Fibre-reinforced concrete in fib Model Code 2010: principles, models and test validation

In the fib Model Code for Concrete Structures 2010, fibre-reinforced concrete (FRC) is recognized as a new material for structures. This introduction will favour forthcoming structural applications because the need of adopting new design concepts and the lack of international building codes have significantly limited its use up to now. In the code, considerable effort has been devoted to introducing a material classification to standardize performance-based production and stimulate an open market for every kind of fibre, favouring the rise of a new technological player: the composite producer.

Starting from standard classification, the simple constitutive models introduced allow the designer to identify effective constitutive laws for design, trying to take into account the major contribution in terms of performance and providing good orientation for structural uses. Basic new concepts such as structural characteristic length and new factors related to fibre distribution and structural redistribution benefits are taken into account. A few examples of structural design starting from the constitutive laws identified are briefly shown.

FRC can be regarded as a special concrete characterized by a certain toughness after cracking. For this reason, the most important constitutive law introduced is the stress-crack opening response in uniaxial tension. A wide discussion of the constitutive models introduced to describe this behaviour, which controls all the main contributions of fibres for a prevailing mode I crack propagation, is proposed. The validity of the models is discussed with reference to ordinary cross-sections as well as thin-walled elements by adopting plane section or finite element models.

**Keywords:** fibre-reinforced concrete, constitutive equations, identification, modelling, structural characteristic length, structural behaviour, redundancy, structural design

### 1 Introduction

Fibre-reinforced concrete (FRC) is a composite material that is characterized by an enhanced post-cracking residual tensile strength due to the capacity of the fibres to bridge the crack faces.

During the last two decades, a wide range of research has been performed on FRC material properties, in both the fresh and hardened states [1–9]. The investigations started in the USA, driven by research into closely spaced wires and random metallic fibres [10–18]. This research was the basis for a patent on steel fibre-reinforced concrete (SFRC) based on fibre spacing in 1969 and in 1970 [19]. The Portland Cement Association (PCA) started investigating fibre reinforcement in the late 1950s. The principles of composite materials were applied to analyse FRC. The addition of fibres was shown to significantly increase toughness after the onset of the first cracking. Another patent based on bond and the aspect ratio of the fibres was granted in 1972 [19]. Since the time of these original fibres, many new steel fibres have been produced. The usefulness of SFRC was aided by other new developments in the concrete field. High-range water-reducing admixtures able to improve the workability of some harsh SFRC mixtures were formulated and break through the suppliers’ and contractors’ reservations regarding the use of SFRC. Although many experimental campaigns have been developed since the 1960s, research on the structural response of FRC elements mainly developed over the last 15 years. As a consequence, there is still a lack of international building codes for the structural design of FRC elements, even though a number of design guidelines were recently drawn up. This may explain the limited use of FRC among practitioners, who hardly accept the adoption of voluntary guidelines or, even worse, research results available in scientific papers.

FRC now appears in the fib Model Code 2010 after a huge amount of research and a historical development spanning more than 50 years. Two main reasons justify the long time needed: a theoretical aspect that forces designers to consider fracture mechanics concepts to describe the post-cracking residual strength in tension due to fibre bridging, and the technological aspects mainly related to workability and fibre alignment, which has asked concrete chemistry for new products to favour the introduction of increasingly large fibre contents in cement-based composites.

Early design considerations were produced by ACI 544 [20], and even in ACI 318 [21] some new rules were just introduced with reference to minimum shear reinforcement, while RILEM TC162-TDF produced design guidelines for typical structural elements [22, 25]. Afterwards, recommendations were produced by other countries, e.g. France [24], Sweden [25], Germany [26], Austria [27], Italy [28], Japan [29] and Spain [30].
Owing to better knowledge of FRC and the recent worldwide developments in guidelines for structural design, fib Special Activity Group 5 (SAG 5), which prepared the new fib Model Code, decided to introduce some sections on new materials [31] and in particular on FRC structure design [52]. The fib Working Groups TG 8.3 (“Fibre-reinforced concrete”) and TG 8.6 (“Ultra-high-performance fibre-reinforced concrete”) prepared these sections of the fib Model Code 2010 concerning FRC design rules to provide guidance to engineers for the proper and safe design of FRC structural elements at both serviceability and ultimate limit states, based on state-of-the-art knowledge.

In fib Model Code 2010, FRC is introduced in two sections: 5.6 and 7.7 – the former focusing on material behaviour, the latter on structural behaviour. The basic principles introduced in these two sections are mainly obtained from research on SFRC, but fib Model Code 2010 is open to every kind of fibre, following a performance-based design approach [33]. Nevertheless, several warnings are introduced regarding the long-term behaviour of some fibres, especially those characterized by a low Young’s modulus value.

This paper aims to present some basic principles governing the structural design of FRC elements made of regular concrete which were mainly introduced by fib TG 8.3. The main concepts were derived from some national guidelines for FRC structural design [24, 28] and from the guidelines proposed by RILEM TC162-TDF [22, 23]. The principles discussed here are mainly related to SFRC having a softening post-cracking behaviour in uniaxial tension (Fig. 1a), even though they can be extended to hardening materials (Fig. 1b). Since hardening behaviour in one direction is sometimes related to a softening behaviour in the orthogonal direction [34] for the known materials with aligned fibres, the two fib committees active on softening FRC materials (TG 8.3) and hardening UHPFRC materials (TG 8.6) are cooperating in the writing of two bulletins in relation to the design rules in order to favour a unified approach.

2 FRC classification

Classification is an important requirement for structural materials. When referring to ordinary concrete, designers choose the compressive strength, workability or exposition classes that have to be provided by the concrete producer.

