughly similar to ISO 6784 [39]).

The average compressive strength at 28 days ($f_{cm,28}$) was also evaluated in specimens from the different concrete compositions, without any fire exposure. Results of those tests, plotted in Fig. 4, show an approximately linear decrease of the average compressive strength with the RCCA replacement level — a maximum variation of about −13% was obtained for concrete C100, when compared to RC.

4. Results and discussion

4.1. Thermal response

Fig. 5 shows the evolution of temperatures inside the oven and at different depths of specimens from all concrete compositions, for the three different types of thermal exposure.

Temperature profiles plotted in Fig. 5 show that for all types of thermal exposure the temperature inside the specimens increases during the heating phase, although more slowly than that of the oven. During the period of constant oven temperature, specimens continue

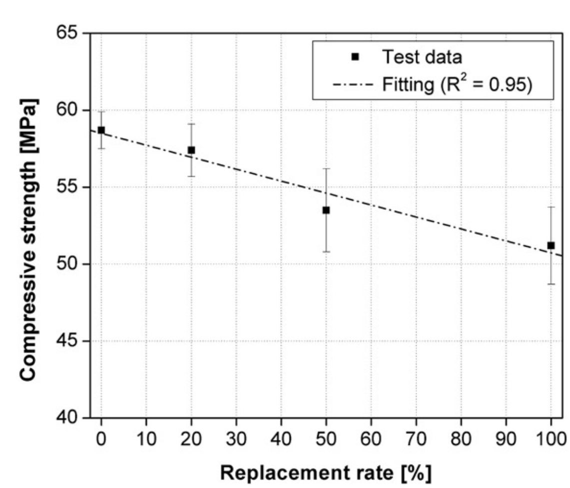

Fig. 4. Variation of compressive strength at 28 days with the rate of replacement of NCA by RCCA.

