Crack Control in RC Elements with Fiber Reinforcement

Fausto Minelli, Giuseppe Tiberti and Giovanni Plizzari

SYNOPSIS

Durability is nowadays a key-parameter in Reinforced Concrete (RC) structures. Several codes require that structures have a defined service life during which the structural performance must satisfy minimum requirements by scheduling only ordinary maintenance.

Durability can be associated to permeability, defined as the movement of fluid through a porous medium under an applied pressure load, which is considered one of the most important property of concrete. Permeability of concrete is strictly related to the material porosity but also to cracking. The former is basically controlled by the water/cement (w/c) ratio while microcracks and cracks are related to internal and external strains or deformations experienced by the RC structures. Shrinkage, thermal gradients and any factor determining volumetric instability, as well as the loads acting on a structure, lead to both microcraking and visible cracking.

It is well known that, after cracking, tensile stresses are induced in the concrete between cracks and, hence, stiffen the response of a Reinforced Concrete (RC) member under tension; this stiffening effect is usually referred to as “tension stiffening”. After the formation of the first crack, the average stress in the concrete diminishes and, as further cracks develop, the average stress will be further reduced.

When considering Fiber Reinforced Concrete (FRC), an additional significant mechanism influences the transmission of tensile stresses across cracks, arising from the bridging effect provided by the fibers between the crack faces; this phenomenon is referred to as “tension softening”. Fibers also significantly improve bond between concrete and rebars and act to reduce crack widths.

The combination of these two mechanisms results in a different crack pattern, concerning both the crack spacing and the crack width.

The present paper describes results from a collaborative experimental program currently ongoing at the University of Brescia and at the University of Toronto, aimed at studying crack formation and development in FRC structures. A set of tensile tests (52 experiments) were carried out on tensile members by varying the concrete strength, the reinforcement ratio, the fiber volume fraction and the fiber geometry.

Keywords: cracking, durability, Steel Fiber Reinforced Concrete, Tension Softening, Tension Stiffening, Toughness.

Biography:

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INTRODUCTION

Durability is nowadays a key-parameter in Reinforced Concrete (RC) structures. Several codes require that structures have a defined service life during which the structural performance must satisfy minimum requirements by scheduling only ordinary maintenance.

Durability can be associated to permeability, defined as the movement of fluid through a porous medium under an applied pressure load, which is considered one of the most important property of concrete [1]. Permeability of concrete is strictly related to the material porosity but also to cracking. The former is basically controlled by the water/cement (w/c) ratio as well as by the degree of hydration and compaction. Microcracks and cracks, on the other hand, are related to internal and external strains or deformations experienced by the RC structures. Shrinkage, thermal gradients and any factor determining volumetric instability, as well as the loads acting on a structure, lead to both microcraking and visible cracking.

Cracking in reinforced concrete structures causes degradation, corrosion of rebars (Fig. 1), decrease in the steel bar resisting area and concrete spalling, that are all detrimental factors leading to a loss of structural durability [2].

In order to contrast cracking phenomena, concrete should have an enhanced toughness which is generally provided by structural fibers added to the concrete matrix.

A significant and comprehensive research on durability of RC structural elements should therefore be addressed not only towards the study of the permeability, as it is currently done by controlling the w/c ratio, but also towards the control of cracking phenomena.

The durability issue will be herein considered by focusing the attention on cracking in ordinary (RC) or Fiber Reinforced Concrete (FRC) elements. Several tests were performed on tensile members that allow a better understanding of the basic principles governing the behavior of RC elements. Notice that the classical theory usually refer to an element under tension or to a beam under constant bending moment, which is a rather reasonable simplification of most structures under a rather uniform distributed load. An extension to a beam with a variable bending moment, as it occurs at the internal supports of a continuous beam, has been proposed by Giuriani et al. [3], [4], who focused on the local effect at a crack in a region with a significant high gradient of tensile force in the reinforcing bar (rebar).

The deformation of a rebar embedded in concrete is significantly influenced by the bond between the two materials. In fact, it is well known that, after cracking, bond transfers tensile stresses from the rebar to the surrounding concrete (between cracks) that stiffen the response of a RC member subjected to tension; this stiffening effect is referred to as "tension stiffening". Back in 1908, Mörsch [5] explained this phenomenon using the following way: *"Because of friction against the reinforcement, and of the tensile strength which still exists in the pieces lying between the cracks, even cracked concrete decreases to some extent the stress of the reinforcement"* [6]. Several Authors studied this mechanism in traditional RC elements [7, 8].

The use of Fiber Reinforced Concrete (FRC) has gained considerable attention in recent years, as demonstrated by its recent inclusion in the new fib Model Code 2010 [9] and by many international committees [10] or conferences, [11], [12].

When fibers are added into concrete, an additional significant mechanism influences the transmission of tensile stresses across cracks, arising from the bridging effect of fibers that provide a considerable post-cracking residual strength to FRC; hence, the concrete tensile stresses at a crack remain significant until a critical crack width, depending on the fiber type/s, dosage and toughness, is reached. This phenomenon is generally referred to as "tension softening", even if, for high-performance FRCs, a "strain-hardening" behavior may occur. Furthermore, fibers significantly enhance the bond between concrete and rebars and also act to reduce crack widths [13].

It clearly turns out that, in FRC elements, the combination of these two mechanisms (tension stiffening and the post-cracking residual strength provided by fibers at any crack) results in a different crack pattern, namely both the crack spacing and crack width. The collapse mode and the ductility of the elements may also be affected by stress concentrations due to optimized bond and the residual tensile stress at a crack [14].

Under a design point of view, the final crack spacing provided in many analytical models or codes for RC elements (which refers to the stabilized crack stage) might be a rough and not adequate estimation for FRC structures, where fibers could be adopted for crack control. Therefore, crack development in FRC elements should be adequately investigated. Many preliminary studies have been carried out so far. Mitchell [15] presented one of the first studies by clarifying the beneficial effect of fibers in determining narrower and closely spaced cracks, as well as in mitigating the splitting cracks in the end regions while having low concrete covers. Bischoff ([16, 17 and 18]) performed monotonic and cyclic tension-stiffening tests and included shrinkage effects in the analysis. Tension-stiffening results were used to determine the average tensile response of concrete after cracking, as well as to estimate the crack spacing. Shrinkage causes an initial shortening in the member response that, according to the Author, must be accounted for correctly evaluating tension-stiffening effects. Noghabai [19] proposed an analytical model which describes the behavior of tie-elements based on the observation of experimental tests.

