Materials and Structures / *Matériaux et Constructions*, Vol. **36**, October 2003, pp. 560-567

RILEM TC 162-TDF: 'Test and design methods for steel fibre reinforced concrete'

σ–ε-design method

Final Recommendation

TC Membership: Chairlady: L. Vandewalle, Belgium; **Secretary:** D. Nemegeer, Belgium; **Members:** L. Balazs, Hungary; B. Barr, UK; J. Barros, Portugal; P. Bartos, UK; N. Banthia, Canada; M. Criswell, USA; E. Denarié, Suisse; M. Di Prisco, Italy; H. Falkner, Germany; R. Gettu, Spain; V. Gopalaratnam, USA; P. Groth, Sweden; V. Häusler, Germany; A. Kooiman, the Netherlands; K. Kovler, Israel; B. Massicotte, Canada; S. Mindess, Canada; H.-W. Reinhardt, Germany; P. Rossi, France; S. Schaerlaekens, Belgium; P. Schumacher, the Netherlands; B. Schnütgen, Germany; S. Shah, USA; Å. Skarendahl, Sweden; H. Stang, Denmark; P. Stroeven, the Netherlands; R. Swamy, UK; P. Tatnall, USA; M. Teutsch, Germany; J. Walraven, the Netherlands.

1. INTRODUCTION

The design of steel fibre reinforced concrete according to the σ-ϵ-method is based on the same fundamentals as the design of normal reinforced concrete. The proposed method is valid for steel fibre concrete with compressive strengths of up to C50/60. Steel fibres can also be used in high strength concrete, *i.e.* concrete with $f_{fck} \geq 50\ \text{N/mm}^2$. However, care should be taken that the steel fibres do not break in a brittle way before being pulled out.

The European pre-standard ENV 1992-1-1 (Eurocode 2: Design of Concrete Structures - Part 1: General rules and rules for buildings) [1] has been used as a general framework for this design method proposed.

It must be emphasised that these calculation guidelines are intended for cases in which the steel fibres are used for structural purposes and not *e.g.* for slabs on grade. They also do not apply for other applications such as those in which increased resistance to plastic shrinkage, increased resistance to abrasion or impact, etc. are aimed for.

2. MATERIAL PROPERTIES

2.1 Compressive strength

The compressive strength of steel fibre reinforced concrete (= SFR-concrete) should be determined by means of standard tests, either on concrete cylinders (ϕ = 150 mm, h = 300 mm) or concrete cubes (side = 150 mm).

The design principles are based on the characteristic 28-day strength, defined as that value of strength below which no more than 5 % of the population of all possible strength determinations of the volume of the concrete under consideration, are expected to fall. Hardened SFR-concrete is classified in respect to its compressive strength by SFR-concrete strength classes which relate to the cylinder strength f_{fck} or the cube strength $f_{fck,cube}$ (Table 1). Those strength classes are the same as for plain concrete.

2.2 Flexural tensile strength

When only the compressive strength f_{fck} has been determined, the estimated mean and characteristic flexural tensile strength of steel fibre reinforced concrete may be derived from the following equations:

$$f_{fctm,ax} = 0.3 \cdot \left(f_{fck}\right)^{\frac{2}{3}} \qquad (\text{N/mm}^2) \qquad (1)$$

$$f_{fctk,ax} = 0.7 \cdot f_{fctm,ax} \qquad (\text{N/mm}^2) \qquad (2)$$

$$f_{fct,ax} = 0.6 \cdot f_{fct,fl} \qquad (\text{N/mm}^2) \qquad (3)$$

$$f_{fctk,fl} = 0.7 \cdot f_{fctm,fl} \qquad (\text{N/mm}^2) \qquad (4)$$

Table 1 - Steel fibre reinforced concrete strength classes: characteristic compressive strength f_{fck} (cylinders), mean $f_{fctm,fl}$ and characteristic $f_{fctk,fl}$ flexural tensile strength in N/mm²; mean secant modulus of elasticity in kN/mm²

Strength class of SFRC	C20/25	C25/30	C30/37	C35/45	C40/50	C45/55	C50/60
f_{fck}	20	25	30	35	40	45	50
$f_{fctm,fl}$	3.7	4.3	4.8	5.3	5.8	6.3	6.8
$f_{fctk,fl}$	2.6	3.0	3.4	3.7	4.1	4.4	4.8
E_{fcm}	29	30.5	32	33.5	35	36	37

The corresponding mean and characteristic values for the different steel fibre reinforced concrete strength classes are given in Table 1.

If bending tests are performed, the following method [2] can be used to determine the characteristic value of the limit of proportionality (LOP) (cf. bending test) [3]:

$$f_{fctk,L} = f_{fctm,L} - k_x s_p \qquad (N/mm^2) \qquad (5)$$

with:

$f_{fctk,L}$: characteristic value of LOP (N/mm^2)
$f_{fctm,L}$: mean value of LOP (N/mm^2)
s_p : standard deviation (N/mm^2)

$$s_p = \sqrt{\frac{\sum \left(f_{fctm,L} - f_{fct,L}\right)^2}{(n-1)}} \qquad (6)$$

n : number of specimens
k_x : factor dependent on the number of specimens; some values are given in Table 2.

