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Materials and Structures **38** (April 2005) 335-342

Mechanical behaviour of thermally damaged high-strength steel fibre reinforced concrete

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Received: 5 January 2004; accepted: 14 June 2004

ABSTRACT

The incorporation of steel fibres can compensate the inherently brittle behaviour of high strength concrete. This paper studies the residual mechanical behaviour of thermally damaged high strength steel Fibre Reinforced Concrete (FRC). The type and content of fibres were included as variables, a mortar and a normal strength fibre concrete were also tested. Two exposure conditions were selected, 1 hour at 500°C and 24 hours at 150°C. FRC follow similar residual compressive behaviour as the plain concrete, but the presence of fibres lead to slight increase in strength and in the stress at which cracks initiate. Flexural tests on notched beams were performed (RILEM TC 162-TDF recommendation). It was found that the shape of the load-deflection curves in FRC exposed to 150°C was similar to the undamaged concrete. The reductions in flexural strength were lower in FRC than in plain concrete, and the equivalent post-peak strength was less affected than first-crack strength, showing the effect of fibre reinforcement. For the most severe exposure condition the degradation of the material is reflected by an increased non-linearity, nevertheless some FRC still exhibited a strengthening type behaviour and kept an almost constant load capacity during the post-peak.

RÉSUMÉ

L'incorporation des fibres d'acier dans le béton peut compenser le comportement fragile du béton de haute résistance. Est étudié dans ce travail le comportement mécanique résiduel du béton de haute résistance renforcé de fibre d'acier (BRF) endommagé par la température. On a considéré comme variables le type et le dosage de fibres; et on a aussi étudié un mortier et un béton de résistance normale. On a adopté deux conditions d'exposition à haute température, 1 heure à 500°C et 24 heures à 150°C. Les bétons renforcés de fibres d'acier (BRF) présentent un comportement résiduel en compression identique à celui du béton normal, mais la présence de fibres produit un léger accroissement de la résistance à la compression et de la contrainte où la formation de fissures commence. On a réalisé des essais de traction en flexion dans des éprouvettes prismatiques entaillées (recommandation de la commission technique RILEM TC 162-TDF). Les courbes charge-déformation du BRF exposé à 150°C présentent la même forme que celles du béton de référence. Par rapport au béton normal, les BRF ont une réduction inférieure de la résistance en flexion ; de plus, la résistance équivalente après pic a été moins affectée que la résistance des premières fissures, mettant en évidence l'effet de la présence des fibres. On a trouvé un accroissement de la non-linéarité dans les échantillons plus endommagés, alors que les BRF présentaient toujours un comportement de renforcement et maintenaient une capacité de charge pratiquement constante pendant l'après-pic.

1. INTRODUCTION

Fibre Reinforced Concrete (FRC) has been studied over the past three decades in terms of the improved crack control. This is of further importance when fibres are incorporated in an inherently brittle material such as high strength concrete (HSC), the use of which has increased progressively not only due to the higher load carrying capacity but also due to the improvement in durability and service life. The higher load carrying capacity of HSC is normally accompanied by more brittle behaviour, which can be compensated in a rational manner through the incorporation of steel fibres [1].

Many studies on the degradation of concrete when it is exposed to high temperatures have been reported. Concrete structures may be exposed to high temperatures, by accidental causes or by the characteristics of the structural application. As a consequence, concrete undergoes changes that may result, in many cases, in extensive cracking. In this sense, it is interesting to study the contribution of fibres to control crack formation and propagation. The effect of steel or polypropylene fibre reinforcement on mitigation of explosive

doi:10.1617/14175

spalling or on the reduction of the deterioration of normal and high strength concrete exposed to high temperatures over than 600°C has been recently studied [2, 3].

Many researchers have verified the degradation of concrete microstructure due to the exposure to high temperatures, and its effects on the behaviour of concrete, the changes being a function of the heating and cooling cycle [4-9]. The alteration of concrete structure due to the exposure to high temperatures affects the failure mechanism, which is reflected in the shape of the stress-strain curves [7]. Recently the authors [10] have studied the residual mechanical properties of plain concrete in tension. It was found that the degradation of the structure notably affects crack propagation and that this is reflected in the shape (ascending and softening) of the load-deflection curves. While damage produces a drop in strength, the reduction of the fracture energy is less important, due to the greater branching in crack propagation. The residual tensile strength appears to be more affected by the cracking than the compressive strength.

Regarding the behaviour under compressive loading, it was found that in damaged concrete the growth and propagation of matrix cracks start earlier and there is an important decrease in the period of stable matrix crack growth as the degradation is more severe [11]. In the mentioned study the concepts of initiation stress (defined as the stress at which there is a clear increase in lateral / axial strain ratio, it is associated with the beginning of cracking through the matrix) and critical stress (stress corresponding to the minimum of volumetric strain curve, it is associated with a pronounced and unstable crack propagation) were applied.

It was observed that microcracking process affects the stiffness much more than compressive strength [12] even for temperatures as low as 150°C, which usually do not significantly affect the compressive strength. At these temperatures other properties like water penetration under pressure and the coefficient of permeability of concrete increased in a very significant way; the cover concrete was the most affected [11]. While the residual compressive strength of HSC increased up to 200-300°C and after that decreased, the residual modulus of elasticity continuously decreased. Stress-strain or load-deformation relationships for concrete exposed at high temperatures are not widely reported, although these relationships are necessary to develop constitutive models for HSC [13].