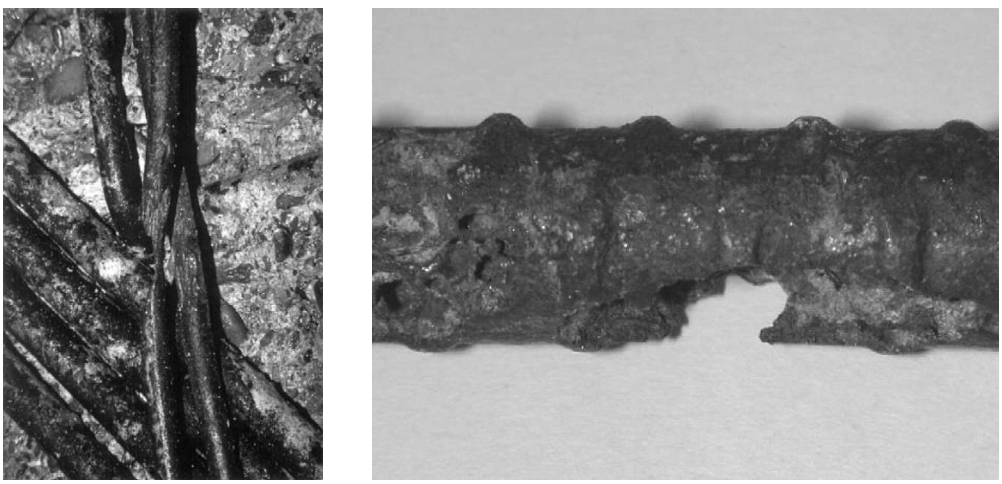


Fig. 1. Effect of corrosion on rebars.

The present paper describes a number of experimental results from a collaborative research program currently ongoing at the University of Brescia (Italy) and at the University of Toronto (Canada), aimed at studying crack formation and development in FRC structures. A set of tension stiffening tests was carried out by varying the concrete strength, the reinforcement ratio, the fiber volume fraction and the fiber geometry. Brescia tested and interpreted tension members made of Normal Strength Concrete (NSC) while Toronto tested identical members made of High Strength Concrete (HSC; with a concrete strength of around 60 MPa). This common research is intended to investigate the cracking formation and development of FRC members toward a consistent modeling of the cracking phenomena in these materials. Only results on 52 tests, obtained at the University of Brescia on normal strength concrete, are presented herein.

EXPERIMENTAL PROGRAM

GEOMETRY AND MATERIAL PROPERTIES

Eighty-eight RC prismatic members, having the geometry shown in Fig. 2, were cast with NSC; each specimen was 950 mm long (Fig. 2). Five square cross sections were selected (50, 80, 100, 150 and 200 mm size). Reinforcing bars having a diameter of 10, 20 and 30 mm (B450C steel), corresponding to a reinforcement ratio varying from 1.24% to 3.24%, have been used throughout this study. The concrete cover was at least 2-3 times the bar diameter in all cases, in order to avoid splitting phenomena during the tests [15, 20].

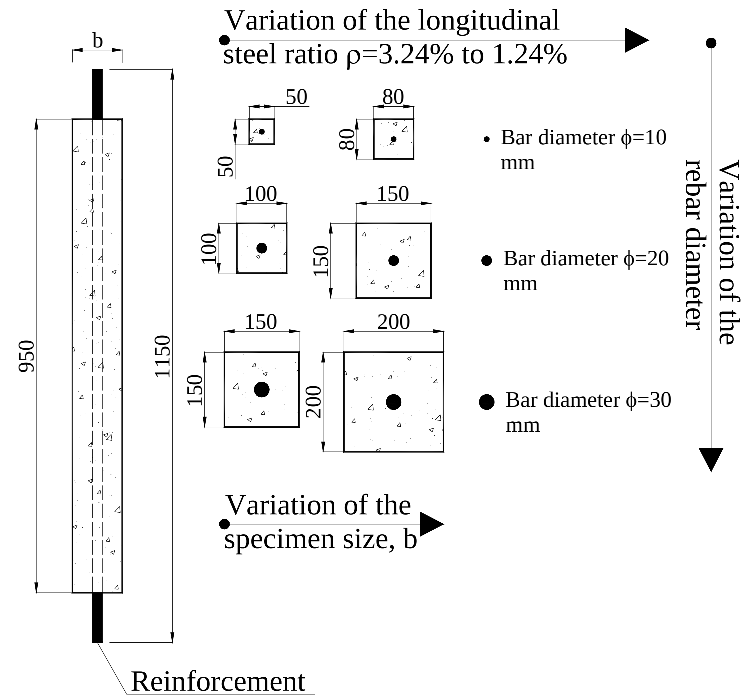


Fig. 2. Geometry and reinforcement details of specimens.

It should be noticed that secondary reinforcement (links or stirrups) was not used in this research since they can alter the crack spacing [21].
All samples were cast with the same NSC having a characteristic compressive strength ($f_{ck,cube}$) of 45 MPa (C35/45 according to Eurocode 2, [22]).
Different dosages and types of steel fibers were included into the concrete matrix: both macro and micro fibers were adopted with three different volume fractions (0.5%, 1% and 2%). The micro fibers were only used in addition to macro fibers, determining a hybrid system that can help both with regard to early cracking (controlled by micro fibers) and for diffused macro-cracking (mainly controlled by macro fibers [23, 24]). Steel macro fibers were hooked-end, 30 mm long and had a diameter of 0.62 mm (aspect ratio equal to 48), while steel micro fibers were straight, 13 mm long, with a diameter of 0.2 mm (aspect ratio of 65). The fiber types are quoted in Table 1 while

Table 2 shows a representation of the different fibers adopted and their dosages, with the mix composition designation. 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 the present paper, the results concerning plain and SFRC (Steel Fiber Reinforced Concrete) tests with a volume fraction (V_f) of fiber up to 1.0%, will be presented, for a total number of 52 specimens (

Table 3).

NSC mix (Table 4), identical in all batches, was designed by adopting the same grain size distribution of the aggregates, according to the Bolomey curve [25]. To ensure a greater workability, especially in the case of V_f=1%, a

10 mm maximum nominal aggregate size was adopted and the percentage of fine aggregates was conveniently increased (for all materials). To this aim, the Bolomey parameter A, representing the mix workability and consistency, was increased up to 16.