The maximum value of expressions (4) and (5) can be taken as the flexural tensile strength of the SFR-concrete. In Table 2, $k_{xunknown}$ means that the coefficient of variation of the population is unknown; instead of the standard deviation of the population, the standard deviation of the spot check will be used.

Table 2 – k_x as a function of the number of specimens

n	1	2	3	4	5	6	8	10	20	30	∞
k_{xknown}	2.31	2.01	1.89	1.83	1.80	1.77	1.74	1.72	1.68	1.67	1.64
$k_{xunknown}$	-	-	3.37	2.63	2.33	2.18	2.00	1.92	1.76	1.73	1.64

The mean value of the secant modulus E_{fcm} in kN/mm^2 is also given in Table 1.

2.3 Residual flexural tensile strength

The residual flexural tensile strength $f_{R,i}$, which is an important parameter characterising the post-cracking behaviour of steel fibre reinforced concrete, is determined by the CMOD (crack mouth opening displacement) - or deflection controlled bending test [3].

The residual flexural tensile strengths $f_{R,1}$, $f_{R,4}$ respectively, are defined at the following crack mouth opening displacement ($CMOD_i$) or mid span deflections ($\delta_{R,i}$):

$CMOD_1 = 0.5$ mm - $\delta_{R,1} = 0.46$ mm
$CMOD_4 = 3.5$ mm - $\delta_{R,4} = 3.00$ mm

and can be determined by means of the following expression:

$$f_{R,i} = \frac{3\, F_{R,i}\, L}{2\, b\, h_{sp}^2} \qquad (N/mm^2) \qquad (7)$$

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)
$F_{R,i}$ = load recorded at $CMOD_i$ or $\delta_{R,i}$ (N) (see Fig. 1)

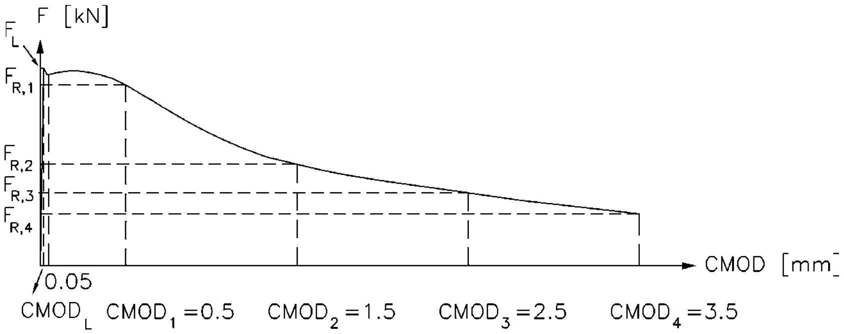


Fig. 1 - Load – CMOD diagram.

The relation between "characteristic" and "mean" residual flexural tensile strength is given in 2.2 (Equation (5)).

Hardened SFR-concrete is classified by using two parameters that are determined by the residual flexural strengths $f_{R,1}$ and $f_{R,4}$. The first parameter $FL_{0.5}$ is given by the value of $f_{R,1}$ reduced to the nearest multiple of 0.5 MPa, and can vary between 1 and 6 MPa. The second parameter $FL_{3.5}$ is given by the value of $f_{R,4}$ reduced to the nearest multiple of 0.5 MPa, and can vary between 0 and 4 MPa. These two parameters denote the minimum guaranteed characteristic residual strengths at CMOD values of 0.5 and 3.5 mm, respectively. The residual strength class is represented as FL $FL_{0.5}/FL_{3.5}$, with the corresponding values of the two parameters. For example, a SFRC with a characteristic cylinder compressive strength of 30 MPa, and $f_{R,1}$ = 2.2 MPa and $f_{R,4}$ = 1.5 MPa would have $FL_{0.5}$ = 2.0 MPa and $FL_{3.5}$ = 1.5 MPa and be classified as C30/37 FL 2.0/1.5.

3. DESIGN AT ULTIMATE LIMIT STATES

3.1 Ultimate limit states for bending and axial force

3.1.1. General

The design method was originally developed without size-dependent safety factors. A comparison of the predictions of the design method and of the experimental results of structural elements of various sizes revealed a severe overestimation of the carrying capacity by the design method. In order to compensate this effect, size-dependent safety factors have been introduced. It should be outlined that the origin of this apparent size-effect is not yet fully understood. Further investigation is required in order to identify if it is due to a discrepancy of material properties between different batches, to a size-effect intrinsic to the method, or a combination of both.

