

Crack control in concrete members reinforced by conventional rebars and steel fibers

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Abstract. Tension stiffening is still a matter of discussion within the scientific community. The ability of resisting tensile stresses by un-damaged concrete portions that spans in between cracks is significant and could be improved with adoption of tougher material as Fiber Reinforced Concrete (FRC). In addition, FRC may provide noticeable residual stresses at a crack location, linking the two adjacent faces due to the bridging effect provided by the fibers. Within this framework, this paper aims at further investigating the ability of Steel Fiber Reinforced Concrete (SFRC) in reducing the crack spacing and width by utilizing SFRCs with high post-cracking residual strengths.

1. Introduction

The employment of Fiber Reinforced Concrete (FRC) has gained considerable attention in recent years, as numerous researches have demonstrated its effectiveness in many structural applications. Moreover, a number of physical, semi-empirical and empirical models have been recently developed toward the formulation of appropriate design procedures that are useful for practitioners; especially for strain-softening materials. The recent inclusion of FRC in the fib Model Code 2010 [1], referred to as MC2010 herein, in national codes as well as the organization of FRC-devoted conferences [2, 3, 4] confirms this positive development.

FRC has been particularly used in structural elements when crack propagation control is of primary importance, e.g. in precast tunnel segments [5] or in beams where little or no shear reinforcement is provided [6]. In several of these applications, the reinforcement generally consists of a combination of conventional rebars and fibers [7].

FRC significantly improves the behavior at the Serviceability Limit State (SLS), with respect to crack and deflection control. It is well known that, after cracking, tensile stresses are induced in the concrete between cracks and hence they stiffen the response of a reinforced concrete member subjected to tensile stresses. This stiffening effect is referred to as “tension stiffening”. Several authors already studied this mechanism in traditional Reinforced Concrete (RC) elements [8, 9, 10]. In fibrous RC elements, the transfer of non-negligible residual stresses at crack locations provides an additional significant mechanism that influences the member response. The combination of these two mechanisms (tension stiffening and the post-cracking residual stresses provided by fibers at any crack,



referred to as “residual strength” or “tension softening” herein) results in a different crack pattern, characterized by a reduced crack spacing and crack width.

A number of research studies have been carried out so far on the tensile behavior of FRC members since late 90’s. Mitchell [11] presented one of the first studies; more recently, Bischoff [12, 13] performed monotonic and cyclic tests and included shrinkage effects in the analysis.

Within this framework, the present paper describes the main results obtained from 17 tests on tension-ties to the aim of evaluating crack formation and development in structures reinforced by SFRC having high post-cracking residual strength. Besides a description of the tests, particular attention was devoted to the influence of fibers in terms of crack formation and development: the crack initiation, the crack formation stage, the crack spacing and its progression will be evaluated. The results will be also compared against the formulation proposed by fib MC 2010 [1].

2. Experimental program

In previous research works carried out at the University of Brescia [14, 15] a broad experimental program was developed and about one-hundred uniaxial tension Reinforced Concrete (RC) and Steel Fiber Reinforced Concrete (SFRC) members containing one central steel rebar were tested. Several key-parameters were investigated such as element size (b), rebar diameter (ϕ), ϕ/ρ_{eff} ratio and SFRC post-cracking properties. These fibrous and non-fibrous members will be identified as RC and SFRC tensile ties, respectively. Note that the effective reinforcement ratio (ρ_{eff}) is the conventional reinforcing rebar area divided by the area of concrete in tension surrounding the reinforcement: in the presented samples, $\rho = \rho_{eff}$.

In order to further improve the knowledge on the cracking behavior of SFRC members, 17 SFRC tensile ties were cast within a collaborative program with the American University of Sharjah. The principal aim was to investigate the effects of high-performance SFRCs, presenting noticeable high post-cracking nominal residual strengths f_{Ri} , according to EN 14651 [16].