This work was carried out with the aim of analysing the contribution of steel fibres to the failure mechanism of damaged concrete. This paper discusses several aspects of the residual behaviour under compression and flexure of steel fibre reinforced concretes exposed to temperatures up to 500°C. Normal and high strength concretes reinforced with different contents and types of steel fibres are included. Temperature was adopted as the damage inducing tool, and the dynamic modulus of elasticity and the ultrasonic pulse velocity were used to asses the degree of damage.

Table 1 - Mixture proportions

Series		A				B		
Identification		A-0	A-40H	A-80H	A-40L	B-M	B-NSC	B-HSC
fibres	type	none	H	H	L	L	L	L
	aspect ratio		80	80	80	80	80	80
	length (mm)		35	35	60	60	60	60
	content (kg/m^3)	0	40	80	40	50	50	50
slump (mm)		170	140	130	150	-	30	50
water/cement ratio		0.35				0.48	0.44	0.33
cement		a				b	b	A
silica fume/cement		0.10				none	none	0.05
agg. max. size (mm)		16				4	19	19

2. EXPERIMENTAL

FRC were prepared with two different types of hooked-ended collated steel fibres with aspect ratio 80: L (low carbon, tensile strength >1100 MPa and 60 mm length) and H (high carbon, tensile strength >2300 MPa and 35 mm length) (see Table 1). Two concrete series were done. Concretes of Series A had the same base materials proportions with varying the type and content of steel fibres. They were prepared with high early strength cement (a), silica fume, and naphthalene based superplastizicer. Series B includes a mortar and a normal strength concrete prepared with normal cement (b) and also a high strength concrete. Siliceous river sand and granitic crushed stone were used. Major details regarding materials and mixture proportions are given in [14]. Cylinders of 150 x 300 mm and prisms of 75 x 105 x 430 mm were cast for compression and flexure tests respectively.

Series A compares the effect of content and type of steel fibres in high strength concrete, while Series B, compares for the same type and content of fibre reinforcement the influence of strength level (normal vs. high strength) and the presence of coarse aggregates (mortar vs. concrete).

To attain uniform curing conditions and to avoid the superposition of other factors (different hydration rates, moisture conditions, etc.) on the effect of high temperatures, all specimens were cured in a fog room (23°C, 95% RH) during 60 days and then they were stored in laboratory at ambient conditions. At the ages of 4 (Series A) and 7 months (Series B) specimens were heated in an electric oven, and kept at maximum temperature, 150 or 500°C, for 24 or 1 hour respectively. Heating rate was 100°C /h. After heat treatment the specimens were kept 30 days in the laboratory and tested.

Non destructive tests are useful tools for indicating the grade of damage produced by high temperature in concrete, and they make possible an estimation of the degradation in specimens of different shape and size [8]. To estimate the changes produced during the exposure to high temperature, loss of weight, variations in ultrasonic pulse velocity (UPV) and dynamic modulus of elasticity were measured. The resonant frequency method appeared to be more sensitive than the ultrasonic pulse velocity method to detect damage in elements exposed at high temperatures, but the later can be applied in situ to evaluate damaged structures [12].

Uniaxial compression tests were performed on cylinders of 150 x 300 mm. Three loading-unloading cycles, up to 40% of the maximum stress, were applied to determine the modulus of elasticity and Poisson's ratio (ASTM C 469), after which the load was increased monotonically up to failure. The axial and lateral deformations were monitored with LVDTs. The development of the failure process under compressive loads can be followed by means of the analysis of the stress - strain curves. Based on them, the initiation and critical stresses can be obtained, representing the starting of matrix crack growth and the onset of unstable propagation of cracks in the matrix of concrete. When fibres are incorporated in concrete, these stresses tend to increase. The fibres delay the starting of the growth of cracks at the matrix and extend the period of crack propagation, leading to a higher failure stress [1].

The flexural behaviour was analysed following the general guidelines of a RILEM TC 162-TDF recommendation [15]. More recently a final recommendation was published [16]. It adopts the centre-point loading arrangement in tests of notched beams, measuring deflections and, as an option, crack mouth opening displacement (CMOD). Based on the load-displacement curves, the limit of proportionality and two equivalent flexure tensile strengths which identify the behaviour of the material up to a selected deflection are calculated. An offset of 0.05 on beams 150 high and 450 mm span was proposed. The equivalent tensile strength (f_{eq}) represents an average post-peak strength over a range of CMOD (w). The values of f_{eq} can be used in the drop-constant models of the tensile constitutive stress-crack opening, where the s-w curve is modelled by a vertical drop from the tensile strength and a constant residual strength [17].

The RILEM recommendation, in addition to the equivalent strength, includes the concept of residual strength, which indicates the post-peak strength at a definite deflection or CMOD. This paper only includes equivalent strengths as the term "residual" is used to identify the properties of concrete after heating and cooling down, as usually done when referred to high temperature exposition.