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

- plane sections remain plane (Bernoulli);
- the stresses in the steel fibre reinforced concrete in tension as well as in compression are derived from the

stress-strain diagram shown in Fig. 2 and explained in appendix 1;

- the stresses in the reinforcement (bars) are derived from an idealised bi-linear stress-strain diagram;
- for cross sections subjected to pure axial compression, the compressive strain in the SFR-concrete 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 compressed zone, measured from the most compressed face;
- for steel fibre reinforced concrete which is additionally reinforced with bars, the strain is limited to 25‰ at the position of the reinforcement (Fig. 3);
- to ensure enough anchorage capacity for the steel fibres, the maximum deformation in the ultimate limit state is restricted to 3.5 mm. If crack widths larger than 3.5 mm are used, the residual flexural tensile strength corresponding to that crack width and measured during the bending test has to be used to calculate σ_3. It is recommended that this value, which replaces $f_{R,4}$, should not be lower than 1 N/mm^2;
- in some cases, as mentioned below, the contribution of the steel fibres near the surface has to be reduced. For this reason the steel fibres should not be taken into account in a layer near the surface:

  for exposure class 2 (appendix 2):
  if crack width is larger than 0.2 mm (serviceability limit states: see 4), the height of the cracked zone has to be reduced by 10 mm. This rule is only applicable in the ultimate limit state.
  for exposure classes 3 and higher :
  special provisions have to be taken.

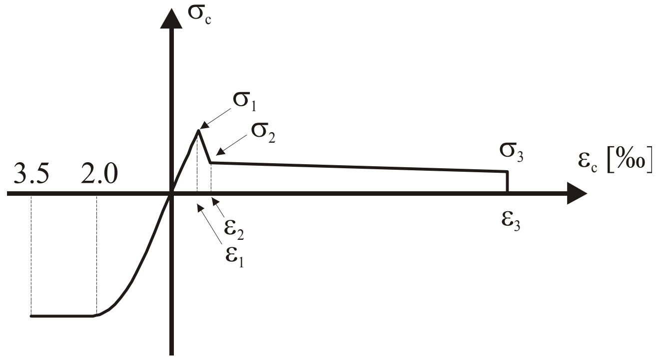
3.1.2 Calculation of crack width

Crack control is required in all structures. This crack control can be satisfied by at least one of the following conditions:

- presence of conventional steel bars,
- presence of normal compressive forces (compression - prestressing),
- crack control maintained by the structural system itself (redistribution of internal moments and forces limited by the rotation capacity).

In structurally indeterminate constructions without conventional reinforcement but with a compression zone in each cross section, the crack width may be determined as follows (see Fig. 4):

- determination of the neutral axis on the basis of Fig. 4,
- determination of the compressive strain $\epsilon_{fc,max}$ of concrete, determination of an idealised tensile strain $\epsilon_{fc,t}$ taking into account the Bernoulli -hypothesis:



$\sigma_1 = 0.7\ f_{fctm,fl}\ (1.6 - d)$ (d in m) (N/mm^2) $\epsilon_1 = \sigma_1 / E_c$

$\sigma_2 = 0.45\ f_{R,1}\ \kappa_h$ (N/mm^2) $\epsilon_2 = \epsilon_1 + 0.1$ ‰

$\sigma_3 = 0.37\ f_{R,4}\ \kappa_h$ (N/mm^2) $\epsilon_3 = 25$ ‰

$E_c = 9500\ (f_{fcm})^{1/3}$ (N/mm^2)

κ_h : size factor

$$\kappa_h = 1.0 - 0.6 \cdot \frac{h\,[\text{cm}] - 12.5}{47.5} \quad \Big|\ 12.5 \le h \le 60\ [\text{cm}]$$

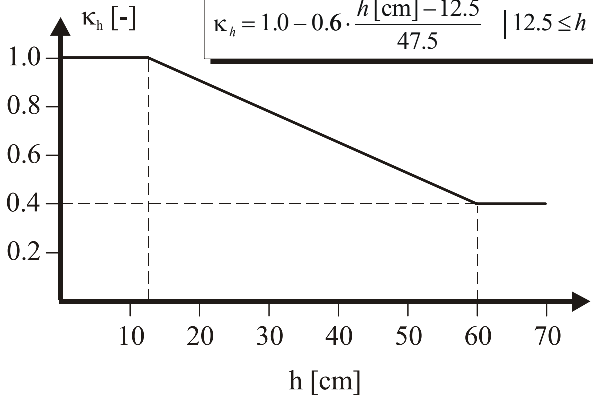


Fig. 2 - Stress-strain diagram and size factor κ_h.

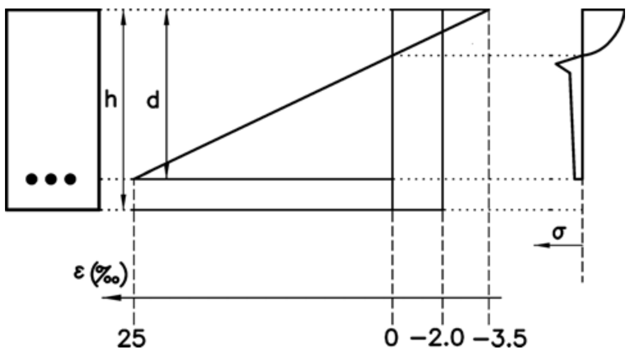


Fig. 3 - Stress– and strain distribution.