2.1. Uniaxial tension SFRC test specimen configurations

The geometry of tested specimen is shown in Figure 1. Five square cross sections were selected: 80, 120, 180 and 200 mm in size (b , Figure 1). Samples having 80 mm size are 950 mm long and have been tested by means of a hydraulic universal servo-controlled machine [15]. All the other specimens are 1500 mm long; holed steel plates were welded at both of the rebar ends. The latter were connected by pins in vertical position to the strong floor and to a steel reaction frame (for more details, see [15]).

Reinforcing bars having a diameter of 10, 20 and 30 mm (B450C steel, according to European standard EN 10080 [17]), corresponding to a reinforcement ratio (ρ) varying from 0.98% to 2.23%, were employed (Figure 1). The properties of the reinforcing bars used in the tie elements are reported in Table 1. The concrete cover was, in all cases, at least 2-3 times the bar diameter to prevent splitting phenomena during the tests [18, 19].

Specimens were reinforced by hooked-end-steel fibers having a length of 30 mm, a diameter of 0.50 mm (aspect ratio of 60) and tensile strength of 3100 MPa. The fiber content was 40 kg/m³ (fiber volume fraction, V_f is about 0.5%).

All samples were cast with the same concrete matrix designed to obtain a C35/45 concrete, according to Eurocode 2 [20]. Specifically, a cement content of 390 kg/m³; water-to-cement ratio of 0.46; sand content (0-4 mm) of 648 kg/m³; coarse aggregate content (4-20 mm) of 1202 kg/m³; and superplasticizer content of 3.3 l/m³.

Table 2 reports the mechanical properties of used concrete and the volume fraction of steel fibers. Each combination of fiber reinforcement, member dimension and steel reinforcement ratio defines a specific set of tests, whose repetitions and notations are listed in Table 3, in which the entire experimental program is reported in detail. Three specimens were in general tested for each material combination. In Table 3 the mean experimental crack spacing s_m is reported as well. The mean crack spacing of a single specimen was evaluated by measuring the distance between visible cracks on the

surface. Furthermore, the mean crack spacing of each set of samples (s_{rm}) was calculated as the mean value of the measured mean values of each single specimen.

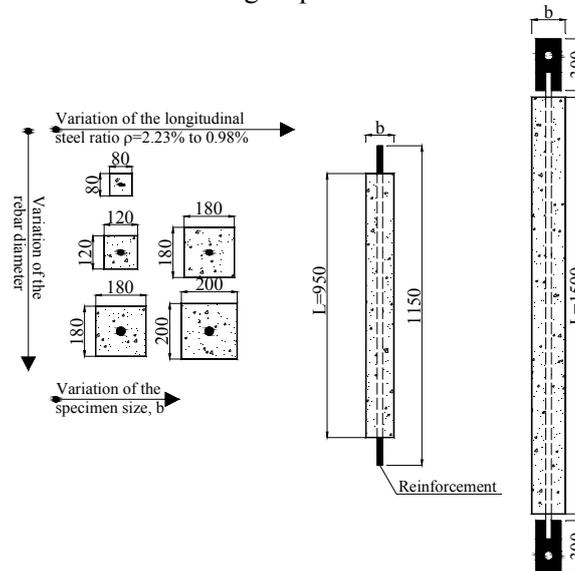


Figure 1. Geometry and reinforcement details of specimens.

Table 1. Properties of steel reinforcing bars.

Rebar	A_s [mm ²]	ϕ	E_s	f_y	ϵ_{sh}	f_{ult}
		[mm]	[GPa]	[MPa]	[x10 ⁻³]	[MPa]
$\phi 10$	77	10	193	511	-	720
$\phi 20$	305	20	192	548	14.72	633
$\phi 30$	711	30	207	510	11.52	621

Table 2. Mechanical properties of concrete and fiber content.

Batch ID	Days after casting	$f_{cm,cube}$	f_{cm}	f_{ctm}	E_c	Fibers, V_f
		[MPa]	[MPa]	[MPa]	[GPa]	[% - kg/m ³]
SFRC 0.5MH	149	53.1 (0.06)	44.1 (*)	3.27 (†)	35.2 (†)	0.50 - 40

Coefficient of variation (CV) was reported in round brackets.

* Calculated by assuming $f_{cm}=0.83 \cdot f_{cm,cube}$.

