DESIGN METHOD FOR STEEL FIBER REINFORCED CONCRETE PROPOSED BY RILEM TC 162-TDF

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Abstract.
This paper describes two design methods, proposed by Rilem TC 162 "Test and Design Methods for Steel Fiber Reinforced Concrete", i.e. the σ-ε-design method on the one hand and the σ-w-method on the other. In the former design method the behavior of steel fiber reinforced concrete in tension is characterized by means of a stress-strain relation and this method is based on Eurocode 2. However, in the latter design method a stress-crack opening relationship is used to describe the postcracking behavior of steel fiber reinforced concrete and this method is based on the fictitious crack model according to Hillerborg.

1. Introduction

After more than 30 years of research and developments, steel fiber reinforced concrete (SFRC) is unfortunately not truly used in structural elements. One of the major obstacles identified preventing a widespread structural use of SFRC is the lack op validated design and test methods.

Therefore a Rilem Technical Committee, i.e. TC 162-TDF (Test and Design Methods for Steel Fiber Reinforced Concrete) has been set up in April 1995. The objectives of TC 162-TDF are :

- to set up a design method for steel fiber reinforced concrete
- to make recommendations for suitable test methods.

From the beginning it was decided that both items should be treated simultaneously because they are interrelated.

The Technical Committee consists of two subgroups, i.e. one group describes the postcracking behavior of SFRC by means of a stress-strain diagram (σ-ε) and the other by using a stress-crack opening relation (σ-w).

This paper mainly describes the work done by the σ-ε-group since the "design method" proposed by the σ-w-group still has to be discussed within the Technical Committee.

Two draft recommendations have already been published in the Rilem-journal "Materials and Structures", i.e. one regarding the "BENDING TEST" [1], necessary to determine the σ-ε-diagram in tension and the other concerns the "σ-ε-DESIGN METHOD" [2].

The target date for completion of the Committee's work is 2000.

2. σ-ε-Design method

The objective of the σ-ε-group was to propose a design method [2] which fulfills the following requirements :

- it should be simple enough so that it can be used by a structural engineer for practical applications
- it should be compatible with the present design regulations so that it is possible to combine it with the available calculation methods for reinforced and prestressed concrete
- it should make optimum use of the postcracking behavior of SFRC.

The European pre-standard ENV 1992-1-1 (Eurocode 2 : Design of Concrete Structures - Part 1 : General rules and rules for buildings) [3] has been used as a general framework for this design method proposed. The calculation guidelines are valid for SFRC with compressive strengths of up to C50/60. Steel fibers can also be used in high strength concrete, i.e. concrete with $f_{fck} \geq 50$ N/mm^2. However, care should be taken that the steel fibers do not break in a brittle way before being pulled out.

Since Eurocode 2 takes only pre-peak behavior of concrete in tension into account and due to the fact that primarily the post-peak behavior is affected by the presence of steel fibers, a σ-ε-relation which describes the postcracking behavior of SFRC has to be laid down.

In order to determine the parameters, which are used to characterize the postcracking behavior of SFRC, experimentally, displacement controlled three-point bending tests [1] are conducted on notched prisms.

2.1 Bending test

Concrete prisms of 150 x 150 mm^2 cross section with a minimum length of 550 mm are used as standard test specimens. These specimens are not intended for concrete with steel fibers longer than 60 mm and aggregate larger than 32 mm.

The beams are notched using a saw. Each beam is turned 90° from the casting surface and

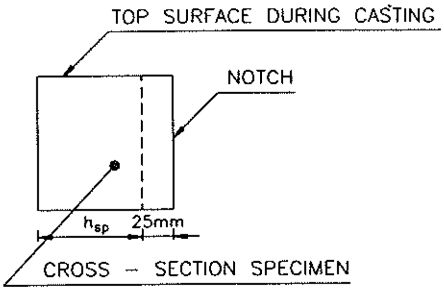

Figure 1 : Position of notch sawn into the test beam.

then sawn through the width of the beam at midspan (see Figure 1).