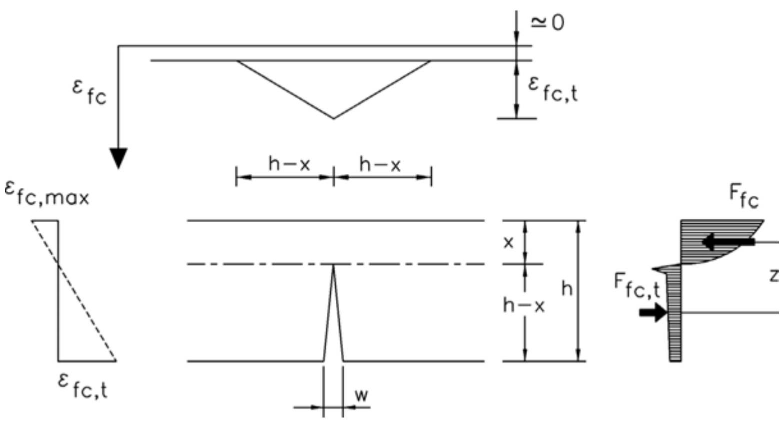


Fig. 4 - Element without ordinary reinforcement.

$$\varepsilon_{fc,t} = \varepsilon_{fc,max} \frac{h-x}{x} \qquad (8)$$

- calculation of the crack width in the ultimate limit state:

$$w = \varepsilon_{fc,t}(h-x) \qquad (9)$$

In statically determinate constructions (bending - pure tension), crack control is only possible if a high amount of steel fibres is used or if there is a combination of steel fibres and conventional reinforcement.

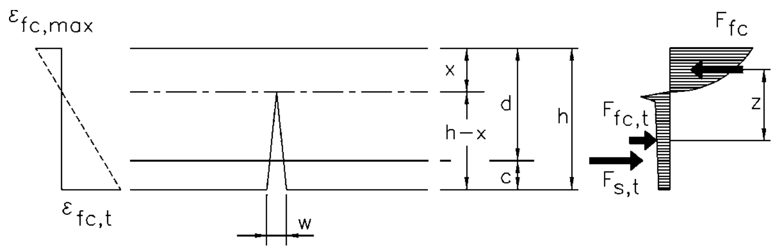


Fig. 5 - Element with ordinary reinforcement.

In structures with conventional reinforcement the calculation of the crack width corresponds to that of normal reinforced concrete. However, the stress in the steel bars has to be calculated, taking into account the beneficial effect of the steel fibres, *i.e.* a part of the tensile force $F_{fc,t}$ which is taken up by the steel fibres (see Fig. 5).

3.2 Shear

The calculation for shear shown here applies to beams and plates containing traditional flexural reinforcement (bar and mesh). It also applies to prestressed elements and columns in which axial compression forces are present. The approach proposed is the best possible until further evidence becomes available.

When no longitudinal reinforcement or compression zone is available, no generally accepted calculation method for taking into account the effect of the steel fibres can be formulated.

Bent-up bars shall not be used as shear reinforcement in beams except in combination with steel fibres and/or stirrups. In this case at least 50 % of the necessary shear reinforcement shall be provided by steel fibres and/or stirrups.

For shear design of members with constant depth, the member is assumed to consist of compressive and tensile zones of which the centres are separated by a distance equal to the internal lever arm z (Fig. 6). The shear zone has a depth equal to z and width b_w . The internal lever arm is calculated perpendicular to the longitudinal reinforcement by ignoring the effect of any bent-up longitudinal reinforcement.

The parameters given in Fig. 6 are:

- α : the angle of the shear reinforcement in relation to the longitudinal axis ($45° \leq \alpha \leq 90°$)
- θ : the angle of the concrete struts in relation to the longitudinal axis
- F_s : tensile force in the longitudinal reinforcement (N)
- F_c : compressive force in the concrete in the direction of the longitudinal axis (N)
- b_w : minimum width of the web (mm)
- d : effective depth (mm)
- s : spacing of stirrups (mm)
- z : the internal lever arm corresponding to the maximum bending moment in the element under consideration (mm) in a member with constant depth. In the shear analysis, an approximate value z = 0.9 d can normally be used.

An example of the standard method, *i.e.*: $\theta = 45°$, will be used for the shear analysis.