† Calculated as suggested by fib Model Code 2010 [1].

Table 3. Experimental program, specimen notation and mean crack spacing.

Rebar	Batch ID	b [mm]	Length, L [mm]	Reinf. Ratio (%)	Clear cover [mm]	Specimen ID	No. of Spec's	s_{rm} [mm]
$\phi 10$	0 Plain*	80	1000	1.25	35	N 80/10 - 0	3	144
	0.5MH					N 80/10 - 0.5MH	4	111
$\phi 20$	0 Plain*	120	1500	2.23	50	N 120/20 - 0	3	170
	0.5MH					N 120/20 - 0.5MH	3	127
$\phi 20$	0 Plain*	180	1500	0.98	80	N 180/20 - 0	3	358
	0.5MH					N 180/20 - 0.5MH	4	215
$\phi 30$	0 Plain*	180	1500	2.23	75	N 180/30 - 0	3	232
	0.5MH					N 180/30 - 0.5MH	3	172
$\phi 30$	0 Plain*	200	1500	1.80	85	N 200/30 - 0	3	310
	0.5MH					N 200/30 - 0.5MH	3	180

* Reference plain concrete specimens which were tested in a previous experimental campaign (2nd phase, [15]).

The main data of reference RC tensile ties were also reported in Table 3. In fact, RC samples tested in a previous broad experimental campaign [15] were assumed as reference for analyzing results obtained from sets of the tensile ties that are labeled as SFRC 0.5MH. It is worthwhile underlining that the mechanical properties of steel reinforcing conventional bars ($\phi 20$ and $\phi 30$) reported in Table 1 are similar to those exhibited by rebars adopted in the previously mentioned research work [15], making the comparison of results valid and consistent.

2.2. Material properties

A number of tests were conducted in order to determine the material properties. Standard tests on 150 mm cubes were carried out for the determination of the concrete compressive strength ($f_{cm,cube}$). The corresponding values with their coefficient of variations are reported in Table 2.

The cylindrical tensile strengths and Young modulus were determined according to MC2010 [1], as reported in Table 2. Similarly, the cylindrical compressive strengths are reported as well. The latter were determined from the experimental cubic strengths by adopting the relationship $f_{cm}=0.83 \cdot f_{cm,cube}$.

In addition, among the many standards that are available for the material characterization, all SFRC were characterized according to the European Standard EN 14651 [16] and MC2010 [1], which require that three point bending tests (3PBT) to be performed on small notched beams (150-150-550 mm). Experimental curves for the entire series SFRC 0.5MH, concerning the nominal stress vs. Crack Mouth Opening Displacement (CMOD) are depicted in Figure 2. The mean of the individual specimens' curves is also depicted. It can be noticed that the post-cracking flexural behavior of series SFRC 0.5MH is generally exhibiting the desirable high residual strength and toughness performance, as well be highlighted below.

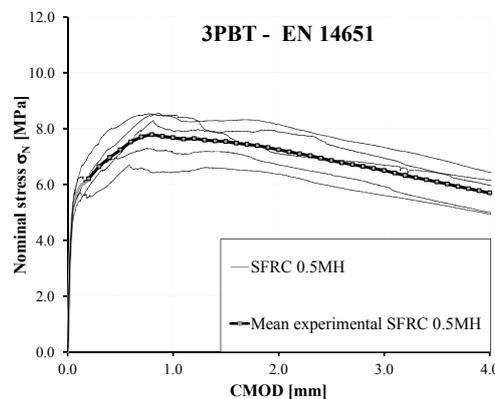


Figure 2. Experimental results of 3PBT SFRC notched beams according to EN 14651 [16].

Based on these curves, the residual strengths f_{Rj} (evaluated at 4 different CMOD values, i.e. 0.5, 1.5, 2.5 and 3.5 mm [16]), and the flexural tensile strength (limit of proportionality) f_L were calculated, as listed in Table 4 (mean values). Also, the corresponding coefficient of variations are reported in Table 4. It is worthwhile underlining that both series present noticeable high residual strengths (i.e. toughness). In a previous experimental campaign [15], it was proven that the higher the f_{R1m} (corresponding to CMOD equal to 0.5 mm) is, the smaller the mean crack spacing (s_{rm}) exhibited by tensile ties. In this regard, it can be observed that the series SFRC 0.5MH present a f_{R1m} of 7.24 MPa, which is much higher than the f_{R1m} shown by SFRCs tested in the aforementioned study [15].