It is well known that fibres reduce the workability of fresh concrete, but workability classes for plain concrete can be adopted for FRC as well [35]. Some studies are still needed for exposition classes since fibres may reduce the crack opening [36–38]. Therefore, for the exposition classes described in EN 206 (2006), different rules may be adopted for FRC structures (i.e. smaller concrete covers, etc.). When using FRC, compressive strength is not particularly influenced by the presence of fibres up to a content of 1 % by vol., so the classification for plain concrete can be used. As fibre content grows, the post-peak progressively increases its toughness, becoming ductile for very high fibre contents [39–41].

The mechanical property that is mainly influenced by fibres is the residual post-cracking tensile strength, and that represents an important design parameter for FRC structures. Owing to the well-known difficulties in performing uniaxial tensile tests, standard methods are generally based on bending tests on small notched beams. Since bending behaviour is markedly different from uniaxial tension behaviour, it may happen that softening materials in tension exhibit a hardening behaviour in bending ([42], Fig. 2). In fact, in bending tests, cracks arise before the peak load is reached and it may happen that softening materials in uniaxial tension exhibit stable crack propagation with increasing load (hardening behaviour in bending or flexure hardening).

The large number of parameters affecting the fibre pull-out mechanism, and consequently residual strengths, does not allow reliable prediction of FRC response in uniaxial tension based on matrix, fibre mechanical characteristics and fibre content. Experimental evidence suggests treating this cement-based material as a unique composite whose characteristics depend on fibre dispersion. The unknown fibre location and post-cracking residual strength represent the most interesting concepts for this material. If its constitutive relationships are examined assuming a simple homogeneous material, some links between the material and the related structure arise and cannot be ignored if we require reliable design predictions.
Material classification for FRC is based on the nominal properties of the composite material, referring to post-cracking tensile strength, determined from bending tests on notched prisms according to EN 14651 (2004 [43], Fig. 3); the diagram of the applied load $F$ vs. the deformation should be produced (Fig. 4). Deformation is expressed in terms of crack mouth opening displacement (CMOD) or mid-span deflection. In order to normalize the load $F$, the nominal tensile stress $\sigma_N$ in bending is considered, i.e. the bending moment $F \cdot L/4$ divided by the elastic modulus in bending, corresponding to that of the critical notched section ($W_{el} = bh_{sp}^2/6$ and $L = 500$ mm).

The classification is based on two post-cracking residual strengths at certain CMODs, which characterize the material behaviour at the serviceability limit state (SLS; CMOD$_1 = 0.5$ mm; $f_{R1k}$) and at the ultimate limit state (ULS; CMOD$_3 = 2.5$ mm; $f_{R3k}$). The latter is not introduced directly, but the $f_{R3k}/f_{R1k}$ ratio is explicitly indicated (Fig. 5). With these assumptions, an FRC material can be classified by using a couple of parameters: the first one is a number denoting the $f_{R1k}$ class, the second is a letter denoting the ratio $f_{R3k}/f_{R1k}$. The $f_{R1k}$ strength values indicating the classes are as follows:

- 1.0, 1.5, 2.0, 2.5, 3.0, 4.0, 5.0, 6.0, 7.0, 8.0 [MPa]

whereas the $f_{R3k}/f_{R1k}$ ratio is denoted by the letters a, b, c, d, e corresponding to:

- a if $0.5 < f_{R3k}/f_{R1k} \leq 0.7$
- b if $0.7 < f_{R3k}/f_{R1k} \leq 0.9$
- c if $0.9 < f_{R3k}/f_{R1k} \leq 1.1$
- d if $1.1 < f_{R3k}/f_{R1k} \leq 1.3$
- e if $1.3 < f_{R3k}/f_{R1k}$

The residual flexural tensile strength $f_{Rj}$ is defined as

$$f_{Rj} = \frac{3F_l l}{2b h_{sp}^2}$$

Fig. 3. Setup for a three-point bending test [EN 14651, 2004]

Fig. 4. Typical load $F$ vs. CMOD curve for plain concrete and FRC [fib MC2010, Fig. 5.6-6]

Fig. 5. Example of $\sigma_N$-CMOD curve with proposed classification rules
Since brittleness must be prevented in structural members, fibre reinforcement can only substitute (even partially) rebars or welded mesh at ULS if the following relationships are fulfilled:

\[ \frac{f_{R1k}}{f_{Lk}} \geq 0.4 \] (2)

\[ \frac{f_{R3k}}{f_{R1k}} \geq 0.5 \] (3)

where \( f_{Lk} \) is the characteristic value of the nominal strength, corresponding to the peak strength in bending (or the highest nominal stress value in the interval 0–0.05 mm of CMOD), determined in a notched beam test (Figs. 3 and 4).

Long-term behaviour of cracked FRC under tension has to be properly taken into account for those materials whose long-term performance is affected by creep and/or creep failure. The creep effects have not been studied enough up to now, even though some research is now in progress at several universities.

3 Constitutive laws in uniaxial tension

The stress-crack opening relationship in uniaxial tension can be regarded as the main reference material property in the post-cracking range. Two simplified stress-crack opening constitutive laws may be deduced from the bending test results: a rigid-plastic model or a linear post-cracking model (hardening or softening), as shown schematically in Fig. 6, where:

- \( w_u \) crack opening corresponding to ULS
- \( f_{Fts} \) serviceability residual strength, defined as the post-cracking strength for a crack opening significant for SLS
- \( f_{Ftu} \) residual strength significant for ULS

Both \( f_{Fts} \) and \( f_{Ftu} \) are calculated using the residual flexural strengths \( f_{R1} \) and \( f_{R3} \) identified in bending.

3.1 Kinematic model, structural characteristic length and ultimate crack opening

When considering softening materials, the definition of a stress-strain law in uniaxial tension requires the introduction of a structural characteristic length \( l_{cs} \) for the structural element. This basic concept represents a “bridge” (Fig. 7) to connect continuous mechanics, governed by stress-strain (\( \sigma\)-\( \varepsilon \)) constitutive relationships, and fracture mechanics, governed by a stress-crack opening (\( \sigma\)-\( w \)) law, initially proposed by Hillerborg (1976, [44]). The structural characteristic length is equal to the crack spacing when multiple cracking takes place and can be considered as equal to the beam depth when a plane section approach is used in the analysis.