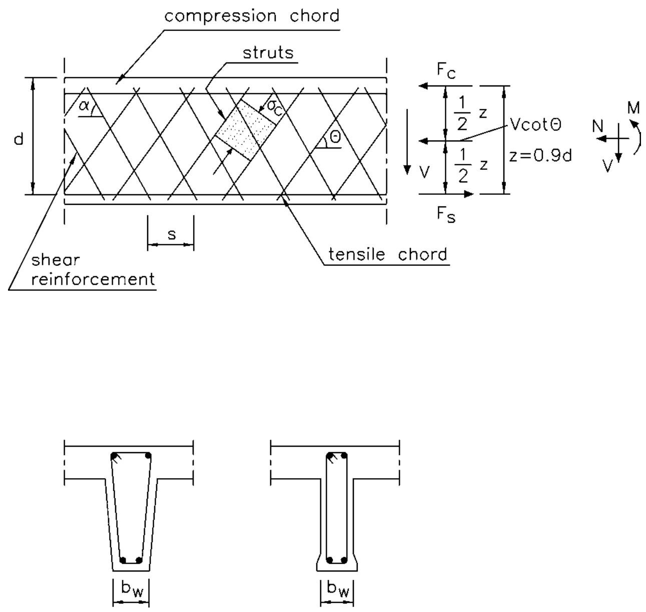


Fig. 6 - Strut and tie model.

3.2.1 *Standard method*

The design shear resistance of a section of a beam with shear reinforcement and containing steel fibres is given by the equation:

$$V_{Rd,3} = V_{cd} + V_{fd} + V_{wd} \qquad (10)$$

with:

V_{cd}: the shear resistance of the member without shear reinforcement, given by:

$$V_{cd} = \left[0.12\ k\ \left(100\ \rho_1\ f_{fck}\right)^{\frac{1}{3}} + 0.15\ \sigma_{cp}\right] b_w\ d\ (N) \qquad (11)$$

where:

$$k = 1 + \sqrt{\frac{200}{d}} \quad (d \text{ in mm}) \quad \text{and} \quad k \leq 2 \qquad (12)$$

$$\rho_1 = \frac{A_s}{b_w d} \leq 2\ \% \qquad (13)$$

A_s = area of tension reinforcement extending not less than "d + anchorage length" beyond the section considered (Fig. 7) (mm^2).

b_w = minimum width of the section over the effective depth d (mm).

$$\sigma_{cp} = \frac{N_{Sd}}{A_c} \quad (N/mm^2) \qquad (14)$$

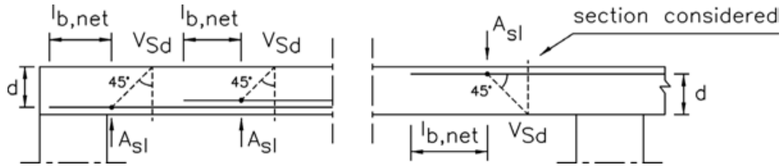


Fig. 7 - ρ_l for V_{cd}.

N_{Sd} = longitudinal force in section due to loading or prestressing (compression: positive) (N). In the case of prestressing, "h" should be used in stead of "d" in formula (11).

V_{fd}: contribution of the steel fibre shear reinforcement, given by:

$$V_{fd} = 0.7\, k_f\, k_l\, \tau_{fd}\, b_w\, d \qquad (N) \qquad (15)$$

where:

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

$$k_f = 1 + n\left(\frac{h_f}{b_w}\right)\left(\frac{h_f}{d}\right) \quad \text{and} \quad k_f \le 1.5 \qquad (16)$$

with

h_f = height of the flanges (mm)

b_f = width of the flanges (mm)

b_w = width of the web (mm)

$$n = \frac{b_f - b_w}{h_f} \le 3 \quad \text{and} \quad n \le \frac{3b_w}{h_f}$$

$$k_l = 1 + \sqrt{\frac{200}{d}} \quad (\text{d in mm}) \quad \text{and} \quad k \le 2 \qquad (17)$$

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

$$\tau_{fd} = 0.12\, f_{Rk,4} \qquad (N/mm^2) \qquad (18)$$

V_{wd}: contribution of the shear reinforcement due to stirrups and/or inclined bars, given by:

$$V_{wd} = \frac{A_{sw}}{s} 0.9\, d\, f_{ywd}\, (1 + \cot\alpha) \sin\alpha \qquad (N) \qquad (19)$$

where:

s = spacing between the shear reinforcement measured along the longitudinal axis (mm)

α = angle of the shear reinforcement with the longitudinal axis

f_{ywd} = design yield strength of the shear reinforcement (N/mm^2).

When checking against crushing at the compression struts, V_{Rd2} is given by the equation:

$$V_{Rd,2} = \frac{1}{2} \nu\, f_{cd}\, 0.9\, d\, b_w\, (1 + \cot\alpha) \qquad (N) \qquad (20)$$

with:

$$\nu = 0.7 - \frac{f_{fck}}{200} \ge 0.5 \qquad (f_{fck} \text{ in } N/mm^2) \qquad (21)$$

For vertical stirrups, or for vertical stirrups combined with inclined shear reinforcement, $\cot\alpha$ is taken as zero.