Table 4. Fracture parameters of the SFRCs according to EN 14651 [16].

Batch ID	f_{Lm}	f_{R1m}	f_{R2m}	f_{R3m}	f_{R4m}
	[MPa]	[MPa]	[MPa]	[MPa]	[MPa]
SFRC 0.5MH	5.08 (0.04)	7.24 (0.09)	7.54 (0.09)	6.86 (0.11)	6.09 (0.12)

Coefficient of variation (CV) was reported in round brackets.

2.3. Set-up and instrumentation

Samples having a length of 950 mm were tested by means of a hydraulic universal servo-controlled (closed-loop) universal testing machine. The test was carried out with stroke-control, while monitoring the specimen behavior up to the onset of the rebar strain-hardening [15]. The deformation rate varied from 0.1 to 0.2 mm/min up to the rebar yield; then the rate was progressively increased up to 1 mm/min, the latter is beyond an average member strain of approximately 2%.

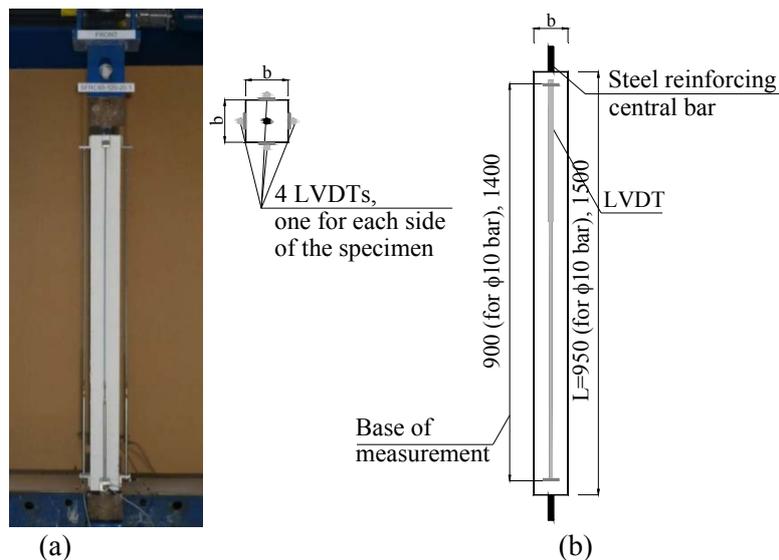


Figure 3. Typical configuration of the tensile tie during test (a) and instrumentation (b).

For the longer specimens ($b=120, 180, 200$ mm, Figure 1), tests were carried out by means of a steel reaction frame conveniently modified for the scope (for more details see [15]). Two previously machined steel plates, with bolt-holes, were welded at the ends of the specimen (Figure 1), which was connected by pins in vertical position to the strong floor and to the steel frame. The upper end of the specimen was connected to an electromechanical screw jack, with a maximum capacity of 1500kN. Note that the plates and the corresponding welding did not change the behavior of the bare bar as preliminary verified through uniaxial tests on the welded joints. Tests were carried out under stroke control and by assuming the same load-procedure of the first phase.

All specimens were stored in a fog room (R.H. > 95%; $T=20 \pm 2^\circ\text{C}$) until 2 or 3 days before testing; then they were air dried in the laboratory. Shrinkage strains were measured by means of free shrinkage prisms: since the measured strains were negligible, no-shrinkage offset strains were herein applied.

A typical instrumented specimen is shown in Figure 3a: four Linear Variable Differential Transformers (LVDTs, one for each side), were employed to measure the mean deformation of the specimen over a length ranging from 900 mm to 1400 mm, the former in the 950 mm long specimens, the latter for the remaining experiments (Figure 3b).

3. Results and discussion

In this Section, the behavior of SFRC tensile ties will be deeply investigated in terms of crack development by analyzing the evolution of mean crack spacing and average member strain with the progressive increment of the applied tensile forces.