Table 1. Geometrical and mechanical properties of the different types of fibers adopted.

Designation	30/0.62	13/0.2
Type of steel	Carbon	High carbon
Shape	Hooked	Straight
Minimum Tensile Strength [MPa]	1270	2000
Length [mm]	30	13
Diameter [mm]	0.62	0.2
Aspect Ratio l/ϕ	48	65
Fibers per kg	13000	314000

Table 2. Representation of different fiber types and contents added to the concrete matrix.

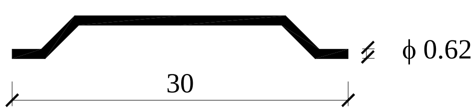
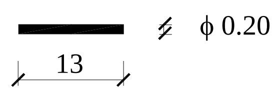
Type of concrete	Fibers 30/0.62	Fibers 13/0.2
		
0 (Plain Concrete)	-	-
0.5M	0.5%	-
1M	1%	-
1M+m	0.5%	0.5%
2M+m	1%	1%

Table 5 reports, for all batches, the compressive strength values, measured from 150 mm side cubes.
The fracture properties of SFRC were determined according to the European Standard EN 14651 [26], which requires bending test (3PBT) to be performed on small beam specimens (150x150x550 mm). For each SFRC batch, 3PBTs on notched beams were carried out. The experimental curves, concerning the total nominal stress vs. crack mouth opening displacement (CMOD), are depicted in Fig. 3. Parameters f_{Ri}, representing the post-cracking residual strengths for different values of $CMOD_j$, and the flexural tensile strength (limit of proportionality) f_L were calculated and are listed in
Table 6.

TEST SET UP AND INSTRUMENTATION

Tensile tests were performed by means of an INSTRON hydraulic servo-controlled (closed-loop) testing machine, having a capacity of 500 kN. Tests were carried out under stroke control (by clamping both the rebar ends) by monitoring the specimen behavior up to the onset of the strain-hardening branch of the rebar. The deformation rate up the yield limit of the steel was varied from 0.1 to 0.2 mm/min. On the yield plateau, the rate was increased to 0.5 mm/min.
A typical instrumented specimen is shown in Fig. 4a: four Linear Variable Differential Transformers (LVDTs, one for each side), were placed to measure the deformation of the specimen over a length of 900 mm (refer to the schematic sketch of Fig. 4b).

All specimens made of plain concrete and SFRC with V_f=1% (macro fiber) were stored in a fog room (R.H. > 95%; T=20 ± 2°C) until 2 or 3 days before testing; then they were air dried in the laboratory. All the other specimens were moist cured with wet burlap under plastic sheet until 2 or 3 days before testing, since it was not possible, for space restriction, using the same fog room. For these specimens, shrinkage effects were not probably totally controlled, even though the member response could be corrected by taking into account the effect of the initial shrinkage strain [17]; however, shrinkage does not significantly influence the final crack pattern and crack spacing.

Table 3. Experimental program and specimen notation (*specimens already tested).

ϕ	V_f	b [mm]	A_s [mm^2]	$A_{c,eff}$ [mm^2]	Reinf. Ratio (%)	Clean cover [mm]	Specimen ID	# of specimens
ϕ 10	0*	50	79	2421	3.24	20	N 50/10 - 0	3
	0.5%*						N 50/10 – 0.5M	3
	1.0 %*						N 50/10 - 1M	3
	0.5%+0.5%*						N 50/10 - 1M+m	3
	1%+1%						N 50/10 - 2M+m	3
ϕ 10	0*	80	79	6321	1.24	35	N 80/10 – 0	2
	0.5%*						N 80/10 – 0.5M	3
	1.0 %*						N 80/10 - 1M	3
	0.5%+0.5%*						N 80/10 - 1M+m	3
	1%+1%						N 80/10 - 2M+m	3
ϕ 20	0*	100	314	9686	3.24	40	N 100/20 - 0	3
	0.5%*						N 100/20 – 0.5M	3
	1.0 %*						N 100/20 - 1M	3
	0.5%+0.5%*						N 100/20 - 1M+m	3
	1%+1%						N 100/20 - 2M+m	3
ϕ 20	0*	150	314	22186	1.41	65	N 150/20 – 0	3
	0.5%*						N 150/20 – 0.5M	3
	1.0 %*						N 150/20 - 1M	3
	0.5%+0.5%*						N 150/20 - 1M+m	3
	1%+1%						N 150/20 - 2M+m	3
ϕ 30	0*	150	707	21793	3.24	60	N 150/30 – 0	3
	0.5%						N 150/30 – 0.5M	3
	1.0 %						N 150/30 - 1M	3
	0.5%+0.5%						N 150/30 - 1M+m	3
	1%+1%						N 150/30 - 2M+m	3
ϕ 30	0*	200	707	39293	1.80	85	N 200/30 – 0	2
	0.5%						N 200/30 – 0.5M	3
	1.0 %						N 200/30 - 1M	3
	0.5%+0.5%						N 200/30 - 1M+m	3
	1%+1%						N 200/30 - 2M+m	3

Table 4. Mix composition of NSC for the specimens without fibers (quantities refer to 1 m^3). Note that with fibers the aggregate amount lowers up to a small 4%.

	Weight [kg]	Volume [litre]
Cement Portland 42,5 R	400	127
Water	189	188.6
Superplasticiser	4	3.3
Aggregates	1742	652.3
Air assumed	-	28.8
Water/cement ratio	0.47	-

Table 5. Concrete compressive strength.