In these experiences prisms of 75 x 105 x 430 mm were tested using centre-point loading configuration over a span of 350 mm. Each beam was turned 90° from the casting surface and then sawn at mid-span. The depth of the notch was 20 mm. In all cases the notch was made two days before the tests were performed. Smaller beams were used, adopting an offset of 0.035 mm (instead of 0.05), identifying as δ_{FU} the deflection at the limit of proportionality F_u (maximum load in the interval). The equivalent tensile strengths $f_{eq,2}$ and $f_{eq,3}$, calculated from the mean load estimated from the area below the curve until deflections δ_2 (calculated as $\delta_2 = \delta_{FU} + 0.455$mm) and δ_3 (calculated as $\delta_3 = \delta_{FU} + 1.855$mm) were obtained. The tests were controlled by the average of the central deflection using an initial rate of 0.01 mm/min which was increased after the peak-load. The central deflection was measured by LVDT mounted at mid-span on two independent rigid rods that were fixed to each side of the beam, at mid-height, above its supports (with pins that permit rotation perpendicular to the beam axis). The sensors measure the displacement of the bottom (tensile face) of the beam with respect to the rod. The rollers at the supports and loading points, as well as the rest of the loading setup, were designed to eliminate torsional effects due to imperfections of the moulds and non-symmetric deformations during loading (Fig. 1). The CMOD was also measured with a clip gage. Load – deflection and load – CMOD curves of sound and damaged fibre-reinforced concrete were compared.

3. RESULTS AND DISCUSSION

The residual properties of concrete exposed to 150 and 500°C are identified with as *150 and *500 respectively. Reference concretes are identified as Control (undamaged, 20°C).

NDT methods were used as a way to evaluate the internal structure of the material. Table 2 presents the residual results of ultrasonic pulse velocity (UPV), dynamic

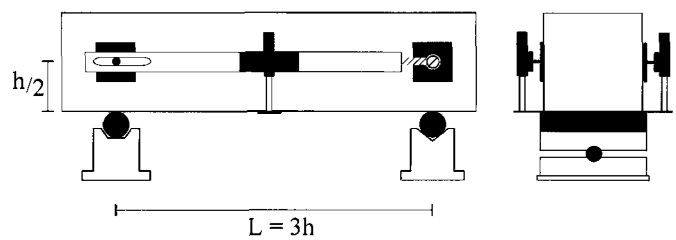


Fig. 1 - Deflection measurement device used in flexural tests.

Table 2 - Results of Non-Destructive Tests and Weight loss (after heat treatment)

Series		A				B		
Identification		A-0	A-40H	A-80H	A-40L	B-M	B-NSC	B-HSC
Prisms								
E_d (GPa)	Control	41.2	38.2	41.3	41.4	38.4	47.2	51.6
	*150	35.1	31.2	35.7	33.0	31.3	38.5	41.6
	*500	21.7	17.0	13.0	19.7	17.0	18.3	16.5
Weight loss%	*150	1.9	3.8	-	3.9	5.9	3.4	3.1
	*500	-	5.7	5.8	5.8	9.1	6.4	6.8
Cylinders								
E_d (GPa)	Control	37.7	34.9	38.3	37.2	33.7	42.3	45.5
	*150	30.7	30.4	32.8	29.7	25.9	32.2	37.0
	*500	10.8	9.7	10.4	9.5	14.4	17.1	17.4
UPV (km/s)	Control	4.27	4.22	4.27	4.29	4.14	4.47	4.55
	*150	4.10	3.95	4.01	3.97	3.69	4.07	4.19
	*500	1.92	1.41	1.41	1.19	2.84	2.99	2.92
Weight Loss%	*150	4.3	3.5	3.4	3.9	10	-	4.9
	*500	5.9	6.4	6.7	6.9	8	-	5.4
RELATIVE VALUES, Control = 100								
Prisms								
E_d (GPa)	*150	85	82	86	80	82	82	81
	*500	53	45	31	48	44	39	32
Cylinders								
E_d (GPa)	*150	81	87	86	80	77	76	81
	*500	29	28	27	26	43	40	38
UPV (km/s)	*150	96	94	94	93	89	91	92
	*500	45	33	33	28	69	67	64

modulus of elasticity (E_d) and weight loss after heating, measured before flexure and compressive tests. There are large reductions in pulse velocity in concretes exposed to 500°C. The dynamic modulus of elasticity changed in a similar way as UPV, the residual values being lower than 80% at 150°C and 50% at 500°C. The decreases in NDT parameters, the same as loss of weight, were greater in fiber reinforced concrete than in plain concrete (A-40H, A-80H and A-40L vs. A-0). This major alteration in the internal structure of FRC can be explained assuming the presence of additional defects at the fibre-matrix interfaces. Nevertheless, as will be discussed, the residual mechanical performance was better in FRC.