3.1. Typical tensile tie behaviour: benefits of fiber reinforcement in combination with rebars

The diagrams reported in Figure 4a and in Figure 4b provide typical axial load vs. average tensile member strain (ϵ_{sm}) plots of SFRC and RC specimens, for rebars having a diameter of 20 mm and 30 mm, respectively. The average member strain ϵ_{sm} was calculated as the mean elongation of the 4

LVDTs, divided by the length of the base measurement. In both the diagrams, a comparison between the corresponding typical RC (plain, 2nd phase, [15]) and SFRC members are provided. In addition, the response of the bare bar is reported.

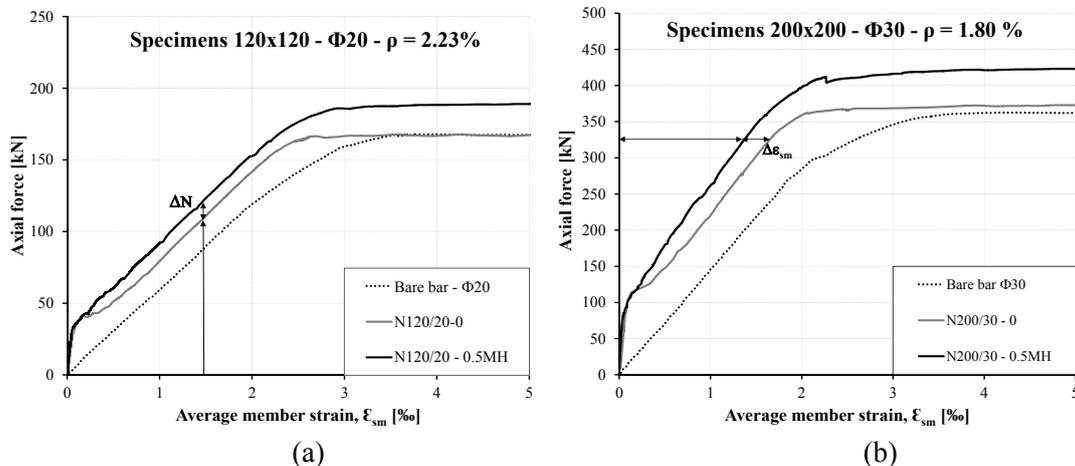


Figure 4. Typical response of tie elements: (a) specimen N120/20 (b) specimen N200/30.

Even though tests were conducted well beyond that value, the results are plotted up to a maximum average strain of $5 \cdot 10^{-3}$ mm/mm in order to appropriately assess the tensile behavior at SLS and capture the behavior at first yielding. By inspection of the diagrams, it is evident that fibrous and non-fibrous samples experience first cracking at approximately the same load level. This is attributed to the fact that fibers generally do not affect the concrete tensile strength, at least for volume fractions lower than 1%, for strain-softening materials. By further increasing the applied load, the phase where no new cracks occur, and those existing widen, is denoted as the “stabilized crack stage”: it takes place for $\epsilon_{sm} = 0.87 \cdot 10^{-3}$ mm/mm and $0.47 \cdot 10^{-3}$ mm/mm in groups N 120/20 and N 200/30, respectively (Figure 4). One should notice that there is no clear evidence to support a possible relationship between strains at the crack stabilized stage and fiber toughness. Counterintuitively, the higher number of cracks reported in SFRC elements, in fact, form in the same range of average strains as in their RC counterpart elements.

The typical responses of FRC tensile ties reported in Figure 4 enable to emphasize one of the two main advantages related to the combination of rebars and fibers; that is the global stiffness increase caused by the transmission of noticeable residual stress across cracks. This tendency can be recognized also during the crack formation stage but it is especially clear during the stabilized crack stage. In fact, as schematically depicted in Figure 4a, by referring to a certain value of average strain (ϵ_{sm}), the improved toughness due to fibers determines an increment of the average tensile stress of the undamaged concrete portions within cracks, resulting in an increment of the applied global force ΔN . Hence, the tension stiffening increases with respect to that of the RC counterpart specimens. This is consistent with reported improvements of incorporating of fiber into different types RC materials and structural elements [21-24]. In the same way, by referring to a certain axial load level, a considerable reduction to the average strain occurs (denoted as $\Delta \epsilon_{sm}$ in Figure 4b). The effect is referred to in literature as “tension stiffening strain”.