4. DESIGN AT SERVICEABILITY LIMIT STATES

4.1 General

When an uncracked section is used, the full steel fibre reinforced concrete section is assumed to be active and both concrete and steel are assumed to be elastic in tension as well as in compression.

When a cracked section is used, the steel fibre reinforced concrete is assumed to be elastic in compression, and capable of sustaining a tensile stress equal to 0.45 $f_{R,1}$.

4.2 Limit states of cracking

In the absence of specific requirements (*e.g.* watertightness), the criteria for the maximum design crack width (w_d) under the quasi-permanent combination (***) of loads, which are mentioned in Table 3 for different exposure classes (see Appendix 2), may be assumed.

Table 3 - Criteria for crack width

Exposure class (*)	Steel fibres	Steel fibres + ordinary reinforcement	steel fibres +	
			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				

(*): *see Appendix 2*

(**): *the decompression limit requires that, under the frequent combination (***) of loads, all parts of the tendons or ducts lie at least 25 mm within concrete in compression*

(***): *see ENV 1992-1-1 [1]*

(****): *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.*

4.3 Minimum reinforcement

The following formula is proposed for calculating the minimum reinforcement A_s in order to obtain controlled crack formation:

$$A_s = (k_c\, k\, k_p\, f_{fct,ef} - 0.45\, f_{Rm,1}) \frac{A_{ct}}{\sigma_s} \qquad (mm^2) \qquad (22)$$

where:

$f_{Rm,1}$ = the average residual flexural tensile strength of the steel fibre reinforced concrete at the moment when a crack is expected to occur (N/mm^2),

A_s = area of reinforcement within tensile zone (mm^2). If A_s is smaller than zero only steel fibres are necessary.

A_{ct} = area of concrete within tensile zone (mm^2). The tensile zone is that part of the section which is calculated to be in tension just before formation of the first crack.

σ_s = the maximum stress permitted in the reinforcement immediately after formation of the crack (N/mm^2). This may be taken equal to the yield strength of the reinforcement (f_{yk}). However, a lower value may be needed to satisfy the crack width limits.

$f_{fct,ef}$ = the tensile strength of the concrete effective at the time when the cracks may first be expected to occur (N/mm^2). In some cases, depending on the ambient conditions, this may be within 3 - 5 days from casting. Values of $f_{fct,ef}$ may be obtained from formula (1) by taking as f_{fck} the strength at the time cracking is expected to occur. When the time of cracking cannot be established with confidence as being less than 28 days, it is recommended that a minimum tensile strength of 3 N/mm^2 be adopted.

k_c = a coefficient which takes account of the nature of the stress distribution within the section immediately prior to cracking. The relevant stress distribution is that resulting from the combination of effects of loading and restrained imposed deformations.
k_c = 1 for pure tension ($e = M/N = 0$)
k_c = 0.4 for bending without normal compressive force ($e = \infty$).
In the range between $e = 0$ and $e = \infty$:

* $e/h < 0.4$

$$k_c = \frac{1+\dfrac{e}{0.4\,h}}{1+\dfrac{6\,e}{h}} \tag{23}$$

* $e/h \geq 0.4$

$$k_c = \frac{1+\dfrac{0.4\,h}{e}}{2.5\left(1+\dfrac{h}{6\,e}\right)} \tag{24}$$

k = a coefficient which allows for the effect of non-uniform self-equilibrating stresses. The value can be taken as 0.8 as a first approximation. For further details, see ENV 1992-1-1 [1].

k_p = a coefficient which takes account of the prestressing effect:

$$k_p = 1-\frac{\alpha}{k_c}\left(1-k_c+2.4\frac{e_v}{h}-6\frac{e_v\,k_c}{h}\right) \tag{25}$$

where:

$$\alpha = \text{the ratio of prestressing} = \frac{\sigma_{cp}}{k\,f_{fct,ef}} \tag{26}$$

$$\sigma_{cp} = \frac{N_{Sd}}{A_c} \qquad (N/mm^2)$$

N_{Sd} = prestressing force (N)
A_c = cross section of concrete (mm^2)
e_v = the eccentricity of the prestressing force (mm)

if $e_v = 0$

$$k_p = 1-\frac{\alpha}{k_c}(1-k_c) \tag{27}$$

for pure bending (k_c= 0.4), it follows that:

$$k_p = 1-1.5\,\alpha \tag{28}$$

4.4 Calculation of crack width

Crack control is only possible if at least one of the conditions mentioned in 3.1.2 is satisfied. The calculation of the design crack width in steel fibre reinforced concrete is similar to that in normal reinforced concrete. However, it has to be taken into account that the tensile stress in steel fibre reinforced concrete after cracking is not equal to zero but equal to 0.45 $f_{Rm,1}$ (constant over the cracked part of the cross section).

Formula (22) can be used to calculate the reinforcement A_{sr} (mm^2) which satisfies the crack width limit.