Table 3.a - Results of compression tests. Series A

Identification		A-0	A-40H	A-80H	A-40L	Relative values			
						A-0	A-40H	A-80H	A-40L
f'c (MPa)	Control	59.3	59.4	75.4	63.1	100	100	100	100
	*150	55.3	57.3	70.0	62.5	93	96	93	99
	*500	35.8	36.3	47.7	38.8	60	61	63	61
E (GPa)	Control	33.3	30.9	33.8	31.9	100	100	100	100
	*150	27.4	26.5	29.3	27.8	82	86	87	87
	*500	13.3	11.0	12.0	10.8	40	36	36	34
μ	Control	0.17	0.17	0.17	0.18	100	100	100	100
	*150	0.15	0.15	0.15	0.15	88	88	88	83
	*500	0.13	0.16	0.16	0.15	74	94	94	83

Table 3.b - Results of compression tests. Series B

Identification		B-M	B-NSC	B-HSC	Relative values		
					B-M	B-NSC	B-HSC
f'c (MPa)	Control	40.2	52.1	71.6	100	100	100
	*150	39.9	51.1	77.0	99	98	108
	*500	31.0	40.4	53.6	77	78	75
E (GPa)	Control	32.1	34.7	38.4	100	100	100
	*150	27.5	34.2	34.6	86	99	90
	*500	15.7	18.2	22.6	49	52	59
μ	Control	0.19	0.17	0.17	100	100	100
	*150	0.18	0.18	0.17	95	106	100
	*500	0.18	0.17	0.16	95	100	94

Table 4 - Characteristic values of the failure mechanism in uniaxial compression

Series		A				B		
Identification		A-0	A-40H	A-80H	A-40L	B-M	B-NSC	B-HSC
f_{crit} (MPa)	Control	53.9	51.8	65.1	48.8	38.7	47.5	65.3
	*150	47.6	48.1	56.0	48.2	37.1	41.4	64.7
	*500	22.0	20.0	24.6	19.1	22.8	27.7	37.5
f_{crit} (% f'c)	Control	91	87	86	77	96	91	91
	*150	86	84	80	77	93	81	84
	*500	62	55	51	49	74	69	70
f_{init} (MPa)	Control	43.0	43.0	49.3	40.0	34.6	38.9	57.2
	*150	39.7	40.8	41.5	38.6	28.3	32.4	55.2
	*500	20.0	19.5	22.4	18.7	20.5	26.1	36.3
f_{init} (% f'c)	Control	73	72	65	63	86	75	80
	*150	72	71	59	62	71	63	72
	*500	56	54	47	48	66	65	68

The measured values of the residual E_d on cylinders and prisms were very similar, indicating that the level of damage of the material is comparable for both types of specimens.

3.1 Behaviour under compressive loading

Under uniaxial compressive loading, extensive cracking is produced in concrete during the final stage of the pre-peak regime. When fibres are present in the concrete, the propagation of such cracking is expected to be restrained, which would result in the alteration of the failure behaviour [18, 19]. In this section, the pre-peak response is studied through an analysis of the evolution of the axial and transversal strains in concrete specimens. The results obtained are summarised in Table 3.

An increase in the compressive strength of near 20% is observed due to the presence of fibres, which is in accordance with data in the literature. As was expected, the elastic properties (elastic modulus, Poisson's ratio) are practically unaffected (Series A, Table 3.a).

Previous work on the behaviour of plain concrete affected by high temperature [10-12] has shown that the exposure to 500°C caused significant damage to the internal structure. Tables 3.a and 3.b show that the residual strengths of concrete exposed to 150°C are similar to the Control concrete, both for plain and FRC, the differences being lower than 10%. When concretes were exposed to 500°C the residual strengths decreased significantly ranging between 60 and 78%. Residual compressive strengths of FRC are very similar or slightly higher than the residual strength of plain concrete. The residual compressive strength was not significantly affected by the type of fibre, the fibre content, or the concrete strength level.

The same occurred in plain concrete: the reductions in the modulus of elasticity are higher than those in compressive strength (Table 3). Residual values of the modulus of elasticity range between 34 to 59% in concrete exposed at 500°C. Finally, while the residual Poisson's ratios clearly decrease in plain concrete [11] it was less affected by high temperatures in FRC. It can also be seen that residual values, as percentage of Control, of the modulus of elasticity and the Poisson's ratio are always lower in fibre reinforced mortar than in reinforced concrete (B-M vs B-NSC) while residual values of compressive strength are almost the same. It is clear that the fissures and microcracks affect stiffness and modify the relationship between lateral and axial strains. Some of these facts can be explained from the stress – strain curves analysis.

To analyze failure mechanism of damaged FRC, Fig. 2 shows typical stress – strain curves of Control concrete and concretes exposed at 150 and 500°C. It includes axial, lateral and volumetric strains of each concrete. The presence of defects modifies the shape of the curves, both in plain and fiber reinforced mixtures.