3.2. Evolution of the mean crack spacing and of the tension stiffening strain

The expected mean crack width of a SFRC member diminishes because of the post-cracking toughness of SFRC, involving two important contributors: a reduction of both the mean crack spacing (s_{rm}) and the average member strain (ϵ_{sm}). The former aspect has been investigated in the diagrams reported in Figure 5a, while the latter in Figure 5b, for series N 120/20.

The mean crack spacing of a single specimen was evaluated by measuring the distance between visible cracks on the surface. Furthermore, for each set of samples, s_{rm} was calculated as the mean

value of the measured mean values of each single specimen. In Figure 5a, the evolution of the mean crack spacing s_{rm} is plotted as a function of the average strain up to the end of the crack formation stage for specimens N 120/20. These diagrams report the average curves of the 3 specimens belonging to series SFRC 0.5MH. The s_{rm} vs. ϵ_{sm} response of the corresponding control RC counterpart specimens are also reported for reference. The latter diagrams were obtained in a previous experimental study [15].

From this plot, the reduction of the mean crack spacing can be readily noticed. The residual post-cracking strength provided by steel fibers (at any crack) contributes to the reduction of the transmission length (l_t) necessary to transfer tensile stresses in concrete through bond. This effect can be considerable even if it is assumed, as a first approximation, that the bond stresses between rebar and surrounding concrete are not affected by fibers. This assumption is currently also supported by MC 2010 [1]. Considering the entire sets of tests, it was estimated an average percentage reduction of mean crack spacing for series SFRC 0.5MH with respect to the plain RC counterparts of about 31.

The average member strain of fibrous and non-fibrous tensile ties (basically the average strain of the rebar embedded in a prismatic tie, ϵ_{sm}) is provided as a function of the bare bar strain ϵ_s . Consider first a range of strains up to 3×10^{-3} mm/mm, corresponding to the stabilized crack stage up to the onset of yielding. By referring to a given applied force, represented by a certain bare bar strain (ϵ_s), in plain RC elements, the average member strain ϵ_{sm} is lower (tension-stiffening strain). Moreover, in the presence of fibers, a further increase of this phenomenon can be observed as in Figure 5b. From this plot, a reduction of about 28% is noted between SFRCs and the control RC series at a bare bar strain ϵ_s of $1 \cdot 10^{-3}$ mm/mm. This reduction generally depends on the longitudinal steel ratio (ρ_{eff}): the lower the ratio, the higher the percentage reduction is. Moreover, it strictly depends on the concrete toughness, without any influence of the tensile strength. By considering all the samples, series SFRC 0.5MH presents a reduction of about 42% with respect to the control samples.

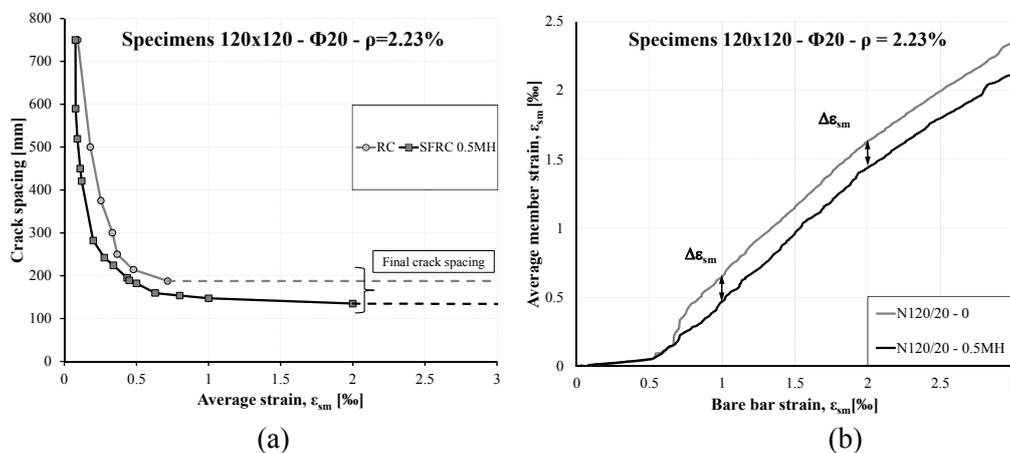


Figure 5. SFRC and RC series: evolution of the crack spacing (a) and of the tension stiffening strain (b).