Type of concrete	Compressive strength	
	Days after casting	$f_{cm,cube}$ [MPa]
0 (Plain)	28	48.5
	34	49.0
0.5M	21	46.0
	42	48.8
1M	58	43.9
1M+m	77	54.2
	116	51.2

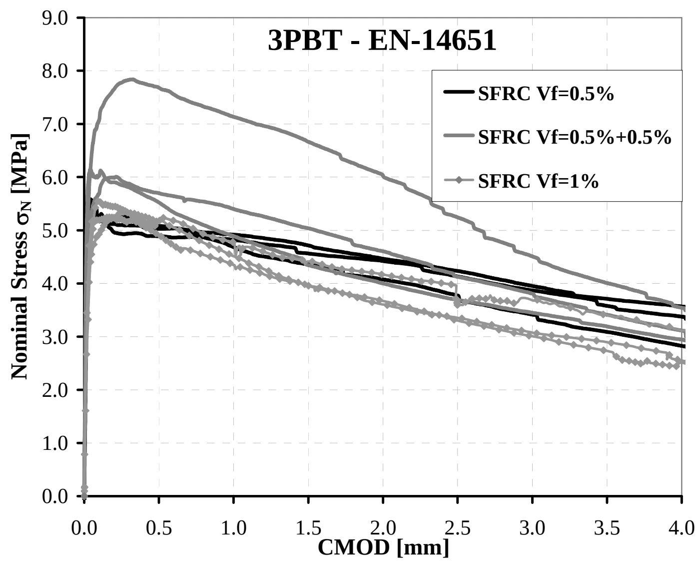
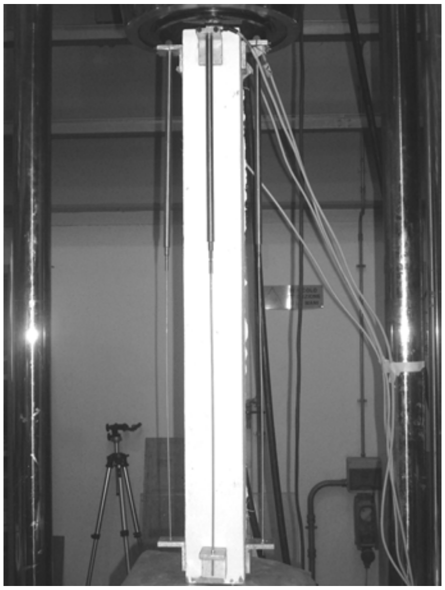
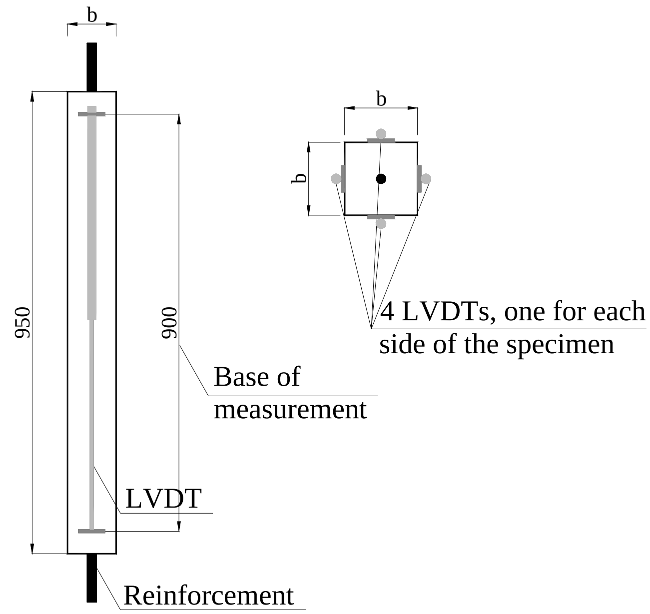


Fig. 3. Experimental results of 3PBT SFRC notched beams according to EN 14651.

Table 6. Principal fracture parameters of the SFRCs adopted.

Batch	f_L [MPa]	f_{R1} [MPa]	f_{R2} [MPa]	f_{R3} [MPa]	f_{R4} [MPa]
0.5M	5.46	5.00	4.55	4.05	3.46
1M+m	5.97	6.30	5.35	4.35	3.54
1M	5.39	5.09	4.11	3.42	3.01





(a) (b)

Fig. 4. Configuration of the reinforced concrete member during test (a); instrumentation of the tie-element (b).

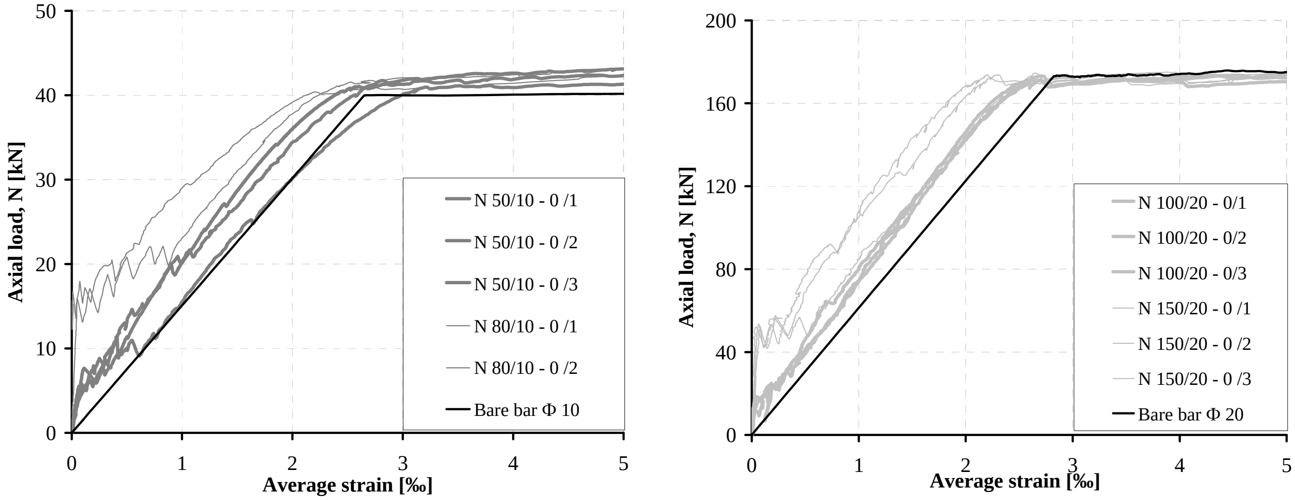
RESULTS AND DISCUSSION

CONTROL SPECIMENS

The diagrams reported in Fig. 5a,b evidence the response in terms of axial load vs. average tensile member strain for the control specimens (Φ=10 mm and 20 mm bar diameter, respectively). The average member strain has been calculated as the average elongation of the 4 LVDTs, divided by the length of the base measurement (about 900 mm). In both the diagrams, a comparison between plain and the corresponding bare bar is reported, whose response was determined by means of preliminary uniaxial tests.