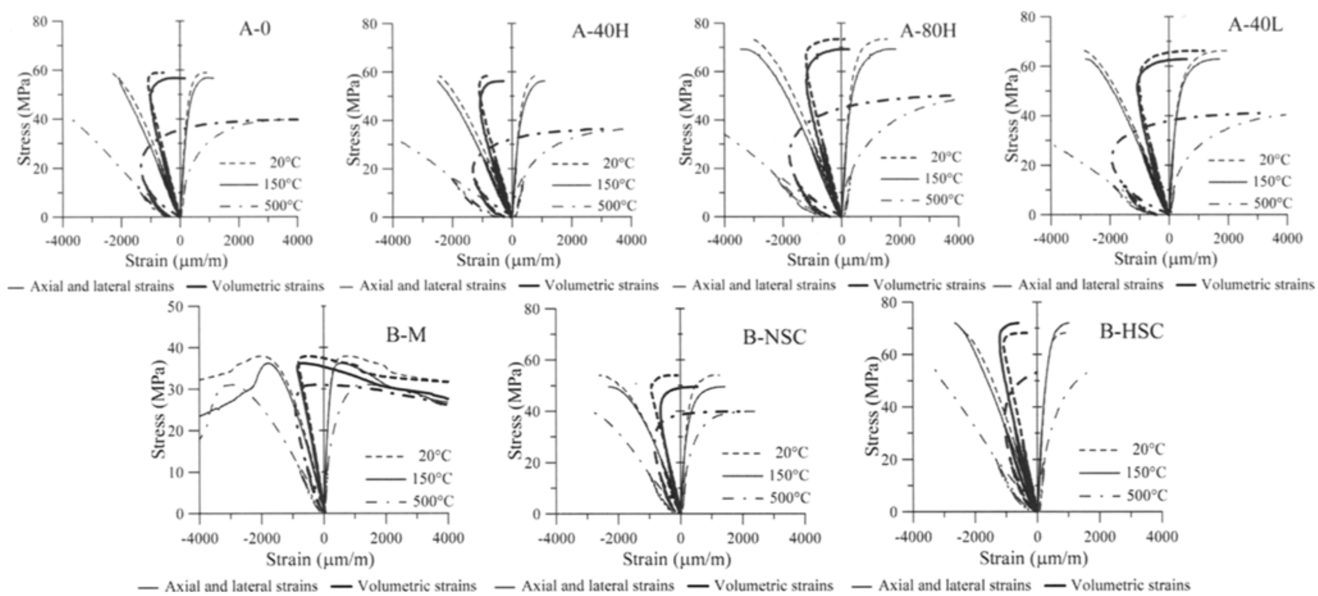


Fig. 2 - Stress vs. axial, lateral and volumetric strain curves in compression.

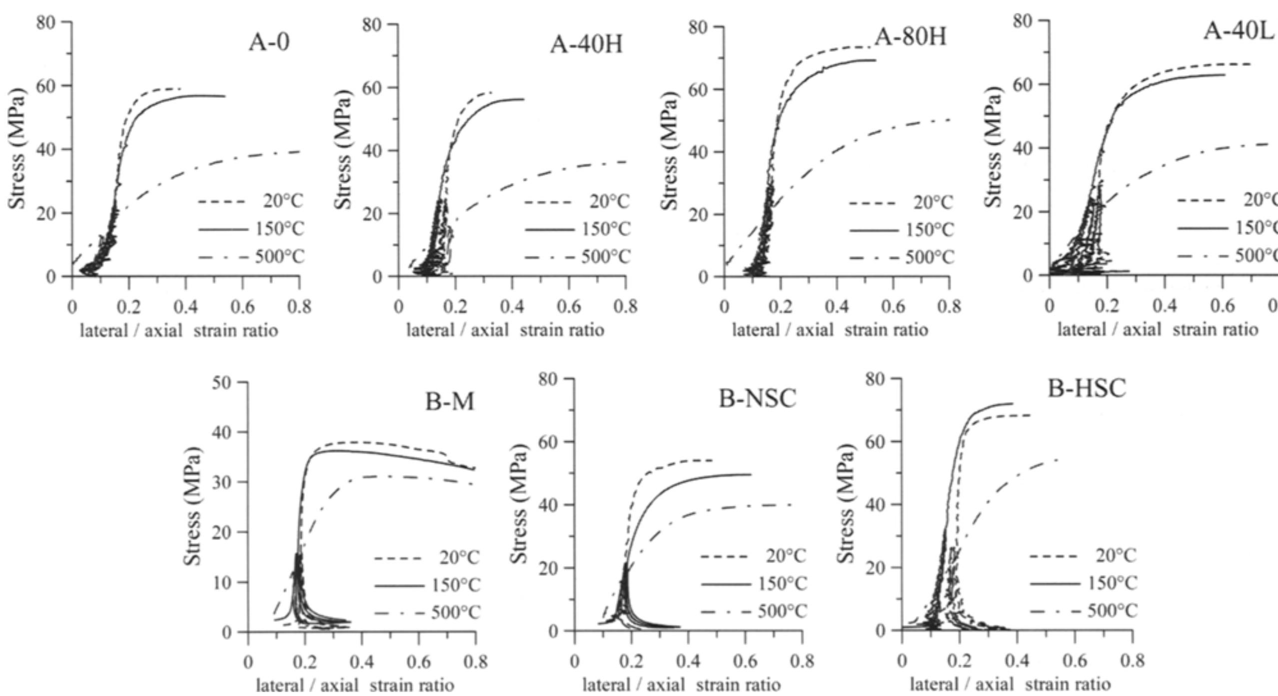


Fig. 3 - Variation of the lateral / axial strains ratio with the applied stress.

Typical stress vs. lateral / axial strain ratios of specimens are given in Fig. 3. It can be seen that as cracking increases the lateral to axial strain ratio tends to increase at lower stresses and more rapidly, for every type and content of fibers.