When a finite element (FE) model is used, several approaches related to an internal length defined in relation to physical parameters, such as maximum aggregate size for non-local approaches, or element size for local approaches, can be used in order to prevent a mesh dependency of the results [45–47].

The introduction of the characteristic length allows the designer to define the strain as

\[ \varepsilon = \frac{1}{l_{cs}} \int \sigma \, dW \]

Fig. 6. Simplified constitutive laws: stress-crack opening (solid and dashed lines refer to softening and hardening materials respectively) [fib MC2010, Fig. 5.6-7]

Fig. 7. Examples of characteristic lengths
The ultimate crack width $w_u$ can be defined on the basis of a ductility requirement and therefore as

$$ w_u = l_{cs} \cdot \varepsilon_{Fu} $$

(5)

by assuming $\varepsilon_{Fu}$ equal to 2 % for a neutral axis crossing the cross-section and 1 % for a neutral axis external to the cross-section. Moreover, a limited value strictly correlated to the fibre length should also be considered. In relation to the actual market availability and practical considerations regarding RC crack opening observed at ULS, a value of 2.5 mm is assumed. Eq. (5) can be rewritten as

$$ w_u = \min \left( l_{cs} \varepsilon_{Fu}, 2.5 \text{ mm} \right) $$

(6)

Multiple cracking occurs in hardening materials. Therefore, the identification of crack openings is not necessary because a conventional stress-strain law may be directly determined by a uniaxial tension test, typically unnotched (like a dog-bone specimen), by dividing the relative displacement by the gauge length. It is interesting to note that sometimes the same self-compacting concrete can exhibit a softening and a hardening behaviour depending on the stretching direction with reference to fibre alignment [34]. This is one of the main reasons why standards have to model FRC material behaviour following a similar approach for both hardening and softening materials. Moreover, every time fibres are aligned, the material cannot be considered isotropic, and suitable anisotropic constitutive laws should be introduced.

### 3.2 The $\sigma$-$w$ curve identified from bending tests

Accepting the Hillerborg idea of a cohesive approach to describe the uniaxial tension behaviour of FRC [44, 48], two possible simplified models can be introduced to describe FRC response after cracking. This is done by emphasizing that the most significant effect induced by fibres is related to the pull-out mechanism: the rigid-plastic and the linear elastic-softening models.

The rigid-plastic model requires the identification of only one parameter: $f_{Fu}$. It can be easily identified by rotational equilibrium at ULS by assuming that the compressive stress distribution is concentrated at the top fibre, whereas a tensile post-cracking residual stress distribution is uniformly applied to the overall critical cross-section (Fig. 8).

By equating the internal moment of resistance $M_{u,\text{int}}$ to the external applied moment $M_{u,\text{ext}}$, it is possible to write the following equation, which corresponds to the rotational equilibrium of the cross-section:

$$ M_{u,\text{ext}} = \frac{f_{R3} b h^2}{6} = \frac{f_{Tu} b h^2}{2} = M_{u,\text{int}} $$

(7)

$$ f_{Fu} = \frac{f_{R3}}{3} $$

(8)

The linear model identifies two reference values: $f_{Fts}$ and $f_{Ftu}$. They can be defined by residual values of flexural strengths by using the following equations:

$$ f_{Fts} = 0.45 f_{R1} $$

(9)

$$ f_{Ftu} = f_{Fts} - \frac{w_u}{CMOD} \left( f_{Fts} - 0.5 f_{R3} + 0.2 f_{R1} \right) \geq 0 $$

(10)

where $w_u$ is the maximum crack opening accepted in structural design.

The two equations are introduced according to different assumptions valid at SLS and ULS respectively and briefly summarized in Fig. 9. At SLS the constitutive relationship for FRC is assumed to be elastoplastic in uniaxial tension and elastic in uniaxial compression. Two equations can be written to impose longitudinal and rotational equilibrium. According to the notation used in Fig. 9, and assuming

$$ \sigma = \frac{w}{y l_{cs}} x; w_s = 0.5 \text{ mm}; y = h_{sp} - x; l_{cs} = h_{sp} $$

Fig. 8. Simplified model adopted to compute the ultimate residual tensile strength in uniaxial tension $f_{fu}$ by means of the residual nominal bending strength $f_{fu}$ (fib MC2010, Fig. 5.6-8)

Fig. 9. Stress distributions assumed for determining the residual tensile strength $f_{Fts}$ (b) and $f_{Ftu}$ (c) for the linear model (fib MC2010, Fig. 5.6-9)
then the result is

\[
\begin{align*}
\frac{\sigma b x}{2} - \frac{f_{R3} b}{2} y - \frac{f_{R3} b}{2} x = 0 \\
\frac{f_{R1} b y}{2} + \left( \frac{2}{3} x + \frac{y}{2} \right) - \frac{f_{R3} b}{2} x + \frac{1}{3} f_{R3} x = \frac{f_{R1} b h_{sp}^2}{6}
\end{align*}
\]  

(11)  

(12)

The resolution of the non-linear system of Eqs. (11) and (12) gives a correlation between \( f_{R_{ts}} \) and \( f_{R1} \) that depends on Young’s modulus \( E \) and on the choice of the structural characteristic length \( l_{cs} \):

\[ f_{R_{ts}} = k \left( E, l_{cs} \right) f_{R1} \]  

(13)

By assuming a structural characteristic length \( l_{cs} \) equal to the critical cross-section \( h_{sp} \), the factor \( k \) changes with Young’s modulus \( E \), as shown in Fig. 10. The \( k \) value ranges between 0.362 and 0.378 and therefore an average value of 0.37 could be considered.