A schematic picture of the test set-up is shown in Figure 2.

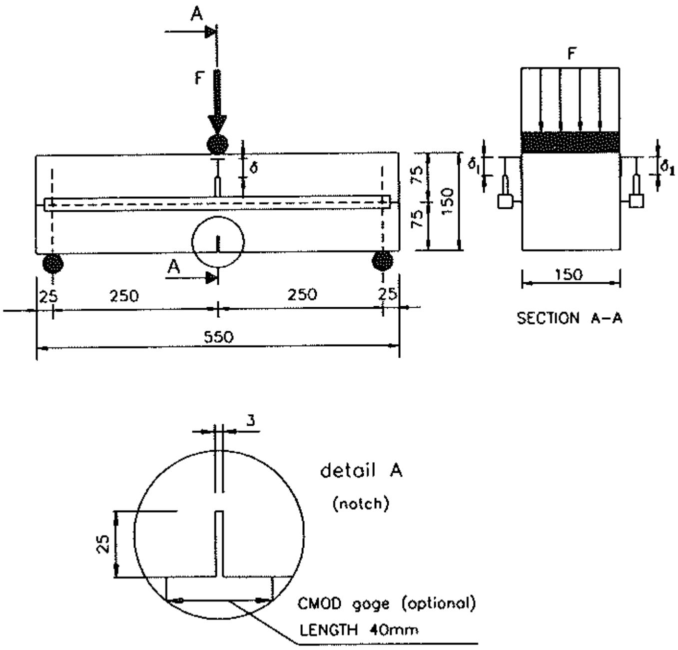

Figure 2 : Test set-up of three-point bending test.

The apparatus measuring deflection should be capable of recording accurately the net-deflection at midspan, i.e. excluding extraneous deformations due to deformations of the machine and/or of the specimen supports. The deflection has to be measured at both sides of the prism (=> δ_I, δ_{II}). The recording of the displacement due to the opening of the mouth of the notch (CMOD) by means of a linear displacement transducer is optional.

The testing machine should be operated so that the measured net-deflection of the specimen at midspan increases with a constant rate of 0.2 mm/min until the specified end-point deflection is reached. During testing the value of the load (F) and the net-deflection at midspan ($\delta = (\delta_I + \delta_{II})/2$) are recorded continuously.

From the measured F-δ-diagram (see Figure 3) the following material parameters are calculated :

- limit of proportionality $f_{fct,fl}$
- equivalent flexural tensile strength $f_{eq,2}$ and $f_{eq,3}$.

2.2 $f_{fct,fl}$, $f_{eq,2}$ and $f_{eq,3}$

The load at the limit of proportionality (F in N) is determined according to an appropriate diagram in Figure 3. Assuming a stress distribution, as shown in Figure 4, at midspan of the test beam, the limit of proportionality can be calculated using the following expression :

$$f_{fct,fl} = \frac{3\ F_u\ L}{2\ b\ h_{sp}^2} \quad (\text{N/mm}^2) \qquad (1)$$

where b = width of the specimen (mm)
h_{sp} = distance between tip of the notch and top of cross section (mm)
L = span of the specimen (mm).

F_2 (F_3) is equal to the mean force recorded in the shaded area $D^f_{BZ,2}$ ($D^f_{BZ,3}$) and can be calculated as follows :

$$F_2 = \frac{D^f_{BZ,2,I}}{0.65} + \frac{D^f_{BZ,2,II}}{0.50} \quad (\text{N}) \qquad (2)$$

$$F_3 = \frac{D^f_{BZ,3,I}}{2.65} + \frac{D^f_{BZ,3,II}}{2.50} \quad (\text{N}) \qquad (3)$$