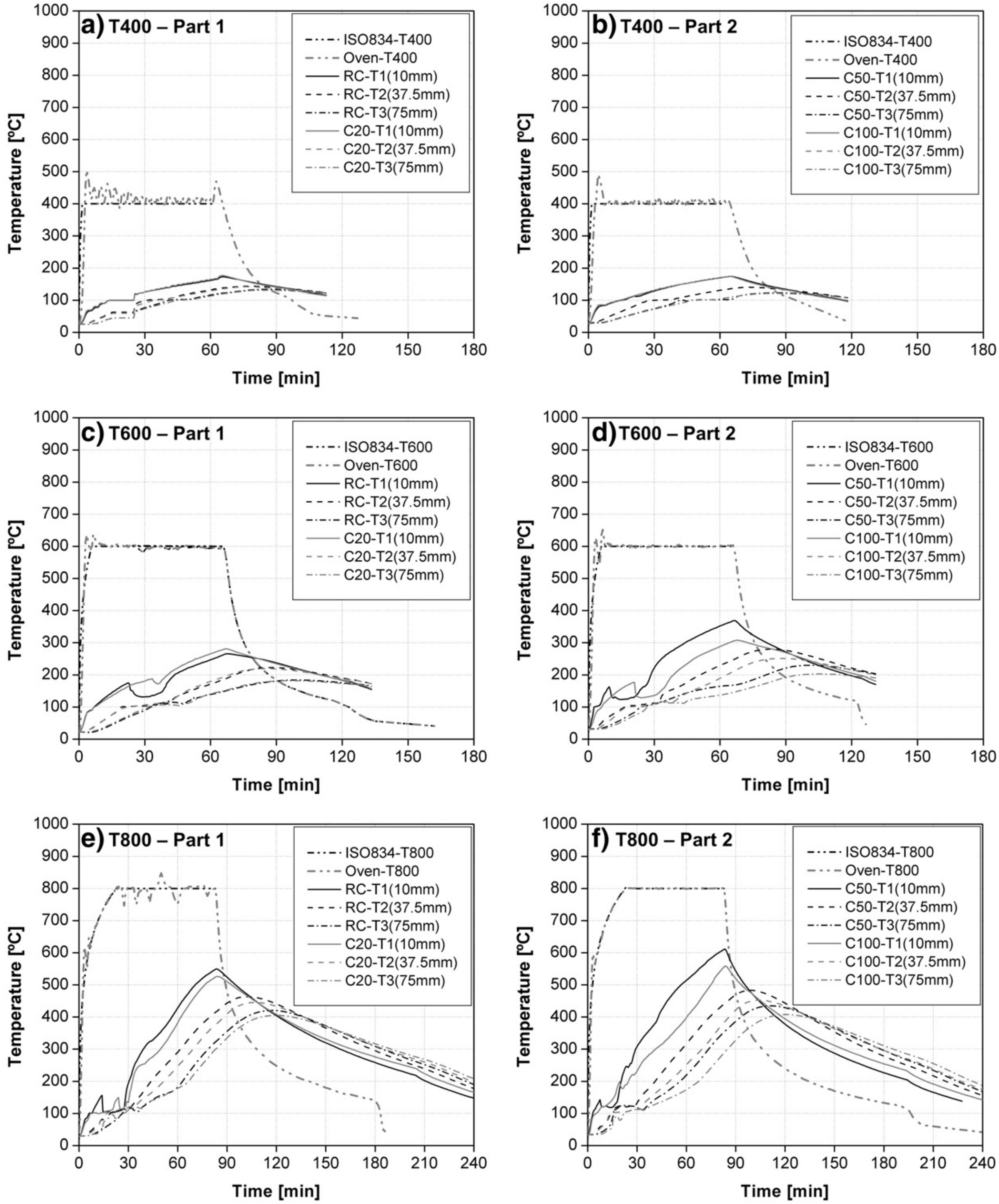

Fig. 5. Time–temperature curves inside the oven and at different depths of the specimens for all thermal conditions: a) T400 (batch 1); b) T400 (batch 2); c) T600 (batch 1); d) T600 (batch 2); e) T800 (batch 1); and f) T800 (batch 2).

exhibiting a gradual increase of temperature at all depths. As expected, temperatures increase during this stage from the surface of the specimens (T1) towards their core (T3). When the oven is turned off, temperatures begin to fall and an inversion in the temperature profiles can be seen during the cooling phase inside the oven, as temperatures at the core of the specimens (T3) become slightly higher than those near the surface (T1).

Fig. 6 presents, for all fire exposures (T400, T600 and T800) and concrete compositions, the maximum temperatures measured at different depths (10 mm, 37.5 mm and 75 mm) of the specimens. As expected, there was an increase of maximum temperatures at all depths with increasing temperature of exposure. For T800 exposure the maximum temperatures achieved at the core (75 mm) of the specimens were slightly higher than 400 °C (they varied between 406 °C and 435 °C), while the maximum surface temperature reached as much as 600 °C. For those levels of temperature a considerable reduction should already occur in the mechanical properties of concrete. For T600 and T400 exposures maximum temperatures achieved inside the specimens were considerably lower, around 300 °C and 175 °C, respectively.

For all thermal exposures, the evolution of temperatures within the cubic specimens followed a similar pattern for the different