In Figure 6a, the mean crack spacing (s_{rm}) is plotted versus the key parameter ϕ/ρ_{eff} , which is generally included in many building codes for the prediction of s_{rm} . In particular, the experimental results are plotted for the plain RC specimens as well as the SFRC 0.5MH. The diagrams reported in Figure 6a confirm the tendency previously observed in Figure 5a: for the same value of ϕ/ρ_{eff} , the use of fibers results in a considerable reduction of the mean crack spacing due to the enhanced material toughness. Since several formulations proposed in the literature and in design codes define the crack spacing to be linearly proportional to the parameter ϕ/ρ_{eff} , a linear regression was utilized in order to evaluate the dispersion of the experimental results. As depicted in Figure 6a, the coefficient of correlation R^2 is equal to 0.92 for RC (plain control) samples whereas it is 0.97 for SFRC 0.5MH

members. Basically, a possible linear relationship between s_{rm} and ϕ/ρ_{eff} can be found appropriate. Even though, for SFRC, that should be considered with the addition of a further built-in parameter taking into account the post-cracking residual strength provided by fibers.

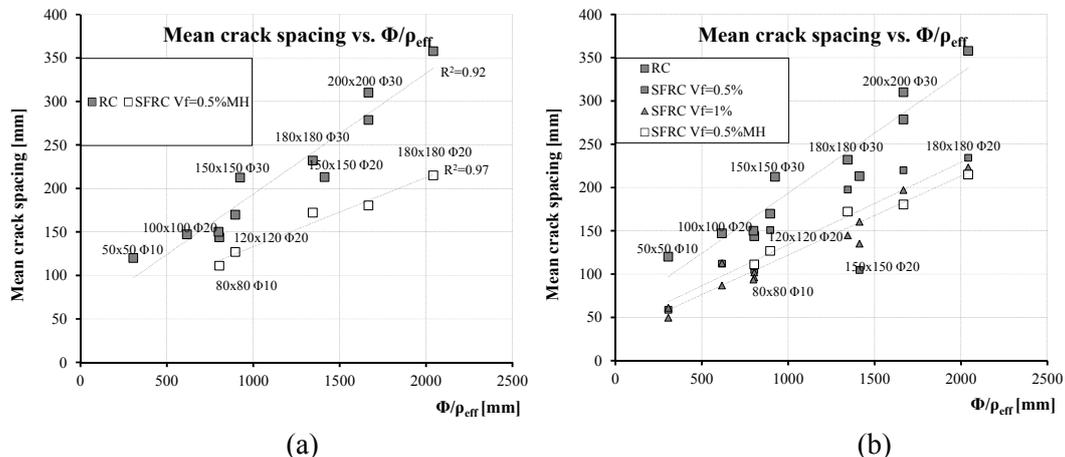


Figure 6. Mean crack spacing vs. ϕ/ρ_{eff} non-fibrous series/fibrous series: (a) SFRC 0.5MH; (b) comparison with samples of previous experimental campaigns [15].

In Figure 6b, the comparison between s_{rm} and ϕ/ρ_{eff} was plotted by considering results from the SFRC 0.5MH samples investigated in the research work presented herein and those of a previous study [15]. It can be observed a rather similar behavior of previously tested SFRCs series, whose global mean reductions of s_{rm} was equal to 30% for SFRC with $V_f=0.5\%$ and 37% for SFRC with $V_f=1\%$. The latter are rather similar to the average reductions exhibited by series SFRC 0.5MH (31%), even though the SFRC 0.5MH series demonstrate much higher post-cracking residual strengths (f_{R1m}). Based on these results, it seems that the increment of f_{R1m} beyond typical ranges of 4-5 MPa does not provide a corresponding reduction of mean crack spacing. This is evidenced by the fact that the SFRC 0.5MH group had an f_{R1m} in excess of 7 MPa.