These plots clearly emphasize the influence of the different longitudinal steel ratios adopted, basically corresponding to different concrete covers, c (with all samples herein considered having the same rebar). It can be noticed that, for the smallest cover (N 50/10 and N 100/20 specimens), the cracks are distributed to such an extent that their localization cannot be so clearly distinguished in the load vs. average strain response. On the other hand, for specimens having the largest cover, the localization on the surface cracks is evidenced in distinctive peaks; the same tendency was stated by Noghabai [27].

The effect of concrete cover on crack spacing is still a matter of a strong debate. The concrete cover is included in some codes or analytical models [22] whereas it is neglected in others. The abovementioned fact that larger covers involve more distinct load-drops, once cracks form, can be well explained, for the present set of tests, also by considering the lower reinforcement ratio (as a consequence of higher "c"), which is not able to control the sudden local instability at the crack formation, as a higher bar diameter would do. To solve this riddle, further studies should be done focusing only on the effect of concrete cover (by keeping all other parameters constant).



(a) (b)

Fig. 5. (a) N 50/10, 80/10 and (b) N100/20, N150/20 member responses for plain concrete.

No significant splitting cracks appeared in the members, also in the post-yielding branch, except for the control samples having the largest rebar diameter (Φ=30 mm) and for specimens having Φ=20 mm and size of 100 mm (only for high deformation values). Splitting phenomena appeared when the concrete body was segmented into separated blocks for deformations, well beyond the yielding of the reinforcing bar [18].

For each specimen it was possible to calculate ΔN, which is basically the difference in load between the experimental curve (from the tensile specimen) and the corresponding bare bar. As a first approximation, the ΔN term was then divided by the area of concrete in tension in order to obtain an estimation of the average concrete tensile contribution, σ_c. One should notice that the term “average” refers to the equilibrium conditions in any uncracked transverse section, as stated by Vecchio and Collins [28] and by Collins and Mitchell [6].

In Fig. 6 the mean value of the tension-stiffening contribution (σ_c) is plotted vs. the average member strain (the mean value is obtained by each set of plain concrete samples); the diagram evidences that tension stiffening is not constant once the crack pattern is stabilized [15]. Bischoff [17] describes this phenomenon as a progressive reduction of the bond-factor, β, which represents the average load (or stress) carried by the cracked concrete, normalized with the cracking strength.

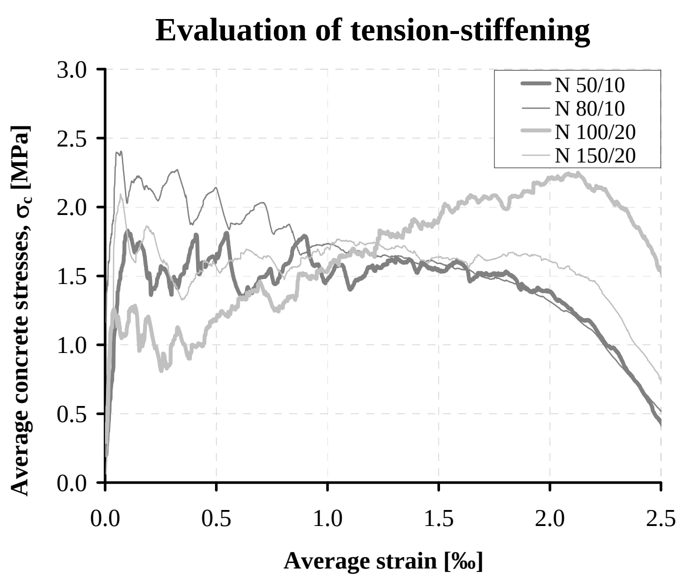


Fig. 6. Evaluation of the average tension-stiffening response of plain concrete reference samples.

FRC SPECIMENS

SFRC tensile members present a different structural behavior with respect to plain concrete ones. In fact, after cracking, fibers provide a noticeable increment of concrete toughness, guaranteeing a considerable residual strength between cracks; this phenomenon is generally neglected in plain concrete, due to its brittleness.

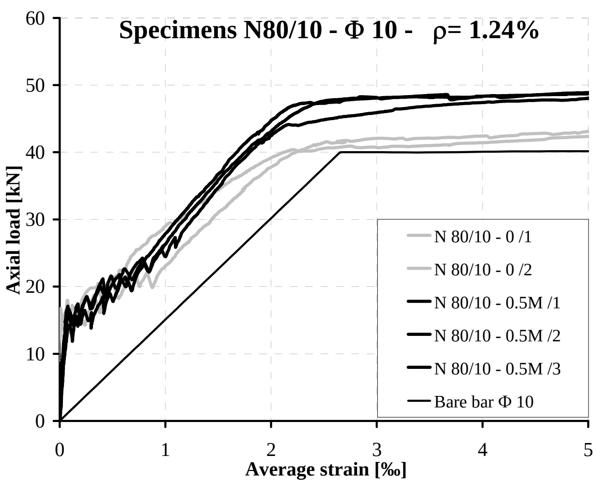
The increased stresses after cracking influence the behavior of the specimens under investigation for two principal aspects [18]:

1) by referring to a certain average member strain (Fig. 7 and Fig. 8), the improved toughness of SFRC determines an increment of the average tensile stress of the undamaged concrete portions between two consecutive cracks; hence, the tension stiffening increases. In the same way, by referring to a certain axial member load, a considerable reduction of the average member strain will occur (tension stiffening strain).

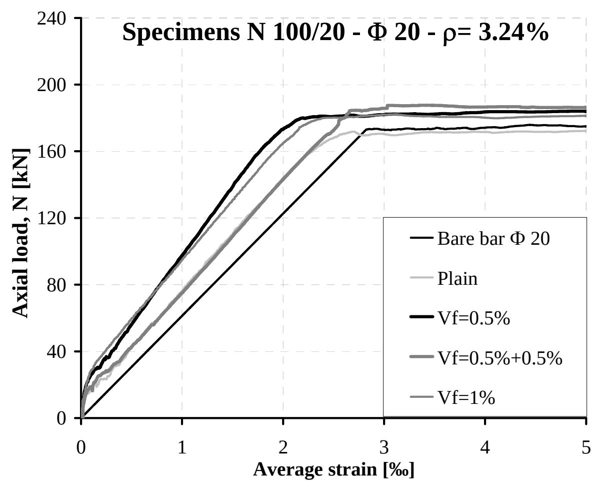
2) The residual post-cracking strength provided by steel fibers (tension softening or hardening, well pictured in Fig. 7 and Fig. 8) contributes to the reduction of the transmission length, measured from a crack, which is necessary to transfer tensile stresses in concrete by means of bond between traditional rebars and surrounding concrete; hence, the average crack opening diminishes. It should be noticed that this advantage could be further amplified by considering the improvement in the steel-to-concrete bond provided by fibers [13].