Let us consider the ULS. In this case the compressive stress distribution is concentrated on a very small portion of the cross-section and therefore, once again, it is assumed that a concentrated compressive force acts at the upper fibre. If at the underside a crack opening of 2.5 mm is considered, a linear softening model involves a linear distribution of stresses that is characterized by a value \( k_b f_{R1} \) for \( w = 0 \), \( k_d f_{R1} \) for \( w_{i1} = 0.5 \) mm and \( f_{R_{2.5}} \) for \( w_{i2} = 2.5 \) mm. The rotational equilibrium becomes

\[ f_{R_{ts},2.5} = \frac{b h_{sp}^2}{2} + \left( k_b f_{R1} - f_{R_{ts},2.5} \right) \frac{b h_{sp}^2}{3} = f_{R3} f_{R3} \frac{b h_{sp}^2}{6} \]  

(14)

By solving Eq. (14), it is possible to compute the unknown \( f_{R_{ts},2.5} \) as

\[ f_{R_{ts},2.5} = 0.5 f_{R3} - \frac{k_b f_{R1}}{2} f_{R1} \]  

(15)

Now, taking into account that the linear model also has to fit for the point \( (w_{i1} = 0.5 \text{ mm}, \sigma = 0.37 f_{R1}) \), it is possible to express \( k_b \) as follows:

\[ k_b = 0.529 - 0.143 \frac{f_{R3}}{f_{R1}} \]  

(16)

If \( f_{R3} \) is considered to be equal to \( 0.5 f_{R1} \), then that represents the constraint introduced to define the composite FRC as a structural composite; \( k_b \) becomes equal to 0.45 and Eq. (15) can be rewritten as

\[ f_{R_{ts},2.5} = 0.5 f_{R3} - 0.225 f_{R1} = 0.5 f_{R3} - 0.2 f_{R1} \]  

(17)

Taking into account the definition of the ultimate crack opening \( w_u \) previously introduced, and in particular that introduced for thin-walled elements, a reduced crack opening value can be considered and Eqs. (9) and (10) deduced. It is important to note that the shifting of the \( f_{R_{ts}} \)
value at \( w = 0 \) prevents some spurious situations where a class reduction (i.e. FRC composite changes from class b to a, or c to b) could involve a better performance in bending (see Fig. 11) for thin-walled elements where \( w_u \) is close to 0.5 mm.

Finally, it is worth noting that a CMOD value of 2.5 mm does not correspond to a CTOD value of 2.5 mm (see Eq. (18)), but rather to a CTOD close to 2.1 mm according to a rigid-body assumption. Nevertheless, the idea to arrest the cohesive stress at \( w_u \) is in this way partially compensated by the crack opening translation.

### 3.3 Three- vs. four-point bending test

In the literature, the three-point bending test is not the only notched test proposed. The debate on the best test for identifying the uniaxial tension constitutive law \( \sigma - w \) has involved many researchers and is still in progress [49, 50]. It is interesting to observe that the difference between the two tests is not so large when a careful non-linear computation is carried out. According to Ferrara and di Prisco [51], for plain concrete, the main differences between the two tests are related to the peak strength and the first post-peak slope of the load vs. crack opening displacement (COD), whose behaviour is dominated by matrix response. A three-point bending test exhibits a weak higher peak strength and a more brittle first post-peak slope. Of course, there is also a significant difference in the queue value for a large COD, but this is due to the confinement in compression obtained in three-point bending tests just below the central load knife. However, this significant effect becomes negligible when a fibre pull-out mechanism takes place. Several SFRC materials denoted according to a unified system (Tables 1 and 2) have been compared by investigating their bending behaviour according to UNI 11039 [52] and EN 14651 [43]. The designation is expressed by \( M_n-F_n-V_f \), where \( M_n \) indicates a certain matrix (\( n \) ranges between 1 and 8, Table 1), \( F_n \) indicates a certain steel fibre (\( n \) ranges between 1 and 8, Table 2) and \( V_f \) indicates the fibre volume percentage used in the composite. The experimental comparison highlights what was already predicted in [51], showing very similar pull-out strengths, with the tendency to have something more in three-point bending test (Fig. 12). It is important to emphasize that the measurement of the crack opening is not the same for the two tests. In fact, whereas for a three-point bending test the measured parameter is C\( \text{MOD} \), for the four-point bending test the measured parameter is CTOD. If a linear

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<td>620</td>
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h.e. = hooked end; c. = crimped
Fig. 12. Three- vs. four-point bending tests: experimental responses for several materials
opening of the crack edges is assumed, then that is usually accepted for large crack openings, a rough upper-bound linear relation between the two COD measures can be introduced as

$$\text{COD} \approx \frac{150}{125} \cdot \text{CTOD} \approx 1.2 \cdot \text{CTOD}$$ (18)

### 3.4 Refined constitutive relationships

Once the linear stress-crack opening relationship has been identified, the stress-strain relationship can be deduced by introducing the suitable structural characteristic length $l_{cs}$ with reference to softening materials (Fig. 7, [53]).

At SLS, a more refined curve able to fit the peak strength of the matrix can be proposed and it is particularly suggested for FE analyses. The same constitutive relationship adopted for plain concrete in uniaxial tension can be used up to peak strength $f_{ct}$, while in the post-cracking stage, a bilinear relation applies (Fig. 13). The residual strength due to the pull-out mechanism (final branch), which represents the main fibre contribution, is defined by two points corresponding to $(e_{SLS}, f_{FB}) = k_a f_{R1}$ and $(e_{ULS}, f_{ftU})$. This simplification, which is easily understood because it takes into account only the most significant fibre contribution, the pull-out effect, may cause the same problem previously discussed in Fig. 11. For this reason, especially when the mechanical behaviour is strongly affected by small crack openings as for statically redundant structures, the first point, corresponding to SLS, can be shifted on the first softening branch conserving a value equal to $k_a f_{R1}$.