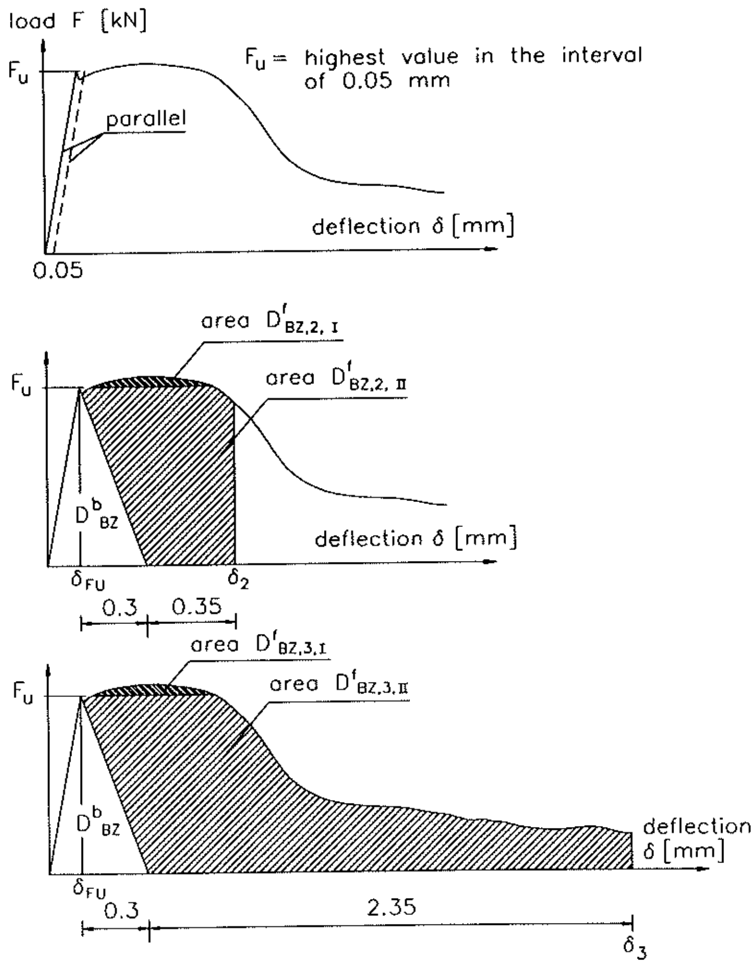

Figure 3 : Diagrams.

where $D^f_{BZ,2,I}$, $D^f_{BZ,2,II}$ ($D^f_{BZ,3,I}$, $D^f_{BZ,3,II}$) = contribution of steel fibers to the energy absorption capacity (Nmm) (see Figure 3).

Assuming at midspan of the prism a linear stress distribution (see Figure 4), the equivalent flexural tensile strength $f_{eq,2}$ and $f_{eq,3}$ can be determined by means of the following equations :

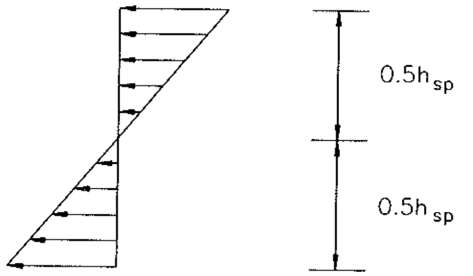

Figure 4 : Stress distribution assumed.

$$f_{eq,2} = \frac{3}{2}\left(\frac{D^f_{BZ,2,I}}{0.65} + \frac{D^f_{BZ,2,II}}{0.50}\right)\frac{L}{b\,h^2_{sp}} \quad (N/mm^2) \qquad (4)$$

$$f_{eq,3} = \frac{3}{2}\left(\frac{D^f_{BZ,3,I}}{2.65} + \frac{D^f_{BZ,3,II}}{2.50}\right)\frac{L}{b\,h^2_{sp}} \quad (N/mm^2) \qquad (5)$$

2.3 σ-ε-diagram

The proposed stress-strain diagram both in compression and tension is shown in Figure 5. The σ-ε-relation for compressed SFRC is identical to that of plain concrete. In tension SFRC is considered elastic till the peak load. The postcracking behavior is characterized by two "stress-values", i.e. 0.45 $f_{eq,2}$ and 0.37 $f_{eq,3}$.