As was mentioned before, the fibres increase the load-carrying capacity in post-cracking regime when compared with plain concrete [1]. This effect can be analysed quantitatively by considering the values of critical stress, f_{crit}, which corresponds to the axial stress at the peak volumetric strain. The critical stress can be associated with the onset of unstable crack propagation in the matrix, and has been used to differentiate failure mechanisms in HSC [20, 21]. From the data in Table 4 (Series A), it is verified that the $f_{crit}/f'c$-value tends to decrease when fibres are present. The stress versus lateral strain response is more nonlinear in the FRC. This implies that the fibres restrain the cracking, which results in a more ductile pre-peak response and higher maximum stress in the high strength concrete. This behaviour was previously observed in normal strength concrete [19].

Fig. 4 presents the initiation stress (f_{init}), the critical stress (f_{crit}), and the compressive strength (f'c) of each concrete for the different exposure temperatures (Table 4). When concretes are exposed to 500°C, f_{init} and f_{crit} decrease, with the decrease in f_{crit} being more significant. This means that, due to the cracking generated by high temperature, the growth and propagation of matrix cracks started earlier. The period of

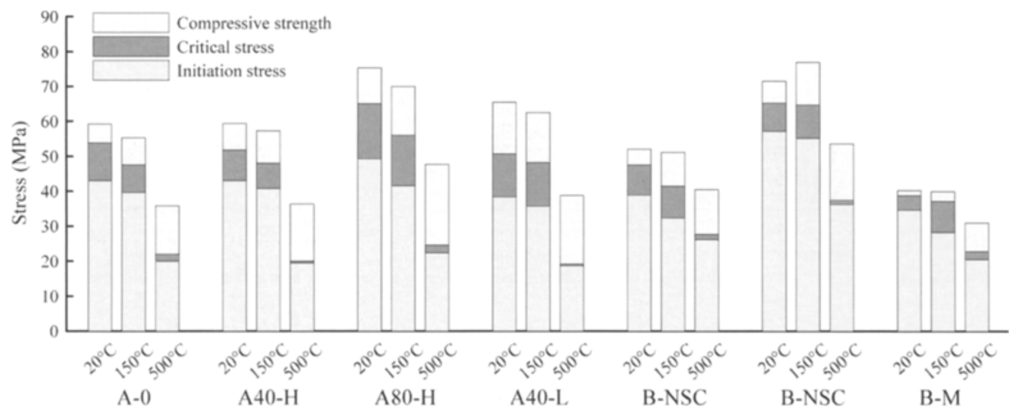


Fig. 4 - Initiation stress (f_{init}), critical stress (f_{crit}) and compressive strength (f'c).

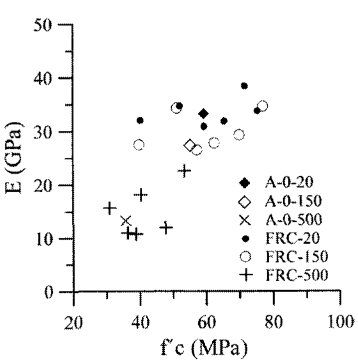


Fig. 5.a - Relationship between compressive strength and static elastic modulus.

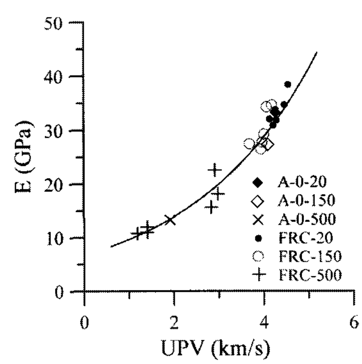


Fig. 5.b - Relationship between ultrasonic pulse velocity and static elastic modulus.

stable crack propagation in the matrix is reduced (given by the difference between f_{crit} – f_{init}) and, although the period of unstable crack growth is extended (given by the difference between f'c – f_{crit}), the capability of controlling crack propagation decreases, leading to premature failure [11]. This behaviour was verified both in plain and in all FRC. When the exposure temperature was 150°C, the residual stresses show a small decrease following the same tendencies.

Comparing different FRC, it can be seen that the values of f_{init}, f_{crit} and f'c tend to increase as fibre content increases and when High-carbon steel fibres were used (Series A). As was expected critical stresses are lower in concrete than in mortar, and there are no significant differences (as a % of f'c) when Normal-strength and High-strength fibre concretes are compared (Series B).

Regarding the use of NDT, it was found that in thermally damaged plain concrete ultrasonic pulse velocity (UPV) strongly differs from the usual values measured in sound concrete. The UPV method can not be used to estimate strength, nevertheless it can be used to identify damaged zones [8, 12]. At the same time, there is not a good correlation between the modulus of elasticity and the compressive strength in thermally damaged concrete. It was also verified that concretes prepared with different materials and mixture proportions and exposed to different grades of alteration (temperature, time of exposure, type of cooling) show a very good relationship between the static modulus of elasticity and the UPV. Then, the ultrasonic method appears as an interesting tool to evaluate the residual stiffness in concrete structures affected by high temperatures [12]. Figs. 5.a and 5.b present the relationships between f'c vs. E and UPV vs. E, corresponding to Series A and B. It can be seen that this approach continues to be valid when FRCs are considered.