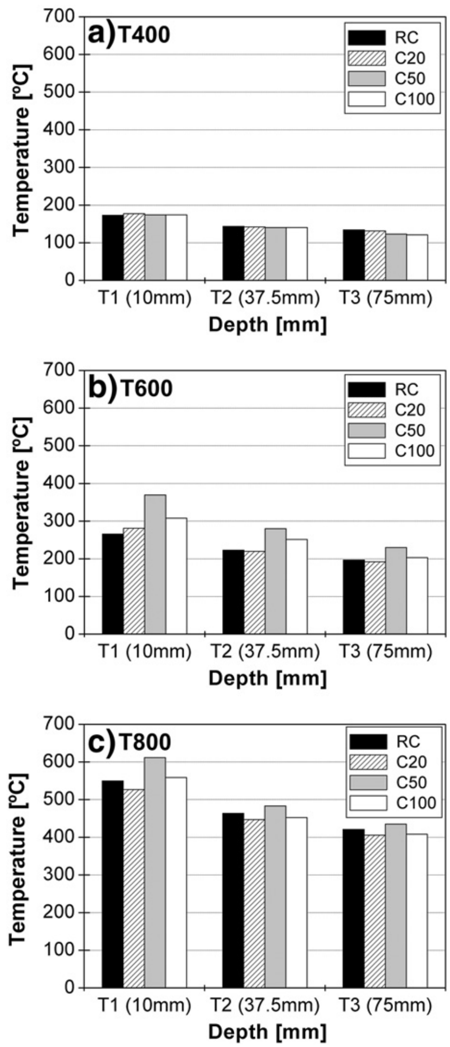

Fig. 6. Maximum internal temperatures reached for each thermal exposure: a) T400; b) T600; and c) T800.

concrete compositions. In fact, it is not possible to identify noticeable differences in the temperature gradients and maximum temperatures reached in the different types of concrete. This means that the higher porosity and the different thermal properties of the matrix–aggregate interface (namely, the thermal expansion coefficient and the thermal conductivity) of CRCCA did not influence their thermal macro-response, when compared with the RC.

4.2. Mechanical performance

Table 3 shows the residual mechanical properties of each type of concrete, for the different thermal exposure conditions — for each condition the percentage deviation from RC (Δ_{RC}) is also provided.

4.2.1. Compressive strength

For each type of thermal exposure average residual compressive strengths do not differ significantly, when compared to RC: a maximum variation of only −12.2% was obtained for concrete composition C20, for thermal exposure condition T800.

Fig. 7 presents the ratio between residual strength for each exposure temperature (f_c^T) and the strength for the reference temperature of 20 °C (f_c^{20}), for all types of concrete. Curves plotted in Fig. 7 confirm that the decrease of strength with increasing temperature is similar for the different types of concrete. The highest decrease occurs between 20 °C and 400 °C, with a much lower decrease taking place between 400 °C and 600 °C. This variation pattern is due, to a great extent, to the effect of the temperature gradient imposed on the samples, stemming from the steep initial temperature increase of ISO 834 curve. In fact, as already mentioned, the maximum temperatures reached inside the concrete specimens at different depths were generally below the typical critical range (between 300 °C and 400 °C) after which concrete experiences an important permanent degradation of its internal structure. For temperatures between 600 °C and 800 °C there is a sharp compressive strength decrease for the different types of concrete. For T800 exposure the maximum temperatures measured inside the concrete specimens at different depths exceeded 400 °C; therefore, in this case the reduction of compressive strength was due not only to the effect of the temperature gradients but also to the absolute temperatures achieved by the material.

On average, for exposure temperatures of 400 °C, 600 °C and 800 °C the concrete compositions tested presented residual compressive strengths 78%, 75% and 60%, respectively, of the corresponding ambient temperature strength (at 20 °C).

Taking into account the similar pattern of variation of the residual compressive strength with temperature exhibited by the different types of concrete compositions, for test conditions similar to those used in the present study, the following equation is suggested to estimate the residual compressive strength of CRCCA as a function of the exposure temperature (where T is the maximum temperature inside the oven, in °C):

$$f_c^T / f_c^{20}(T) = 1.0 - \frac{T-20}{2000}. \tag{2}$$

Fig. 8 compares the results of the present research, in terms of residual strength as a function of both oven temperature (T_{oven}) –

Table 3
Residual mechanical properties and deviation from RC (Δ_{RC}).