$f_{eq,2}$, which follows from the energy absorption capacity provided by the steel fibers till a deflection of the standard specimen of about 0.7 mm, is calculated considering a linear elastic stress distribution in the section (see Figure 4). However, in reality, the stress distribution will be different. To calculate a "more realistic stress σ_f" in the cracked part of the section, the following assumptions are made as shown in Figure 6 :

- the crack height is equal to 0.65 h_{sp} at a deflection of the standard specimen of ±0.7 mm
- the tensile stress σ_f in the cracked part of the SFRC section is constant.

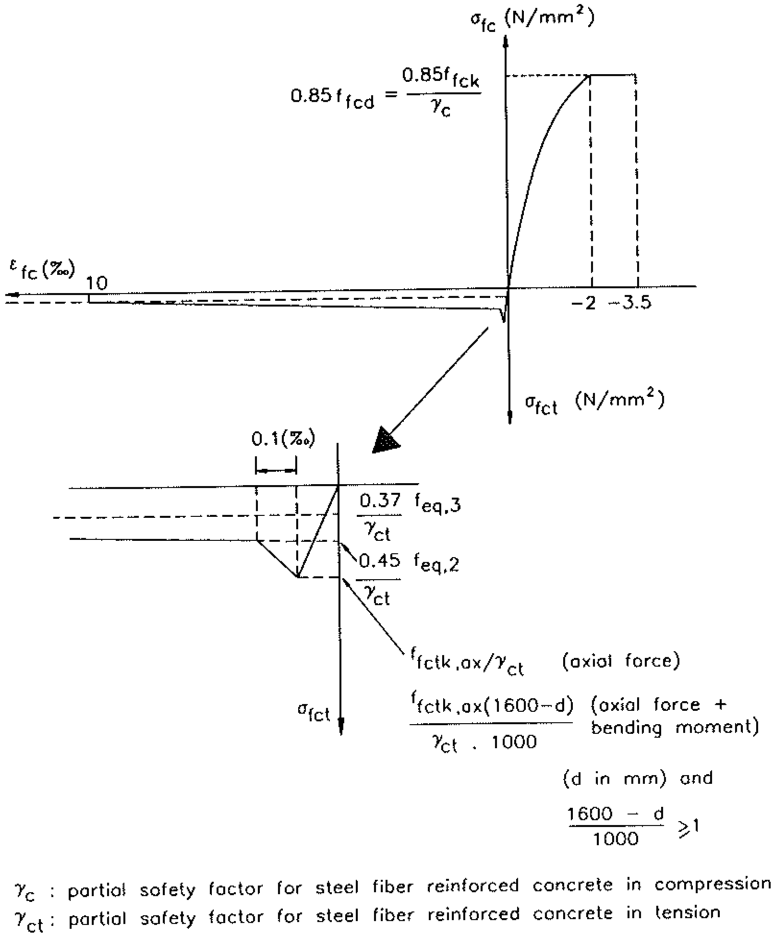

Figure 5 : Stress-strain diagram.

Requiring $M_1 = M_2$, σ_f can then be expressed as :

$$\sigma_f = 0.45 \ f_{eq,2} \qquad (6)$$

The reasoning for the calculation of 0.37 $f_{eq,3}$ is similar to that of 0.45 $f_{eq,2}$. However, since $f_{eq,3}$ is related to a much larger deflection of the standard prism, i.e. $\delta = \pm 2.7$ mm, also the