With $\gamma_R = f_{yk} / \sigma_s = 1.4$ the crack width is approximately limited to 0.25 mm:

$$\frac{A_{sr}}{A_{ct}} = \frac{k_c\,k_p\,k\,f_{fct,ef}-\dfrac{0.45}{1.4}f_{Rm,1}}{\dfrac{f_{yk}}{1.4}} \tag{29}$$

In ordinary reinforced concrete, the following formula is used:

$$w_k = \beta\,s_{rm}\,\varepsilon_{sm} \tag{30}$$

where:

w_k = the design crack width (mm)
s_{rm} = the average final crack spacing (mm)
ε_{sm} = the mean steel strain in the reinforcement allowed under the relevant combination of loads for the effects of tension stiffening, shrinkage, etc.
β = a coefficient relating the average crack width to the design value.
= 1.7 for load induced cracking and for restrained cracking in sections with a minimum dimension in excess of 800 mm.
= 1.3 for restrained cracking in sections with a minimum depth, breadth or thickness (whichever is the lesser) of 300 mm or below.
Values for intermediate section sizes may be interpolated.

ε_{sm} may be calculated from the relation:

$$\varepsilon_{sm} = \frac{\sigma_s}{E_s}\left[1-\beta_1\,\beta_2\left(\frac{\sigma_{sr}}{\sigma_s}\right)^2\right] \tag{31}$$

where:

σ_s = the stress in the tensile reinforcement calculated on the basis of a cracked section (N/mm^2).
σ_{sr} = the stress in the tensile reinforcement calculated on the basis of a cracked section under loading conditions causing first cracking (N/mm^2).
β_1 = coefficient which takes account of the bond properties of the bars
= 1.0 for high bond bars
= 0.5 for plain bars

ß$_2$ = a coefficient which takes account of the duration of the loading or of repeated loading
= 1.0 for single, short term loading
= 0.5 for a sustained load or for many cycles of repeated loading.

For members subjected only to intrinsic imposed deformations, σ_s may be taken as equal to σ_{sr}.

The average final crack spacing for members subjected principally to flexure or tension can be calculated from the equation:

$$s_{rm} = \left(50 + 0.25\, k_1\, k_2 \frac{\phi_b}{\rho_r}\right)\left(\frac{50}{L/\phi}\right) \quad \text{(mm)} \qquad (32)$$

where:
$50/(L/\phi) \leq 1$

ϕ_b = the bar size in mm. Where a mixture of bar sizes is used in a section, an average bar size may be used
k_1 = a coefficient which takes account of the bond properties of the bars;
k_1 = 0.8 for high bond bars and 1.6 for plain bars.

In the case of imposed deformations, k_1 should be replaced by $k_1 \cdot k$, with k being defined in 4.3. Minimum reinforcement.

k_2 = a coefficient which takes account of the form of the strain distribution.
= 0.5 for bending and 1.0 for pure tension.
ρ_r = the effective reinforcement ratio, $A_s/A_{c,eff}$ where A_s is the area of reinforcement contained within the effective tension area $A_{c,eff}$.
The effective tension area is generally the area of concrete surrounding the tension reinforcement of depth equal to 2.5 times the distance from the tension face of the section to the centroid of reinforcement [1].
L = length of steel fibre (mm)
ϕ = diameter of steel fibre (mm)

For steel fibre reinforced concrete, σ_s and σ_{sr} in (31) are calculated taking into account the postcracking tensile strength of the steel fibre reinforced concrete, *i.e.* 0.45 $f_{Rm,1}$, in the cracked part of the section.

5. DETAILING PROVISIONS

The rules applicable to normal reinforcement (bar, mesh) and prestressing tendons can be found in ENV 1992-1-1 [1]. Only requirements applicable to "steel fibre reinforced concrete" will be discussed below.

5.1 Shear reinforcement in beams

A minimum shear reinforcement is not necessary for members with steel fibres. But it must be guaranteed that the fibres have a significant influence on the shear resistance. Fibre type and fibre dosage must be sufficient so that a characteristic residual flexural tensile strength $f_{Rk,4}$ of 1 N/mm^2 is achieved.

6. REFERENCES

[1] European pre-standard: ENV 1992-1-1: Eurocode 2: Design of concrete structures – Part 1: General rules and rules for buildings.

[2] ENV 1991-1: Eurocode 1: Basis of design and actions on structures - Part 1: basis of design, Annex D.3.2: Statistical evaluation of resistance/materials tests, 1994 78-80.

[3] Vandewalle, L. *et al*., Recommendations of RILEM TC162-TDF: 'Test and Design Methods for Steel Fibre Reinforced Concrete: bending test' (final recommendation), *Mater. Struct.* **35** (2002) 579-582.

Appendix 1

The stresses σ_2 and σ_3 in the σ-ϵ-diagram are derived from the residual flexural tensile strength as explained below.