When the identification procedure is carried out starting from a four-point bending notched test, coefficient $k_a$ (Eq. (13)) can be set equal to a different value depending on the specific crack opening range adopted in the standard. If UNI 11039 is considered, a value of 0.59 can be adopted to take into account that the SLS value covers a range between 0 and 0.6 mm of crack tip opening displacement (CTOD), which corresponds to an average value of 0.3 mm instead of roughly 0.41 mm as occurs in EN 14651, where the CMOD value is 0.5 mm. In this case, in Eqs. (9) and (10) the residual strengths $f_{Ri}$ are substituted by $f_{eqi}$ because they are computed as average values in certain CTOD ranges: 0–0.6 mm for SLS and 0.6–3.0 mm for SLU.

In *fib* Model Code 2010, the constitutive relationships for softening materials (Fig. 13) are presented together with those suggested for hardening materials, where, progressively, the matrix contribution cannot be distinguished from the fibre one. Only the softening case is discussed in the following. Further details on hardening material responses will be discussed in an *fib* bulletin currently in preparation.

Note that by introducing the peak strength of the matrix it is possible to better induce localization when an FE model is adopted, thus preventing spurious dissipation due to uncontrolled growths of the cracked band [54].

Finally, the introduction of the fracture energy corresponding to the matrix peak strength contribution is not considered in the equilibrium equations used to identify Eqs. (9) and (10), but its addition is not generally significant for typical structures.

### 3.5 The $\sigma$-$w$ curve identification: theoretical vs. experimental results

A broad experimental campaign was planned in order to ascertain whether the constitutive model $\sigma$-$w$ identified from notched bending tests is reliable. A comparison was
first carried out by casting three specimens made with the same FRC, belonging to the same batch, and carrying out three four-point bending tests according to Italian standard UNI 11039 [52]. Three cylindrical specimens were then core-drilled from one undamaged end (Fig. 14a) and tested in uniaxial tension by means of a closed-loop press able to impose a fixed-end condition [55]. The comparison is shown in Fig. 14, and is really encouraging. In fact, the scattering of the results was very much reduced, not only between the LVDTs at 120° in the same test [55], but also in the three different tests (see small shadow in Fig. 14b). The dashed line shown is computed as the average between the three average responses, computed as the mean curve of the three transducers in each test. Other tests were performed after thermal cycles at different temperatures (T = 200, 400 and 600 °C). Similar comparisons were also carried out on prismatic specimens in order to compare bending tests on notched and unnotched specimens with uniaxial tension tests with fixed and rotating ends made from the same materials [61].

The uniaxial tension test was also simulated by means of Diana Finite Element code, introducing as the constitutive relationship the $\sigma-w$ response identified from bending (Fig. 15) using Eqs. (9) and (10). The axisymmetric mesh (Fig. 16) adopts regular triangular elements [54] and the COD was computed, as in the experimental tests, as the relative displacement between two points at a distance of 50 mm astride the notch. The structural characteristic length in this case is a localization limiter with $l_{cs} = \sqrt{2A}$ [56]. The results (Fig. 17) are obtained by changing the size of the elements ($l_{cs} = 1.163, 2.327$ mm). The higher curve is the constitutive response identified and used as an input for FE modelling, whereas the two FE structural responses of the modelled specimens are practically coincident and a little closer to the average experimental curve measured with the LVDTs at 120°. The main difference is shown in the post-peak response, where a smaller dissipation should be related to the modelling as-
Using the same constitutive relationship, it is also possible to model the original notched beam specimen tested in a four-point bending setup. Two meshes were introduced: they consider a plane stress approach and quadrilateral elements (mesh 1, Fig. 18, [54]) or triangular elements (mesh 2; Fig. 18, [54]). Even if the same constitutive relationship $\sigma$-$\omega$ considered in the uniaxial tension tests was assumed and a proper localization limiter was applied as suggested by Rots [56] for both meshes, different post-cracking responses were obtained. In Fig. 19 the FE results are compared in terms of nominal strength $\sigma_N$ vs. CTOD with experimental values from the four-point bending tests. It should be noted that triangular elements introduce a non-negligible increase in toughness.
The simplified models introduced can sometimes be substituted by more complex models identified by means of back-analysis [57]. With reference to mesh 1 (Fig. 18, [54]), a trilateral softening curve can be introduced as shown in Fig. 20a to reproduce the uniaxial tension test better. The improvement involves a more careful fitting of the bending response (Fig. 20b). Incidentally, a progressive improvement in the identification process is not always associated with the best response if the back-analysis is carried out with reference to a different kinematic model. In Fig. 21a, a multi-linear $\sigma$-$\varepsilon$ curve identified by means of a plane section model can allow the designer to fit perfectly the response with a plane section approach (Fig. 21b), but the result of the FE analysis with the same constitutive law could be far from the ideal fitting (Fig. 21b).

Moving to the use of a plane section model, which is much more efficient for the structural design of bent elements, a series of beam specimens made of different materials as defined in Tables 1 and 2 was made to check the reliability of the proposed linear model (le/ls: linear elastic pre-peak/linear softening). The same constitutive models were adopted in the FE analysis with the proper structural characteristic length by using the two meshes shown in Fig. 18. The results (Fig. 22) confirm good reliability for a plane section model and FE mesh 1, whereas FE mesh 2 always gives an over-resistant response. Of course, the fitting is reasonable, it often overestimates the peak strength and the stiffness before the peak. A comparison between linear and bilinear relationships at pre- and post-peak is also shown (Fig. 23) with reference to the plane section and FE models. The main difference for both the kinematic models appears in the peak region as expected: linear relationships exhibit lower peak strengths due to the lack of matrix contribution and the same pull-out strength.