The diagrams reported in Fig. 7 highlight the first previously mentioned aspect. In fact, the response in terms of axial load vs. average tensile member is presented in Fig. 7a,b for N 80/10 and N 100/20 specimens respectively (note that only SFRC V_f=0.5% batch is plotted in Fig. 7a while all SFRC samples in Fig. 7b). In both the diagrams, a comparison between plain and SFRC members is proposed and, also, the response of the corresponding bare bars is reported.

The results have been plotted up to a maximum average strain of $5 \cdot 10^{-3}$, in order to properly describe the behavior at the serviceability limit state (SLS), where the crack and displacement control is of main importance, but also for assessing the tensile behavior after yielding of the rebar. It can be observed that the SFRC specimens exhibit a clear improvement of the tension stiffening in the stabilized crack stage. This tendency can be observed also after yielding, where the fiber resistant contribution (post-cracking strength) is clearly evidenced. Whereas in the control samples there is no possibility of increasing the ultimate capacity of a member after yielding (assuming a simple elastic-perfect plastic constitutive law for steel), FRC toughness allows for the transfer of residual tensile stresses at crack with a consequent increase of the bearing capacity of the structural member (Fig. 7).

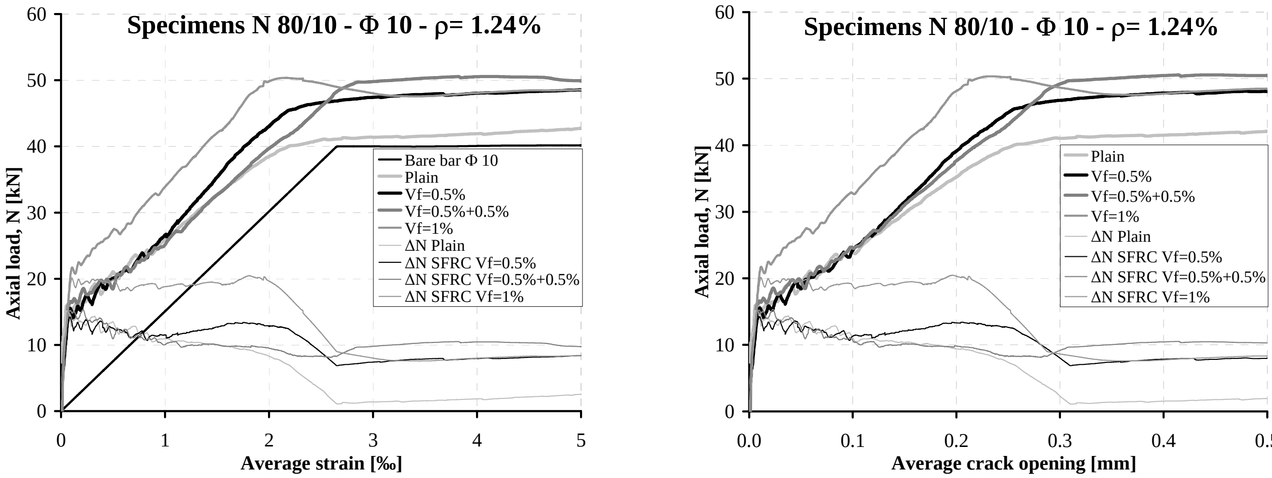


(a)



(b)

Fig. 7. N 80/10 member responses for plain concrete and SFRC (V_f=0.5%) (a); N100/20 average responses for all specimens (b).



(a) (b)

Fig. 8. N 80/10 average responses in terms of strain (a) and crack width (b). Comparison between control specimens and SFRC samples.

The response of specimens N 80/10 (for all the reinforcement configurations adopted) is presented in detail in Fig. 8a, by plotting the average load response vs. the average strain. Fig. 8a also depict the ΔN term (as illustrated in the dashed lines on the bottom of plots). The results evidence that the enhanced tension stiffening due to fibers tends to be less pronounced up to an average strain equal to $1{\cdot}10^{-3}$ (approximately end of the crack formation stage), except for specimens with a volume fraction V_f=1% of macro fibers (which experienced the same curing condition of the control samples). On the other hand, the combined effect of tension stiffening and residual stresses at crack, both incorporated in ΔN, increases quickly till the occurrence of a stabilized crack stage: at this level, the cracks already formed grow and, hence, the fiber resistant contribution can be exploited. Differently from plain concrete elements, in SFRC ΔN remains constant or even improves in some samples. When yielding occurs (for a strain of about $2{,}7{\cdot}10^{-3}$), the higher bearing capacity of SFRC specimens, with respect to the control samples, is mainly due to the higher residual post-cracking strength provided by fibers (ΔN is basically attributed to tension softening or hardening). It can be also noticed, from the diagrams plotted in Fig. 8a, that for specimens N 80/10 (having low concrete cover and longitudinal steel ratio equal to 1.24%) the cracking process is not characterized by a series of small local peaks but the curve is rather stable; this phenomenon is clear in specimens with 1% of macrofibers. The curves plotted in Fig. 8a and b seem to emphasize that, by using a fiber content 1% of macro fibers, it is possible to gain a noticeable improvement of the tensile response. Specimens with a combination of micro and macro fibers exhibit a similar maximum load but a different tension stiffening; this behavior is probably related to shrinkage due to the curing conditions (specimens SFRC 1%M were cured in the fog room, while specimens SFRC 0.5%M+0.5%m were cured in laboratory conditions).

CRACK WIDTHS AND SPACINGS

The second significant aspect herein investigated concerns the crack pattern and its evolution in terms of number of cracks, crack spacing and, more important, average crack opening.

The average crack spacing of a single specimen was evaluated by measuring the distance between visible cracks on the surface. Furthermore, both the average crack opening and the average crack spacing of each set of samples were calculated as the mean values of the measured average values on single specimen.