Residual mechanical properties	Concrete mix	Number of specimens	Thermal exposure							
			T20		T400		T600		T800	
			Average	Δ_{RC} [%]	Average	Δ_{RC} [%]	Average	Δ_{RC} [%]	Average	Δ_{RC} [%]
Compressive strength:	RC	5	67.6	–	53.7	–	50.1	–	41.1	–
f_c [MPa]	C20		67.7	0.2	51.1	−4.9	51.0	1.8	36.1	−12.2
	C50		65.3	−3.4	51.5	−4.2	48.5	−5.0	37.0	−10.0
	C100		63.6	−5.9	50.8	−5.4	48.8	0.7	42.9	4.3
Splitting tensile strength:	RC	3	3.2	–	2.8	–	2.2	–	1.3	–
f_{ct} [MPa]	C20		2.8	−12.2	2.5	−11.6	2.1	−3.3	0.8	−36.1
	C50		3.9	22.6	3.0	6.1	2.1	−2.8	1.4	8.6
	C100		3.4	5.5	2.5	−11.9	2.2	−0.7	0.8	−34.8
Elasticity modulus:	RC	3	43.7	–	13.2	–	11.1	–	5.4	–
E_c [GPa]	C20		36.7	−16.2	12.1	−7.9	10.0	−9.9	3.9	−28.0
	C50		40.4	−7.6	12.4	−5.6	10.0	−9.8	4.6	−15.1
	C100		33.2	−24.1	11.3	−14.1	8.7	−21.7	3.5	−35.1

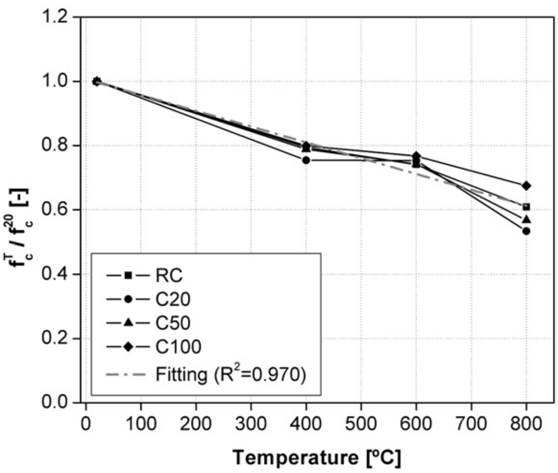

Fig. 7. Residual compressive strength as a function of temperature for concrete compositions with different RCCA replacement rates.

used to derive Eq. (2) – and maximum temperature reached by the test specimens (T_{max}), with results of Xiao and Zhang [23] and the reduction factors recommended in EC2 — part 1.2 [40] and DTU [41]. In addition, one should note that the curve suggested by Xiao and Zhang was derived based on experimental results obtained under different conditions compared to those used in the present study: siliceous NCA were used (instead of calcareous) and the different concrete compositions were exposed to high temperatures for a period exceeding 2 h (maximum temperatures reached by test specimens were not reported). Also reduction factors of EC2 are for normal strength concrete with total incorporation of calcareous NCA tested at very high temperatures, while reference values provided in DTU refer to normal strength concrete without incorporation of RCCA, also tested at high temperatures — thereby, the reduction factors suggested in EC2 and DTU are plotted only as a comparative reference, since they do not correspond to residual strength after cooling; furthermore, they refer to concrete specimens uniformly heated throughout their cross-section up to a given temperature.

Fig. 8 shows that Xiao and Zhang's curve is more conservative than the one suggested herein regarding the values of residual compressive strength — this must be attributed to the longer heat exposure (2 h) used in those experiments. In the present study for temperatures below 360 °C predictions are more conservative than those corresponding to EC2 and DTU reduction factors — taking into account the maximum temperatures achieved in the concrete specimens for the less severe heat exposure (T400), such difference must be due to the thermal gradients caused by the steep heating regime. For temperatures between 360 °C and 440 °C the reduction curve suggested lies in-between the reference values of EC2 and DTU and for higher temperatures the proposed reduction factors become higher than those suggested in those two standards. As a matter of fact, for the most severe thermal exposure (T800) the maximum temperatures within the test specimens remained well below the oven temperature (they varied between 526 °C and 611 °C). For this higher temperature range it is noted that the reduction factor corresponding to the maximum temperatures achieved by the test specimens, whose average value is also plotted in Fig. 8, is in much better agreement with those of EC2 and DTU.