A final investigation in terms of good fitting can be performed by considering the choice of the structural...
characteristic length. In fact, this variable is not only related to the ideal condition of the plane section kinematic constraint, but also plays a role in predicting pre-peak behaviour and peak strength. By examining different choices of $l_{cs}$ and two different materials with a linear elastic model in pre-peak and a bilinear softening model ($l_e/bs$) in post-peak, it is possible to highlight the role played by the structural characteristic length. Fig. 24 clearly highlights the value of $l_{cs} = h$ suggested in fib Model Code 2010 as a reasonable choice.

4 The “structural” specimen

The identification of the uniaxial tension constitutive law in the post-cracking regime is severely affected by fibre distribution and other parameters as observed by many researchers [58]. FRC is a cementitious composite and therefore, to be regarded as a homogeneous material, the specimens used to characterize its behaviour should have a volume that can be representative of the FRC heterogeneity grade. Changing casting and handling procedures
as well as the mixer, as necessitated by larger cast volumes, can drastically change mechanical characteristics in bending [58]. In thin-walled elements the use of fibre reinforcement is quite appropriate but, in general, FRC should present a hardening response in bending. The representativeness of standard notched beam specimens, with a cross-section of $150 \times 150$ mm, for characterizing the behaviour of this type of element is questionable. At the same time, fibre dispersion and orientation are seriously affected by a casting procedure that, in thin elements, is different from that adopted in standard specimens. Furthermore, standard specimens are notched and it is difficult to guarantee the hardening behaviour of the material (in statistical terms) by their response. In fact, the notch favours stable crack propagation and significantly modifies the first cracking process, especially if the fibre content is not high enough to change the mechanical behaviour at peak [59–61]. For all these reasons, an unnotched prismatic specimen cast using the same procedure as for the structure and with the same thickness is preferred. The specimen can be tested by means of a four-point bending setup that favours a crack propagation starting from the weakest cross-section between the load points. A transducer attached between two points on the underside of the specimen measures the relative displacement between these points (COD). This type of thin specimen can be very representative of the behaviour of thin FRC structures; therefore, it can be named “structural” specimen as first suggested in the French guidelines on ultra-high-performance fibre-reinforced concrete AFGC-SETRA [24] and in the Italian guidelines on SFRC [62]. In Fig. 25 a set of five different material tests highlights as the structural specimen usually gives a weaker response in relation to the standard notched tests used for classifying the material, even if a higher performance could be expected for the most favourable fibre distribution. For this reason, when thin-walled elements have to be designed safely, a careful identification of the mechanical response by means of structural specimens is strongly suggested. This choice could allow a partial reduction in the safety factors as suggested in [28, 58].

5  Reliability of structural behaviour prediction

The constitutive models introduced to describe uniaxial tension can also be used to check the behaviour of beams with a conventional cross-section. To this end, three beams 3 m long with a $300 \times 300$ mm square cross-section were tested in a four-point bending test setup to check the reliability of the approach proposed in fib Model Code 2010. No conventional reinforcement was introduced [63, 64, 54]. The beam geometry and the setup adopted are shown in Fig. 26. The material is M2-F4-0.62, as specified in Tables 1 and 2.

According to the measurements indicated in the setup (Fig. 29), both the load vs. vertical displacement and the bending moment vs. measured central curvature are proposed in Figs. 27a,b. Careful measurement of the fibre numbers in the critical cross-sections was also carried out and is indicated in Fig. 28. Fig. 29 also shows the final crack patterns. The numerical prediction carried out using the plane section approach fits quite well with the mechanical response measured experimentally. It is important to emphasize that the comparison is performed by considering the av-

![Fig. 26. Geometry and test setup for full-scale beam test](image-url)
verage curves in the identification process of the uniaxial tension law and not the characteristic ones. The smaller stiffness of the global curve (Fig. 27b) is due to the lack of localization and to a softening zone assumed to be spread homogeneously over the overall zone with the same maximum bending moment. FE analyses were also performed using the same constitutive relationship and quadrilateral elements. In this case average and characteristic curves fit the structural behaviour quite well, even if the pre-peak response simulated by the FE analysis is not able to take into account the defect distributions adequately as well as inhomogeneous shrinkage effects in the cross section, thus exhibiting a stiffer behaviour. In order to favour the localization in the critical section detected experimentally, an initial geometric defect was introduced by assigning a local width reduction <10% in the proximity of the main crack propagation.

To conclude the modelling of unreinforced concrete structures, an example of the role played by the structural characteristic length is shown in Fig. 30, where the same uniaxial tension constitutive law is assumed and different depths are considered [50]. The structural characteristic length is therefore able to reproduce a significant size effect without the need for any special coefficient as proposed by Rilem TC 162 TDF [23].

The same constitutive relationships were also used to model the bending behaviour of prefabricated FRC roof elements, where only prestressed longitudinal reinforcement remained – all the transverse reinforcement was substituted by different types of steel fibre. Further details for these cases can be found in [65, 66].

Several research projects are in progress to check the global behaviour of R/C structures where an FRC composite is adopted. In section 7.7, the models discussed to reproduce the uniaxial tension behaviour are used coupled with conventional reinforcement. The bending moment resistance at ULS with longitudinal reinforcement, which
represents the usual case, can be investigated by means of the addition of a fibre contribution as clearly shown in Fig. 31. The bending failure stage is supposed to be reached when one of the following conditions applies:

- attainment of the ultimate compressive strain in the FRC, $\varepsilon_{cu}$
- attainment of the ultimate tensile strain in the steel (if present), $\varepsilon_{su}$
- attainment of the ultimate tensile strain in the FRC, $\varepsilon_{Fu}$

It is important to emphasize that in this case the structural characteristic length usually depends on the crack distance and therefore on the reinforcement ratio and bar diameters used. Furthermore, fibres help to increase the ductility of the plastic hinge by increasing the passive confinement in the compression zone, but this effect is not taken into account.