As already mentioned, shrinkage does not considerably affect the crack spacing which strictly depends on the steel-to-concrete bond, to the longitudinal steel ratio, to the bar diameter and to the toughness provided by fibers. Cracks in SFRC specimens were well distributed and rather closely spaced. Fig. 9 shows schematic sketches of crack patterns of specimens N 50/10. No particular crack localization was observed, even after yielding of the rebars. All SFRC specimens exhibit an average crack spacing that is less than one-half of the control samples. In addition, the crack patterns of the 1%M specimens are quite similar to those of 0.5%M+0.5%m specimens.

The beneficial effects provided by fibers in terms of crack control can be further clarified by looking at the diagram of the axial load vs. average crack opening, plotted in Fig. 8b for the N 80/10 specimens. As an example, at yielding point, well after serviceability limit states, the average crack width is about 0.25 mm for SFRC elements; this value is still compatible with the requirements of most of the design codes for service conditions.

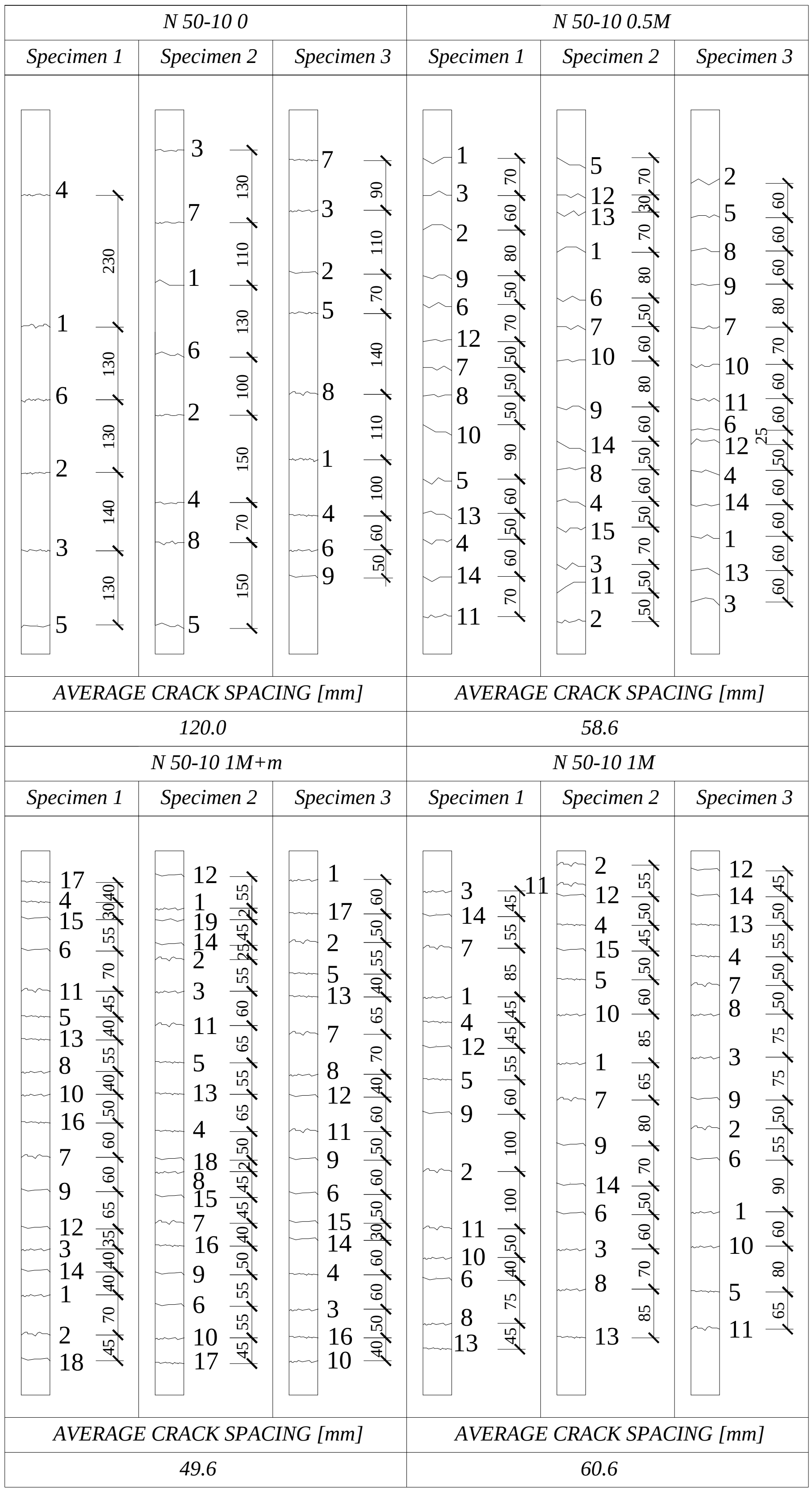


Fig. 9. Final crack patterns along tie elements for the different SFRC considered: specimens N 50/10.