4.2.2. Splitting tensile strength

As for compressive strength the residual splitting tensile strengths of the different compositions of CRCCA present small variations compared to RC, regardless of the type of thermal exposure. The only exception concerns thermal exposure T800, for which a maximum variation of −36.1% was obtained for concrete composition C20.

For all concrete compositions Fig. 9 illustrates the ratio between the residual splitting tensile strength for each exposure temperature (f_{ct}^T) and the corresponding strength for the reference temperature of 20 °C (f_{ct}^{20}). The evolution of the curves of the different types of concrete is roughly similar. The highest reduction occurred between temperatures of 400 °C and 800 °C for types RC and C50, and between 600 °C and 800 °C for types C20 and C100.

On average, the concrete compositions tested presented residual splitting tensile strengths 82%, 65% and 32% of the ambient temperature strength (at 20 °C), for exposure temperatures of 400 °C, 600 °C and 800 °C, respectively. Those values are considerably lower than the corresponding compressive strength residual values — this is in agreement with previous studies (e.g. [42]) and stems from the fact that tensile strength is more sensitive than compressive strength to micro- or macro-cracks produced in the specimens due to thermal incompatibilities.

For conditions similar to those used in the present study the following quadratic equation is put forward in order to estimate the residual splitting tensile strength of CRCCA as a function of temperature (where T is the maximum temperature inside the oven, in °C):

$$f_{ct}^T / f_{ct}^{20}(T) = 0.997 + 2.19 \times 10^{-5} \times T - 1.07 \times \left(\frac{T}{1000}\right)^2 \quad (3)$$

Fig. 10 compares this experimental equation (Eq. (3)), in which temperature corresponds to the heat exposure (T_{oven}), with the

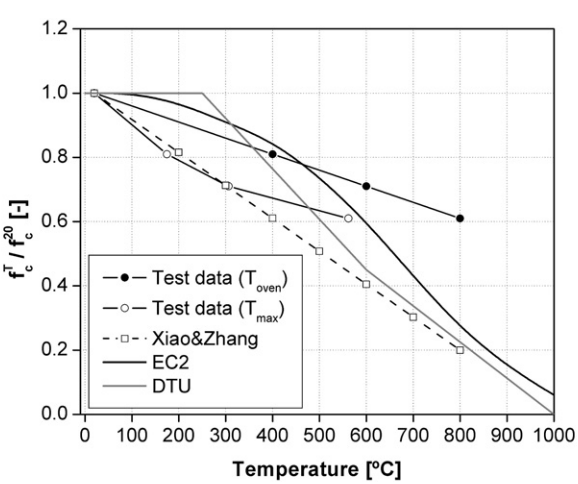

Fig. 8. Comparison between the curves proposed in this study, the one suggested by Xiao and Zhang and the reference values of EC2 and DTU.

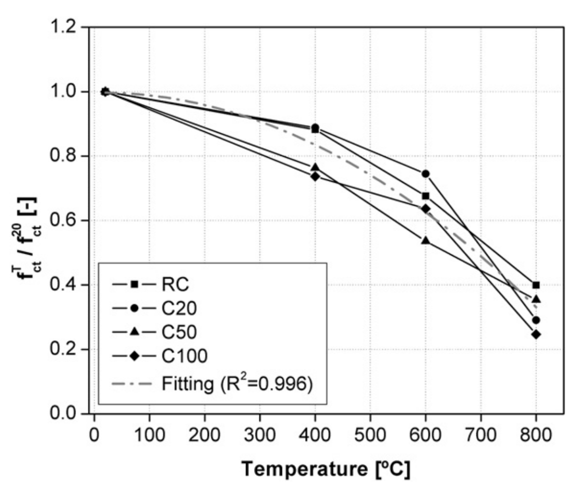

Fig. 9. Residual splitting tensile strength as a function of temperature for concrete compositions with different RCCA replacement rates.