Several investigations are in progress to check the effectiveness of fibres in reducing crack distance and openings as suggested in *fib* Model Code 2010, with suitable relationships in agreement with conventional reinforced concrete [36, 37, 38].

### 6 Partial safety factors and redundancy coefficients

For ultimate limit states, recommended values of partial safety factors $\gamma_F$ are shown in Table 3. For serviceability limit states, the partial factors should be taken as 1.0. However, some special observations have to be made. FRC is scantly homogeneous and isotropic because fibres location is random and mainly depends on casting procedure, formwork geometry and mix consistency affected by flowability, viscosity and filling ability. Therefore, the scattering of its response mainly depends on numbers of fibres in the cracked section, their location and their orientation. On the basis of the previous considerations, the choice of the safety factors should take into account the following:

- the representativeness of the specimens used to characterize the mechanical response of the material, in relation to the structure considered
- the number of specimens for mechanical characterization
- the stress redistribution capacity of the structure under consideration
- the fracture volume involved in the failure mechanism

Besides the safety factor indicated in Table 3, suitable coefficients $K$, which take into account the representativeness of the specimen used for the identification in relation to the structure and the casting procedure adopted, are also introduced. In general, an isotropic fibre distribution is assumed so that the fibre orientation factor $K$ is equal to 1. For favourable effects, an orientation factor $K < 1.0$ may be applied if verified experimentally. For unfavourable effects, an orientation factor $K > 1.0$ must be verified experimentally and applied. The values $f_{Ftsd}$ and $f_{Ftud}$ should then be modified to

$$f_{Ftsd,mod} = \frac{f_{Ftsd}}{K}$$

$$f_{Ftud,mod} = \frac{f_{Ftud}}{K}$$

A careful analysis of the role played by the safety factor when a non-linear mechanical analysis is carried out according to *fib* Model Code 2010 is described by Cervenka et al. [67].

### 7 Basic aspects for design

Fibre reinforcement is suitable for structures where diffused stresses are present. In structures with both localized and diffused stresses, which is the usual case, it is better to base the reinforcement on a combination of rebars and fibre reinforcement.

In structural elements where fibres aim to replace conventional reinforcement (even partially), some restrictions on the minimum residual strength are applied (Eqs. (2) and (3)). This residual strength becomes significant in structures characterized by a high degree of redundancy, where a remarkable stress redistribution occurs. For this reason, in structures without rebars, where fibres completely replace conventional reinforcement, a minimum redundancy level is required for the structural member.
On the contrary, in structures with rebars, where fibres constitute additional reinforcement, ductility is generally provided by conventional reinforcement that makes a major contribution to the tensile strength. For hardening FRCs (in uniaxial tension), fibres can be used as the only reinforcement (without rebars), also in statically determinate structural elements. The heterogeneity of the mechanical behaviour in the post-cracking regime is often significantly penalized due to the high scattering mainly related to fibre distribution and orientation. When a significant redundancy is guaranteed for the structure by its geometry and its boundary conditions, and a large volume of the structure is involved in the failure process, the experimental investigation has highlighted that the average mechanical behaviour – rather than the characteristic one – takes place. For this reason, a suitable coefficient $K_{Rd}$, aimed at increasing the load bearing capacity of the structure, is introduced [54].

Section 7.7, after a rough classification, also introduces semi-empirical equations for designing FRC structural elements when subjected to shear according to the multi-level approach [68], punching [69] and torsion. There are even suitable equations for slabs and walls as well as specific equations to compute crack distance and crack opening taking into account fibre contribution, which thus allows us to design new FRC structures according to these principles.

8 Concluding remarks

The implementation of fibre reinforced concrete (FRC) in the fib Model Code 2010 is a very important milestone. In

### Table 4. Experimental results of four-point bending tests

<table>
<thead>
<tr>
<th>Material</th>
<th>Specimen No.</th>
<th>$f_{ld,av}$ (std %) [MPa]</th>
<th>$f_{eq,1,av}$ (std %) [MPa]</th>
<th>$f_{eq,2,av}$ (std %) [MPa]</th>
</tr>
</thead>
<tbody>
<tr>
<td>M1-F1-0.62</td>
<td>3</td>
<td>4.9 (12.1 %)</td>
<td>7.43 (19.6 %)</td>
<td>8.11 (23.7 %)</td>
</tr>
<tr>
<td>M1-F2-0.62</td>
<td>3</td>
<td>5.15 (4.9 %)</td>
<td>7.02 (19.6 %)</td>
<td>6.16 (5.2 %)</td>
</tr>
<tr>
<td>M1-F3-0.32-F4-0.32</td>
<td>3</td>
<td>5.15 (3.6 %)</td>
<td>6.92 (4.68 %)</td>
<td>5.89 (16.5 %)</td>
</tr>
<tr>
<td>M2-F4-0.62</td>
<td>9</td>
<td>5.94 (10.1 %)</td>
<td>8.39 (6.3 %)</td>
<td>4.87 (15.6 %)</td>
</tr>
<tr>
<td>M3-F2-0.62</td>
<td>3</td>
<td>5.79 (1.1 %)</td>
<td>5.34 (8.5 %)</td>
<td>3.91 (15.8 %)</td>
</tr>
<tr>
<td>M4-F2-0.62</td>
<td>7</td>
<td>5.02 (7.9 %)</td>
<td>6.44 (18.0 %)</td>
<td>6.27 (17.0 %)</td>
</tr>
<tr>
<td>M5-F5-0.45</td>
<td>8</td>
<td>3.54 (10.7 %)</td>
<td>2.91 (20.6 %)</td>
<td>2.69 (36.6 %)</td>
</tr>
<tr>
<td>M6-F6-0.45</td>
<td>6</td>
<td>3.13 (11.4 %)</td>
<td>1.47 (31.2 %)</td>
<td>0.73 (54.5 %)</td>
</tr>
<tr>
<td>M6-F6-0.83</td>
<td>6</td>
<td>3.36 (11.6 %)</td>
<td>2.10 (10.1 %)</td>
<td>1.33 (13.2 %)</td>
</tr>
<tr>
<td>M6-F7-0.45</td>
<td>6</td>
<td>2.84 (10.8 %)</td>
<td>1.88 (21.4 %)</td>
<td>1.26 (37.6 %)</td>
</tr>
<tr>
<td>M6-F7-0.83</td>
<td>6</td>
<td>3.70 (11.0 %)</td>
<td>3.52 (27.1 %)</td>
<td>3.18 (39.0 %)</td>
</tr>
<tr>
<td>M6-F8-0.45</td>
<td>6</td>
<td>2.49 (19.8 %)</td>
<td>1.81 (42.0 %)</td>
<td>1.53 (65.9 %)</td>
</tr>
<tr>
<td>M6-F8-0.83</td>
<td>6</td>
<td>2.90 (19.8 %)</td>
<td>2.46 (18.0 %)</td>
<td>2.51 (18.8 %)</td>
</tr>
<tr>
<td>M7-F2-0.62</td>
<td>8</td>
<td>4.01 (10.3 %)</td>
<td>3.19 (22.2 %)</td>
<td>2.03 (32.5 %)</td>
</tr>
<tr>
<td>M8-F2-0.62</td>
<td>6</td>
<td>6.84 (5.8 %)</td>
<td>8.45 (19.5 %)</td>
<td>3.87 (28.4 %)</td>
</tr>
<tr>
<td>M8-F1-0.62</td>
<td>6</td>
<td>6.76 (7.1 %)</td>
<td>9.80 (12.2 %)</td>
<td>9.22 (14.8 %)</td>
</tr>
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</table>