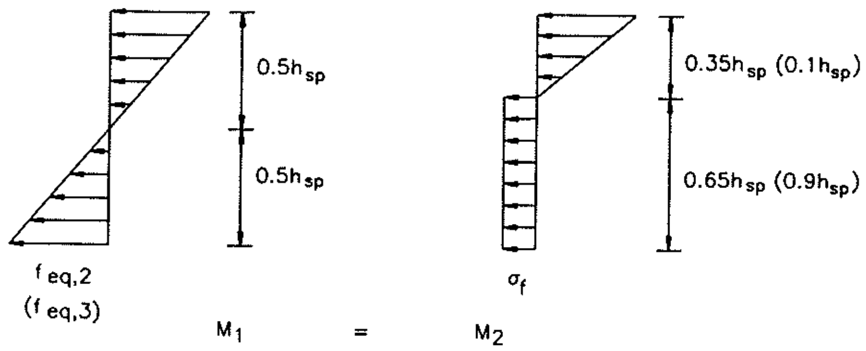

Figure 6 : Calculation of tenisle stress distribution in SFRC.

crack depth corresponding to that deflection increases. From experiments it has been found that at $\delta = \pm 2.7$ mm the crack depth amounts to about 0.9 h_{sp}.

2.4 Limit states

In the proposed σ-ε-design method, two limit states are dealt with :

- ultimate limit state
- serviceability limit state.

The two main loading cases, concerning structural elements behaving like beams, are considered here : bending with or without axial force and shear.

Ultimate limit state - Bending with or without axial load

In assessing the ultimate resistance of a cross section the assumptions given below are used :

- plane sections remain plane;
- σ-ε-diagram for SFRC : see Figure 5 and 2.3;
- the stresses in the reinforcement bars are derived from an idealized bi-linear stress-strain diagram;
- for cross sections subjected to pure axial compression, the compressive strain of SFRC is limited to -2‰. For cross sections not fully in compression, the limiting compressive strain is taken as -3.5‰. In intermediate situations the strain diagram is defined by assuming that the strain is -2‰ at a level $\frac{3}{7}$ of the height of the section from the compressed face;
- for SFRC which is additionally reinforced with bars, the strain is limited to 10‰ at the position of the reinforcement as shown in Figure 7;
- to ensure enough anchorage capacity for the steel fibers, the maximum crack width in

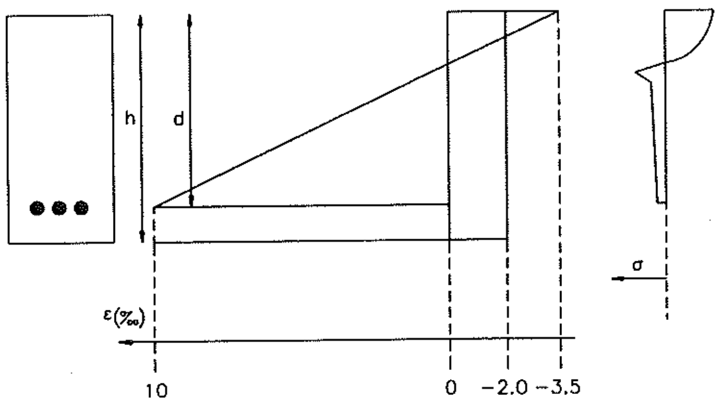

Figure 7 : Strain- and stress-distribution in cross section.

The ultimate limit state is restricted to 1.5 mm. It follows from Figure 3 that "w = 1.5 mm" corresponds approximately to the force F_3 from which the equivalent flexural tensile strength $f_{eq,3}$ has been calculated. If crack widths larger than 1.5 mm are used, the postcracking stress corresponding to that crack width and measured during the bending test has to be adopted in the calculations. It is recommended that this mean value, which replaces $f_{eq,3}$, should not be lower than 1 N/mm²;

- in some cases, the steel fibers should not be taken into account in a layer near the surface :

exposure class 2 [3] : if crack width is larger than 0.2 mm (serviceability limit state) the height of the cracked zone has to be reduced by 15 mm. This rule is only applicable in the ultimate limit state.

exposure class 3 and heigher [3] : special provisions have to be taken.

Ultimate limit state - Shear

The proposed calculation method for shear applies to :

- beams and plates containing longitudinal reinforcement (bar and mesh)
- prestressed elements and columns.