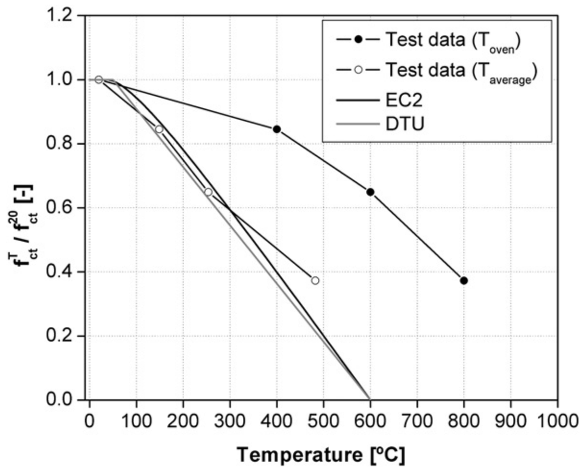

Fig. 10. Comparison between the curves proposed in this study and the reference values of EC2 and DTU.

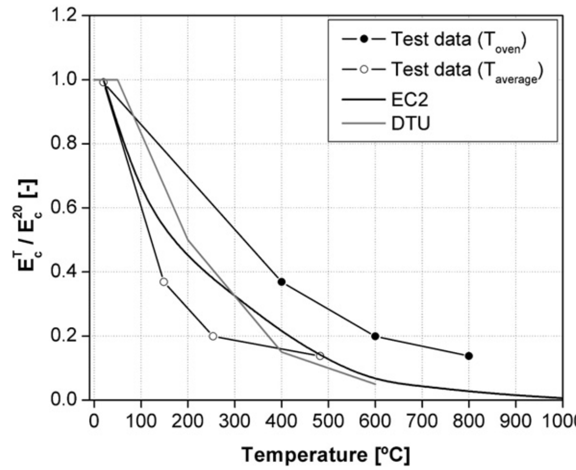

Fig. 12. Comparison between the curves proposed in this study and the reference values of EC2 and DTU.

reference values of EC2 — part 1.2 and DTU. In addition, the reduction factors evaluated in the present study are also plotted in Fig. 10 as a function of the maximum average temperatures reached inside the test specimens ($T_{average}$).

The experimental curve suggested in this study, based on maximum oven temperatures, shows a considerable deviation from the values suggested in EC2 and DTU (that are roughly similar), which are much more conservative. However, when the maximum average temperatures achieved within the test specimens are considered, then the experimental reduction factors present a very good agreement with those of the two standard curves.

4.2.3. Elasticity modulus

For each type of thermal exposure the residual elasticity modulus of CRCCA shows noticeable differences from that of RC: a maximum variation of −35.1% occurred for concrete composition C100, for thermal exposure T800. Results following exposition to very high temperatures are consistent with those at ambient temperature, for which the highest performance reduction, stemming from total replacement of natural aggregates, also occurs for the elasticity modulus.

Fig. 11 shows the ratio between the residual elasticity modulus for each exposure temperature (E_c^T) and the elasticity modulus for the reference temperature of 20 °C (E_c^{20}), for each type of concrete. As for the two other mechanical properties analysed, the values obtained for the decrease in the residual elasticity modulus as a function of exposure temperature are quite similar for the different types of concrete. The highest residual elasticity modulus decrease occurred between 20 °C and 400 °C.

On average, the concrete compositions tested presented residual elasticity modulus 32%, 26% and 11%, of the ambient temperature value (at 20 °C), for exposure temperatures of 400 °C, 600 °C and 800 °C, respectively. The elasticity modulus was the mechanical property whose residual performance was most affected by high temperatures — this can be associated with the high level of cracking observed in the concrete specimens and is consistent with previous studies on conventional concrete (e.g. Ref. [43]).

A quadratic equation is then suggested to estimate the residual elasticity modulus of CRCCA as a function of temperature that, as for the other mechanical properties, can be applied to similar test conditions (where T is the maximum temperature inside the oven, in °C):

$$E_c^T / E_c^{20}(T) = 1.035 - 2.21 \times 10^{-3} \times T + 1.36 \times \left(\frac{T}{1000}\right)^2 \quad (4)$$

Fig. 12 compares this experimental equation (Eq. (4)) with the reference values of EC2 — part 1.2 and DTU.