### Table 5. Experimental results of three-point bending tests

<table>
<thead>
<tr>
<th>Material</th>
<th>Specimen No.</th>
<th>$f_{dl,av}$ (std %) [MPa]</th>
<th>$f_{R1,av}$ (std %) [MPa]</th>
<th>$f_{R3,av}$ (std %) [MPa]</th>
</tr>
</thead>
<tbody>
<tr>
<td>M6-F6-0.45</td>
<td>6</td>
<td>3.54 (9.6 %)</td>
<td>1.30 (23.0 %)</td>
<td>0.68 (42.6 %)</td>
</tr>
<tr>
<td>M6-F6-0.83</td>
<td>6</td>
<td>4.12 (4.35 %)</td>
<td>2.07 (16.8 %)</td>
<td>1.41 (20.8 %)</td>
</tr>
<tr>
<td>M6-F7-0.45</td>
<td>6</td>
<td>3.41 (14.3 %)</td>
<td>1.42 (15.5 %)</td>
<td>1.21 (11.4 %)</td>
</tr>
<tr>
<td>M6-F7-0.83</td>
<td>6</td>
<td>3.95 (20.7 %)</td>
<td>2.54 (38.5 %)</td>
<td>2.31 (40.9 %)</td>
</tr>
<tr>
<td>M6-F8-0.45</td>
<td>6</td>
<td>3.20 (7.5 %)</td>
<td>1.92 (27.1 %)</td>
<td>1.97 (38.9 %)</td>
</tr>
<tr>
<td>M6-F8-0.83</td>
<td>6</td>
<td>4.15 (10.2 %)</td>
<td>2.90 (28.5 %)</td>
<td>3.21 (30.2 %)</td>
</tr>
<tr>
<td>M7-F2-0.62</td>
<td>9</td>
<td>5.06 (13.8 %)</td>
<td>2.53 (30.6 %)</td>
<td>1.86 (35.1 %)</td>
</tr>
</tbody>
</table>
the near future it will probably lead to the development of structural rules for FRC elements in Eurocodes and national codes.

This paper has carefully discussed the simplified models suggested for the composite and presented them in the material section to evaluate the uniaxial tension residual strength, mainly given by fibre pull-out. Their reliability as well as their limitations are indicated with reference to several FRC materials, characterized by different matrices, different steel fibres and different fibre contents. The design rules are derived from a unified classification of FRC composite based on a three-point bending test, already accepted as a European standard.

The identification of the constitutive law is also discussed with reference to the structural characteristic length concept and two different kinematic models that can be adopted: plane section and finite element approaches. In all the cases discussed, the procedure fits the experimental tests reasonably well, thus showing an appreciable robustness of the whole design approach.

For thin-walled elements, the need for a “structural” specimen to identify the material properties better, taking into account the real casting procedure, is also highlighted with reference to various FRC composites. This requirement becomes essential in the case of self-compacting materials too. Suitable coefficients to take into account inhomogeneous fibre alignments are also introduced. Due to the high scatter of FRC responses, a new coefficient able to consider the beneficial effect of redundancy is also introduced.

It is worth noting that although the level of knowledge of FRC has increased tremendously over the last 15 years, further research is needed to verify and optimize the proposed design rules, to investigate the long-term behaviour of different FRCs and other open issues such as the anisotropic behaviour of FRC, fatigue and multi-axial mechanical behaviour. A new generation of FRCs will soon enter the market. They are based on a “cocktail” of different fibre types (material and/or geometry) to enhance different structural performance aspects and fib Model Code 2010 should be ready to open up the way for their usage.

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A special vote of thanks goes to professors L. Vandewalle and G. Plizzari for the excellent cooperation to Prof. J. Wallraven for the fruitful discussions and to Prof. H. Falkner, who shared with us his considerable design experience. The authors are also indebted to all the members of fib Task Groups TG 8.3 and TG 8.6 for the constructive discussions during the several meetings, where many ideas presented in this paper took a definitive shape.

References

20. ACI Committee 544: Design considerations for steel Fiber Reinforced Concrete, ACI 544.4R-88. American Concrete Institute, ACI Farmington Hills, MI, 1996.