The residual flexural tensile strengths $f_{R,1}$ and $f_{R,4}$ are calculated considering a linear elastic stress distribution in the section [3] (Fig. 1.1a). 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 have been made (Fig. 1.1b): the tensile stress σ_f in the cracked part of the steel fibre concrete section is

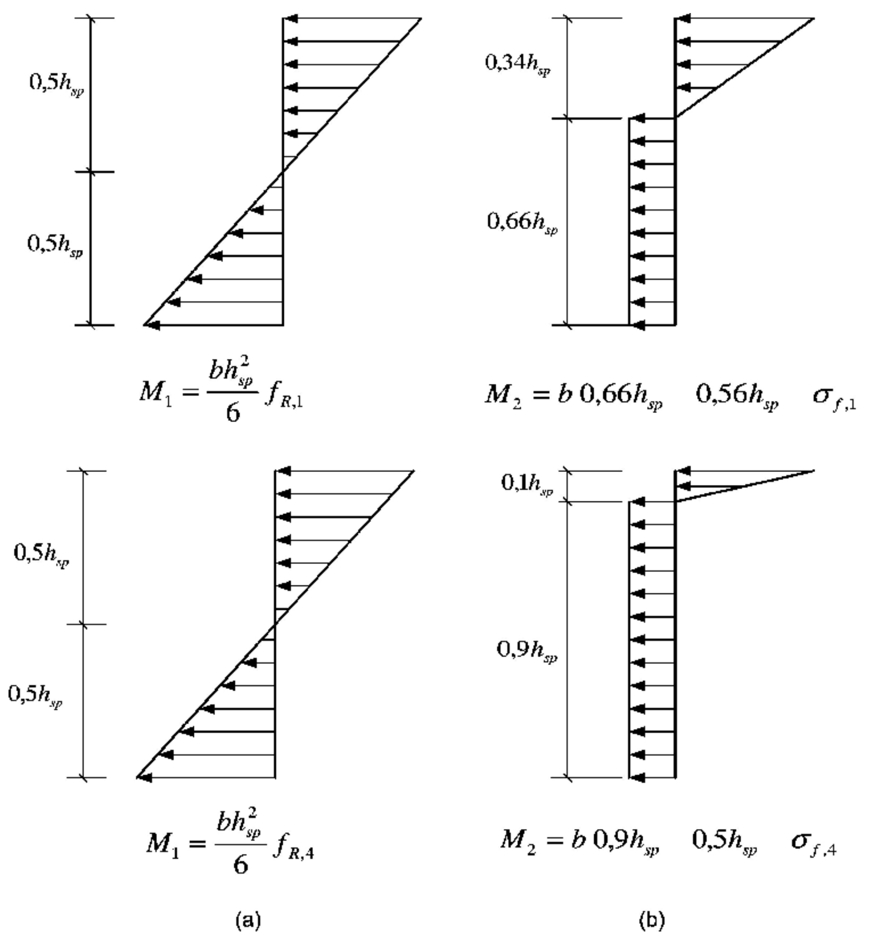


Fig. 1.1 - Stress distribution.

constant;

- the crack height is equal to ±0.66 h_{sp} at $F_{R,1}$, to ±0.90 h_{sp} at $F_{R,4}$ respectively.

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

$\sigma_{f,1} = 0.45\ f_{R,1}$

$\sigma_{f,4} = 0.37\ f_{R,4}$.

Appendix 2

Table A2 – Exposure classes

Exposure class		Examples of environmental conditions
1: dry environment		interior of buildings for normal habitation or offices [1]
2: humid environment	a without frost	- interior of buildings where humidity is high (*e.g.* laundries) - exterior components - components in non-aggressive soil and/or water
	b with frost	- exterior components exposed to frost - components in non-aggressive soil and/or water and exposed to frost - interior components when the humidity is high and exposed to frost
3: humid environment with frost and de-icing salts		interior and exterior components exposed to frost and de-icing agents
4: seawater environment	a without frost	- components completely or partially submerged in seawater, or in the splash zone - components in saturated salt air (coastal area)
	b with frost	- components partially submerged in seawater or in the splash zone and exposed to frost - components in saturated salt air and exposed to frost
The following classes may occur alone or in combination with the above classes:		
5: aggressive chemical environment [2]	a	- slightly aggressive chemical environment (gas, liquid or solid) - aggressive industrial atmosphere
	b	moderately aggressive chemical environment (gas, liquid or solid)
	c	highly aggressive chemical environment (gas, liquid or solid)

(1): *This exposure class is valid only as long as during construction the structure or some of its components is not exposed to more severe conditions over a prolonged period of time.*

(2): *Chemically aggressive environments are classified in ISO/DP 9690. The following equivalent exposure conditions may be assumed:*

Exposure class 5 a: ISO classification A1G, A1L, A1S
Exposure class 5 b: ISO classification A2G, A2L, A2S
Exposure class 5 c: ISO classification A2G, A3L, A3S