The approach proposed is the best possible until further evidence becomes available. However, it is not applicable to elements for which no longitudinal reinforcement or compressive zone is present.

The "standard method" for shear of Eurocode 2 has been used as basis and extended to SFRC.

The shear load capacity, V_{Rd3}, of a beam may be divided into three terms :

$$V_{Rd3} = V_{Rd1} + V_{wd} + V_{fd} \tag{7}$$

with :

V_{Rd1} : the shear resistance of the member without shear reinforcement due to the compression zone, longitudinal reinforcement, aggregate interlock, ... [3]

V_{wd} : contribution of the shear reinforcement (stirrups - inclined bars) [3]

V_{fd} : contribution of the steel fibers, given by :

$$V_{fd} = k_f \, k_\ell \, \tau_{fd} \, b_w \, d \quad \text{(N)} \tag{8}$$

where : k_f = factor taking into account the contribution of the flanges in a T-section

$$k_f = 1 + n \left(\frac{h_f}{b_w}\right) \left(\frac{h_f}{d}\right) \tag{9}$$

and $k_f \leq 1.5$

with h_f = height of the flanges (mm)
b_f = width of the flanges (mm)
b_w = width of the web (mm)

$$n = \frac{b_f - b_w}{h_f} \quad \text{where} \quad n \leq 3 \text{ and } n \leq \frac{3\, b_w}{h_f}$$

$k_\ell = \dfrac{1600 - d}{1000}$ (d : effective depth (mm)) and $k_\ell \geq 1$

τ_{fd} = design value of the increase in shear strength due to steel fibers

$\tau_{fd} = 0.12 \, f_{eqk,3}$ (N/mm^2)

More than 100 test results of SFRC beams have been used to establish the predictive relation for V_{fd}.

Checking against crushing at the compression struts occurs similarly as for plain concrete, i.e. by means of V_{Rd2} [3].

The minimum shear reinforcement (stirrups + steel fibers) must be such that their shear resistance is at least equal to the shear resistance of plain concrete. Full details concerning the calculation procedure of this minimum shear reinforcement are available in [2].

Serviceability limit state
When an uncracked section is used, both SFRC and steel bars are assumed to behave elastically in tension as well as in compression. In a cracked section, however, SFRC is supposed to be elastic in compression and capable of sustaining a tensile stress equal to $0.45\ f_{eq,2}$.

In the absence of specific requirements (i.e. watertightness, ..), the criteria for the maximum design crack width (w_d) under the quasi-permanent combination of loads [3], which are mentioned in Table 1 for different exposure classes, may be assumed.

Table 1 : Criteria for crack width

Exposure class [3]	steel fibers	steel fibers + ordinary reinforcement	steel fibers +	
			post-tensioning	pre-tensioning
1	(**)	(**)	0.2 mm	0.2 mm
2	0.3 mm	0.3 mm	0.2 mm	decompression(*)
3	special crack limitations dependent upon the nature of the aggressive environment involved have to be taken			
4				
5				

(*) : the decompression limit requires that, under the frequent combination of loads [3], all parts of the tendons or ducts lie at least 25 mm within concrete in compression
(**) : for exposure class 1, crack width has no influence on durability and the limit could be relaxed or deleted unless there are other reasons for its inclusion.

The calculation of the design crack width in SFRC is similar to that in normal reinforced concrete [3] :

$$w_k = \beta\ s_{rm}\ \varepsilon_{sm} \qquad (10)$$

where w_k = design crack width (mm)
s_{rm} = the average final crack spacing (mm)
ε_{sm} = mean steel strain allowed under the relevant combination of actions for the effects of tension stiffening, shrinkage, ...
β = a coefficient relating the average crack width to the design value.

However, when calculating ε_{sm} , it has to be taken into account that the tensile stress in SFRC after cracking is not equal to zero but equal to 0.45 $f_{eq,2}$ (= σ_f) as shown in Figure 8.