The evolution of all curves plotted follows a similar trend, but the experimental curve suggested in the present study based on the oven temperature (T_{oven}) is the less conservative one, with reduction factors that are always higher than those suggested in EC2 and DTU — this basically stems from the fact that the maximum temperatures achieved by test specimens are reasonably lower than those of the

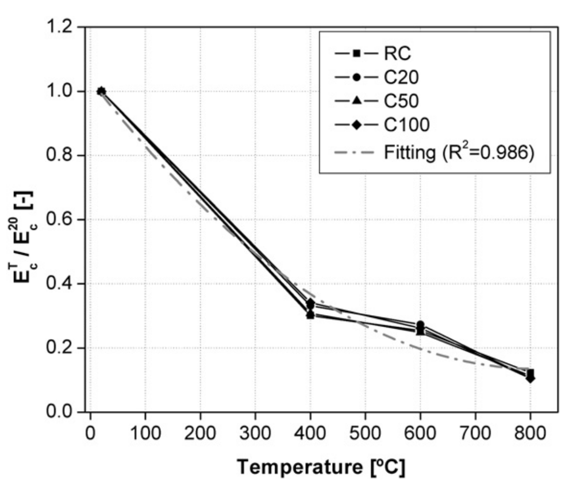

Fig. 11. Residual elasticity modulus as a function of temperature for concrete compositions with different RCCA replacement rates.

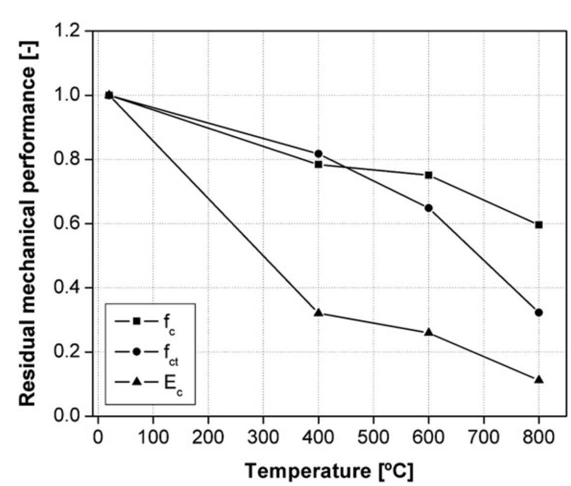

Fig. 13. Summary of the residual mechanical properties of the parameters evaluated.

oven for the different types of thermal exposure. If the maximum average temperatures reached by the test specimens ($T_{average}$) are considered, then a better agreement with the standard curves is achieved, particularly for higher temperatures.

5. Conclusions

Based on the results obtained in this study, on the post-fire mechanical performance of concrete made with recycled concrete coarse aggregates (CRCCA), the following main conclusions have been drawn:

1. In spite of the higher porosity and the different thermal properties of the matrix–aggregate interface of CRCCA when compared to reference concrete (RC), the incorporation of recycled concrete coarse aggregates (RCCA) does not influence the thermal response of the material.
2. The variation of the residual mechanical properties under study (compressive strength, splitting tensile strength and elasticity modulus) presented by CRCCA following exposure to high temperatures is roughly similar to that exhibited by RC. Therefore, no relationship seems to exist between the degradation of residual mechanical properties studied and the replacement rate of natural coarse aggregates (NCA) by RCCA for the different exposure temperatures analysed.
3. Accordingly, in terms of post-fire residual mechanical properties there are no limitations to the structural use of CRCCA when compared with conventional concrete.
4. As for conventional concrete the mechanical properties of concrete with increasing replacement rates of NCA by RCCA are significantly reduced after exposure to high temperatures (Fig. 13) — the maximum reductions of residual performance occurred for exposure temperature of 800 °C: 12.2% for compressive strength (in concrete composition C20), 36.1% for splitting tensile strength (C20), and 35.1% for elasticity modulus (C100).
5. Reduction factors were suggested to estimate the residual mechanical properties of CRCCA as a function of the exposure temperature, which are applicable for test conditions similar to those used in the present study.

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