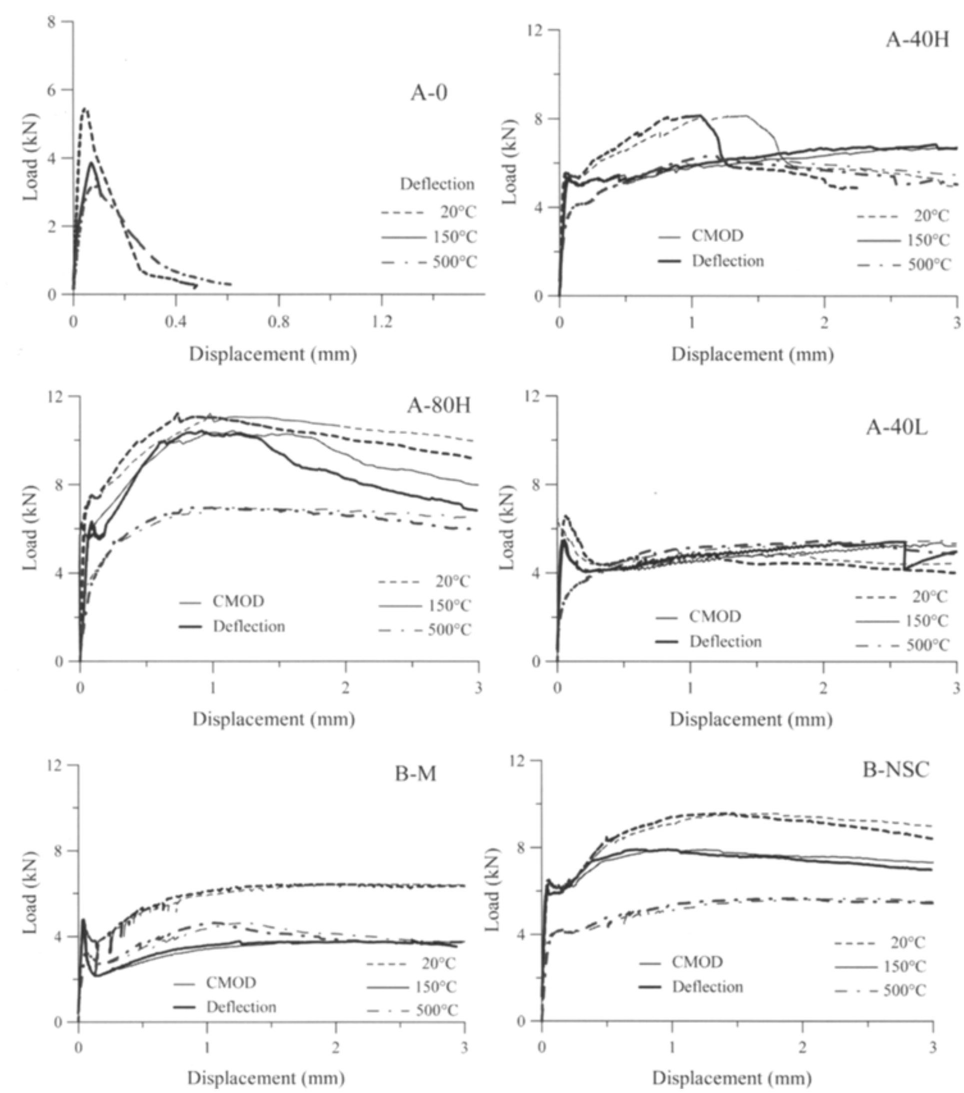
3.2 Mechanical behaviour of notched beams

Fig. 6 presents typical load - deflection and the load - CMOD curves corresponding to sound and damaged FRC of Series A and B. Plain concrete A-0 and fibre mortar B-M are also included. It can be seen that the shape of the curves in specimens exposed to 150°C is similar to the undamaged concrete (Control, 20°C). The main differences are especially notable at first-crack level, indicating that the fibres are particularly effective after matrix cracks. After first crack, there is an increase in inelastic deformation, then the action of fibres increases the concrete loading capacity, depending on fibre type and content.

For the most severe condition (500°C) the degradation of the material is reflected by an increased non-linearity. Nevertheless the FRC still exhibit a strengthening type behaviour and they keep an almost constant load capacity during the post-peak, even exceeding CMOD values of 3 mm. In most cases the maximum load is achieved at deflections as high as 1 mm, that is 10 times higher than plain concrete.

Table 5 shows the results obtained in flexural tests. The Modulus of rupture (MOR) calculated using the maximum load, and the other parameters that characterise FRC behaviour, as the first crack strength ($f_{fct,fl}$) and the equivalent tensile strengths ($f_{eq,2}$ and $f_{eq,3}$) are presented. The relative values referred to the Control are also calculated. It must be noted that concrete A-80H, prepared with high-carbon steel fibres, achieved a MOR of 14.5 MPa and equivalent strengths ($f_{eq,2}$ and $f_{eq,3}$) over 11 MPa.