No modification in the formula to determine mean final crack spacing s_{rm} has been proposed. The value of the crack spacing is assumed to be independent of the fiber content. However, in reality, the crack spacing in SFRC will be smaller due to the following two phenomena :

- improvement of bond between rebar and concrete due to the addition of steel fibers
- postcracking tensile strength of SFRC.

So the supposed calculation method for the crack width can be considered as conservative.

Details regarding calculation of minimum reinforcement A_s in order to obtain controlled crack formation in SFRC can be found in [2].

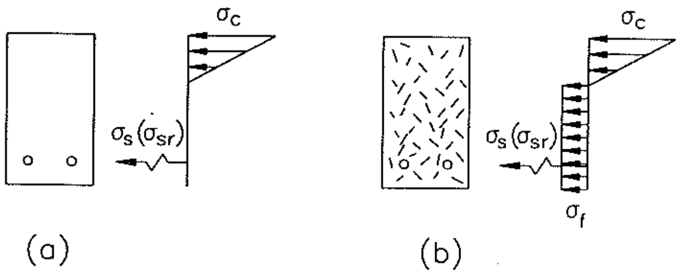

Figure 8 : Stress distribution in a cracked section.

3. σ-w-design method

As mentioned in the introduction, only some general description of the σ-w-design method [4] can be given in this paper since the proposed σ-w-modeling still has to be discussed within Rilem TC 162-TDF.

The ability of the fibers to provide crack bridging forces, i.e. the postcracking behavior of SFRC, is described through the so-called stress-crack opening or σ-w-relationship. This is a well known approach in the description of crack formation in plain concrete, originally suggested by Hillerborg [5]. Later this approach has also been suggested by Hillerborg [6] for use in the description of formation of cracks in fiber reinforced concrete where the model now primarily describes the stresses carried by fibers across a tensile crack in the composite material as function of the crack opening.

Depending on whether the material under uni-axial tension is experiencing multiple cracking or formation of a single crack the overall constitutive behavior for use in structural calculations should be chosen according.

In the proposed σ-w-design method only the case where a single crack is formed under uni-axial tension is considered. In the case of single cracking the σ-w-relationship itself is part of the constitutive relationship. Structural calculations can be formulated either as non-linear finite element calculations describing the cracks through the smeared or discrete crack approach or through some kind of analytical or semi-analytical approach which directly takes the discrete cracking into account using the stress-crack opening relationship. Examples of the application of the proposed σ-w-design method can be found in [8,9].

It is assumed that the SFRC considered here shows a linear response in uni-axial tension up to peak load. After peak load one discrete crack is formed. It is furthermore assumed that the discrete crack formation is described by the stress-crack opening (σ-w) relationship. Thus the following material parameters are fundamental in the constitutive relations of SFRC in tension : the Young's modulus E, the tensile strength f_t and the stress-crack opening relationship denoted $\sigma_w(w)$. In compression it is simply assumed that the behavior is linear elastic and that the Young's modulus is the same as in tension.

In order to determine the stress-crack opening relationship experimentally, deformation controlled tensile tests are conducted on notched cylinders. Full details of the test set-up can be found in [7] and will probably be published as "Recommendations of TC162-TDF" at the end of 2000 in "Materials and Structures".

4. Conclusions

Actually, only a very few design tools are available which can predict the mechanical behavior of structures composed of SFRC. This is due to the fact that primarily the post-peak behavior is affected by the presence of steel fibers while most design tools used by the structural engineer takes only pre-peak behavior into account.

Basically there are two different ways to describe the behavior of SFRC under tension, especially the non-linear postcracking behavior : the stress-strain relation (σ-ε) and the stress-crack opening relation (σ-w). These two models have been restraint by Rilem

TC 162-TDF to set up design methods for SFRC. The σ-ε-design method is based on Eurocode 2 while the σ-w-design method is derived from the fictitious crack model according to Hillerborg.

5. Acknowledgements

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