Recently, the MC2010 [1] proposed a relationship for estimating the mean crack spacing, in which the fiber resistant contribution is taken into account by means of a reduction (f_{Ftsm}) applied to the introduction length ($l_{s,max}$, generally assumed for RC elements), which includes the FRC toughness at SLS ($f_{Ftsm}=0.45 f_{R1m}$). Accordingly, the following expression can be derived:

$$s_{rm} = 1.17 \cdot l_{s,max} = 1.17 \cdot \left[k \cdot c + \frac{1}{4} \cdot \frac{\phi}{\rho_{eff}} \cdot \frac{(f_{cm} - f_{Ftsm})}{\tau_{bm}} \right] = 1.17 \cdot \left[k \cdot c + \frac{1}{4} \cdot \frac{\phi}{\rho_{eff}} \cdot \frac{(f_{cm} - 0.45 \cdot f_{R1m})}{\tau_{bm}} \right] \quad (1)$$

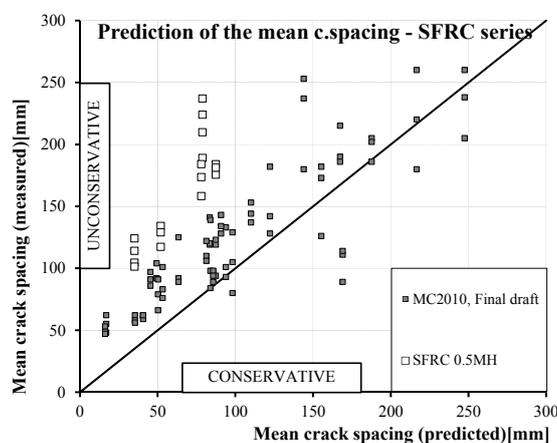


Figure 7. Crack spacing prediction for fiber series according to MC2010 [1].

Eq.1 has been applied based on the fracture parameters (mean values) reported in Table 4. The factor k was assumed equal to 1, while the bond stress (τ_{bm}) over the concrete tensile strength (f_{ctm}) ratio was assumed equal to 1.8 even for SFRC specimens. The predictions of Eq.1 are reported in Figure 7 while indicating the regions where it is conservative or otherwise. It is important to underline that the Mean Absolute Percentage Error (MAPE) for the SFRC 0.5MH series, is about 54%, as a result of un-conservative estimation for most of the samples. Hence, it can be seen that the current MC2010 [1] formulation does not consider that beyond 4-5 MPa values for residual strengths (f_{R1m}), the crack spacing may not be further reduced.

4. Concluding remarks

In the present paper, an experimental study was presented aiming at evaluating the cracking behavior of RC and SFRC ties, with special focus on the enhancement of crack control due to the addition of fibers. A series of 17 tension-tie specimens' tests have been carried out on SFRC samples exhibiting high post-cracking residual strengths. Based on the presented results and findings, the following main conclusions emerge:

- SFRC positively influences the behavior of tension-ties at SLS due to two main aspects: in SFRC elements, the tension-stiffening significantly increases and the mean crack spacing (s_{rm}) reduces in comparison to plain RC members. A crack spacing reduction of around 31% was seen in SFRC elements with $V_f=0.5\%$.
- An increase in reinforcement ratio ρ decreases the mean crack spacing of both SFRC and plain RC elements.
- For a given reinforcement ratio, an increase in the conventional reinforcing bar diameter increases the mean crack spacing of both SFRC and non-fibrous concrete (with similar trend).
- SFRCs with rather high post-cracking strengths (f_{R1m} higher than 7 MPa) do not present a noticeably lower mean crack spacing with in comparison to SFRCs with f_{R1m} around 4-5 MPa. This phenomenon is not currently accounted for by the crack spacing formulation of MC2010 [1].

Acknowledgments

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