The mechanical properties of thermally damaged FRC show the same tendencies as plain concrete. As it was expected, the matrix degradation leads to a reduction of the residual mechanical properties of concrete. However, from Series A it can be seen that the incorporation of fibres leads to an increase in the residual strength. The reductions in flexural strength are lower in FRC than in plain concrete, note that A-0 achieves a residual peak-load capacity of 68% while in FRC are in the order of 80% (Table 5). The residual flexural strength of concrete A-0 exposed at 500°C was 3.5 MPa (53% of the sound concrete) while in A-40H and A-80H it increases up to 7 and 9.2 MPa (70 and 63% of



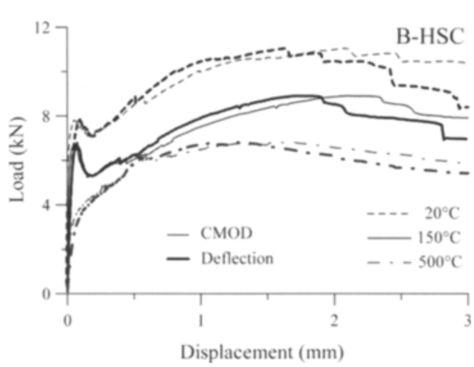


Fig. 6 - Load - deflection and load - CMOD curves corresponding to sound and damaged concretes.

the sound concrete). These FRC present values of residual equivalent strengths over 5 and 7 MPa respectively, for the most severe exposure condition.

Concrete prepared with high-carbon steel fibres tends to show rather higher residual values. Some differences can be attributed to the number of fibres as the low-carbon type has the same aspect ratio but greater length. Nevertheless, it is interesting to note that in concrete A-40L the post-peak loading capacity was very similar between Control, 150°C and 500°C groups, the degradation specially affects the first crack strength.

Series B indicates that the relative values of the residual parameters in FRC tend to increase as strength increases (B-NSC vs. B-HSC). Series B also shows that the relative values were higher in fibre reinforced mortar than in FRC.

4. CONCLUSIONS

This paper studied the residual mechanical behaviour of thermally damaged high strength steel fibre reinforced concrete. Two exposure conditions were selected, 1 hour at 500°C and 24 hours at 150°C. The following conclusions can be drawn:

- FRC exposed to high temperature follow a similar residual compressive behaviour as the plain concrete. Nevertheless the presence of the steel fibres leads to a slight increase in the strength and the stress values at which cracks initiate, and they tend to extend, in relative terms, the periods of crack propagation, especially when high contents of fibres are used.
- In thermally damaged FRC, the same trends as in plain concrete occur, the reductions in the modulus of elasticity are higher than in the compressive strengths. In a similar way, the residual compressive strength tends to be higher than the residual modulus of rupture, for the lower level of damage (150°C).
- The shape of the load deflection curves in FRC exposed to 150°C is similar to the undamaged concrete. After first-crack there is an increase in inelastic deformation. The reductions in flexural strength were lower in FRC than in plain concrete, and the equivalent post-peak strength was less affected than first-crack strength, showing the effect of fibre reinforcement.
- For the most severe condition of exposure (500°C), the residual flexural strength and the residual equivalent tensile strengths ranged between 60 to 70% of the sound concrete. The degradation of the material is reflected by an increased non-linearity. Nevertheless some FRC still exhibited a strengthening type behaviour and kept an almost constant load capacity during the post-peak, even exceeding CMOD values of 3 mm.
- Although there were not marked differences observed between the responses of the studied FRC, it was noted that: (i) the flexural residual parameters tend to increase as fibre content and concrete strength increase, (ii) concretes with high-carbon steel fibres tend to show rather higher values than

Table 5.a - Results of flexural tests. Series A

Identification		A-0	A-40H	A-80H	A-40L	Relative values			
						A-0	A-40H	A-80H	A-40L
MOR (MPa)	Control	6.6	10.0	14.5	7.8	100	100	100	100
	*150	4.5	8.4	12.1	7.1	68	84	83	92
	*500	3.5	7.0	9.2	5.8	53	70	63	74
$f_{fct,fl}$ (MPa)	Control		7.0	8.4	7.8		100	100	100
	*150		6.8	7.4	7.1		97	88	92
	*500		4.7	5.2	3.8		67	62	49
$f_{eq,2}$ (MPa)	Control		7.0	11.2	5.2		100	100	100
	*150		6.6	9.2	3.7		94	82	72
	*500		5.5	7.0	4.5		79	63	87
$f_{eq,3}$ (MPa)	Control		8.6	13.1	5.4		100	100	100
	*150		7.3	10.8	4.3		85	82	79
	*500		6.3	8.4	5.2		73	64	96

Table 5.b - Results of flexural tests. Series B

Identification		B-M	B-NSC	B-HSC	Relative values		
					B-M	B-NSC	B-HSC
MOR (MPa)	Control	7.3	10.5	12.0	100	100	100
	*150	5.7	7.5	10.1	78	71	84
	*500	5.1	5.9	7.9	70	56	66
$f_{fct,fl}$ (MPa)	Control	5.1	8.5	8.9	100	100	100
	*150	4.4	6.3	7.7	86	74	87
	*500	3.7	4.8	4.4	73	56	49
$f_{eq,2}$ (MPa)	Control	5.3	8.7	8.9	100	100	100
	*150	3.0	5.8	7.1	57	67	80
	*500	3.6	4.5	5.4	68	52	61
$f_{eq,3}$ (MPa)	Control	6.1	9.8	10.8	100	100	100
	*150	4.3	6.6	8.8	70	67	81
	*500	4.5	5.3	6.8	74	54	63

those prepared with low-carbon steel fibres, (iii) the relative residual values were higher in fibre reinforced mortar than in FRC.

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