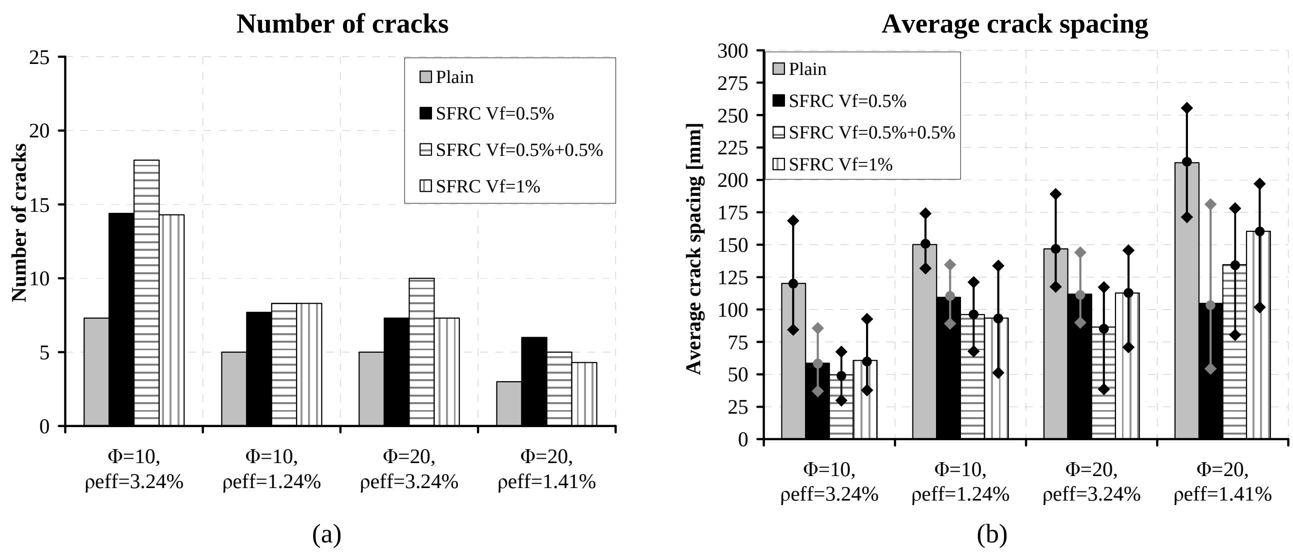


Fig. 10. Average number of cracks (a) and average crack spacing (b) evidenced during tests.

The average number of cracks is summarized in Fig. 10a; it can be observed that fibers enable a noticeable increase in the number of cracks, varying from 45% to almost 100%, and a correspondent reduction of the crack spacing, varying from 25% to 55% (Fig. 10b). By comparing specimens having V_f=1%, it can be noticed that the final crack pattern is approximately the same, even if different types of fibers have been used (macro fibers only vs. macro+micro-fibers).

On the other hand, Fig. 10b clearly evidence that the average crack spacing of SFRC samples with V_f=0.5% is only slightly higher than that of specimens with SFRCs V_f=1%. As a further comment, the minimum and maximum crack spacing (mean values obtained for each set of samples) have been plotted (as dots) in Fig. 10b, in order to evidence the experimental scatter.

Based on these preliminary test results, it seems evident that fibers are able to adequately control cracking in concrete. With this regard, in Fig. 11 the crack spacings are plotted vs. the parameter ϕ/ρ_{eff}, which is a key-parameter generally included in many building codes for the prediction of the average crack spacing (ρ_{eff} is the effective reinforcement ratio, i.e. the reinforcing area over the area of concrete in tension surrounding the reinforcement. Note that in this case, $\rho = \rho_{eff}$). The results have been compared against the formulations proposed by CEB-FIP Model 78 [29] and 90 [30], Eurocode 2 1991 [31] and 2005 [22]. It can be observed that the experimental results from plain concrete specimens are in good agreement with all the formulations, except for the Eurocode 2 (2005) model, which tends to widely overestimate the crack spacing (the concrete cover has a quite strong influence in this case).

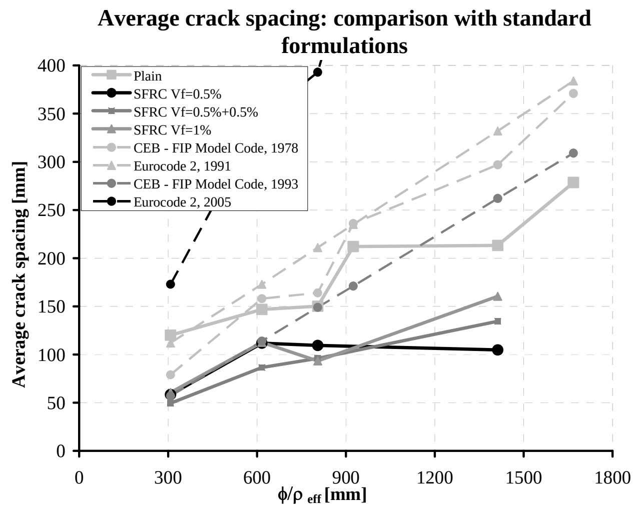


Fig. 11. Average crack spacing vs. the parameter ϕ/ρ_{eff}: comparison of the test results with some formulations proposed by standards.

Since several formulations available in literature predict the crack spacing (for classical reinforced concrete members) through a simple linear relationship depending on the parameter ϕ/ρ_{eff}, the dispersion of results has been evaluated with the linear coefficient of correlation R^2 that resulted equal to 0.89 for the control samples, 0.87 for specimens SFRC 1%M and 0.97 for 0.5%M+0.5%m samples. On the other hand, R^2 drops down to 0.35 in case of batch with V_f=0.5%. In fact, for this lower fiber content, an unexpected behavior was evidenced by specimens having higher ϕ/ρ_{eff} since the average crack spacing decreased.

CONCLUDING REMARKS

In the present paper, an experimental research work finalized to study cracking behavior of SFRC tensile members is presented. A series of 52 tension tests (tension tie tests) have been carried out at the University of Brescia as a part of a significant and broad experimental program between the Universities of Brescia (Italy) and Toronto (Canada), aiming at diffusely investigating the cracking process in reinforced concrete members with the addition of different contents and typologies of fibers.

Based on the results above mentioned, the following main conclusions might be drawn:

- FRC diffusely influences the behavior of tension-ties at Serviceability Limit States, by reducing crack width and determining a crack patterns with narrower and closely spaced cracks;
- FRC stiffens the post-cracking response of RC members, diminishing the deflections of the structures. This is also a key-point for SLS design purposes;
- mixing micro and macro fibers enhances micro-cracking control, as also depicted by notched beam tests, which show fairly higher values of residual post-cracking stresses;
- with higher reinforcement ratios, which determine narrower cracks with respect to lower reinforcement ratios, the adoption of micro fibers (in addition to macro-fibers) is slightly more effective as they are especially efficient at early-cracking stages;
- for the same type of fibers (macro), V_f=1% does not result significantly more efficient than V_f=0.5% in terms of crack spacing, especially for high reinforcement ratios. This is also due to fairly similar fracture properties;
- since the loss of durability could be significant for the life cycle cost (LCC), although a cost analysis was not performed herein, results clearly show that fiber reinforcement can enhance durability and, therefore, reduce LCC.

Further studies have to be planned and performed for better understanding the relationship between FRC toughness and reinforcement ratio and for coming up with an analytical model that could reproduce the crack spacing and tension stiffening effect in members with a broad variety of fiber contents and typologies, with special emphasis on the progression of the cracking phenomenon, which seems to be crucial in FRC elements. Moreover, more research should be devoted to the definition of a possible maximum fiber content, beyond which no improvement in the cracking process can be seen; this dosage would certainly depend on the concrete matrix.

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