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

Recommendations

The text presented hereafter is a draft for general consideration. Comments should be sent to the TC Chairlady : Prof. dr. ir. Lucie Vandewalle, K. U. Leuven, Departement Burgerlijke Bouwkunde, de Croylaan 2, 3001 Heverlee, Belgium Fax : +32 16 321976; e-mail : lucie.vandewalle@bwk.kuleuven.ac.be, by 30 September 2000.

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σ-ε- DESIGN METHOD

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.

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: they 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 (Ø = 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 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

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 SFR-concrete	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
$f_{fctk,fl}$	2.9	3.4	3.9	4.3	4.7	5.1	5.5
E_{fcm}	29	30.5	32	33.5	35	36	37

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.

Steel fibres can also be used in high strength concrete, *i.e.* concrete with $f_{fck} \geq 50$ N/mm².

However, care should be taken that the steel fibres do not break in a brittle way before being pulled out.

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:

$$f_{fctm,ax} = 0.3 \, (f_{fck})^{2/3} \quad (\text{N/mm}^2) \tag{1}$$

$$f_{fctk,ax} = 0.7\ f_{fctm,ax} \quad (N/mm^2) \tag{2}$$

$$f_{fct,ax} = 0.6\ f_{fct,fl} \quad (N/mm^2) \tag{3}$$

$$f_{fctk,fl} = 0.7\ f_{fctm,fl} \quad (N/mm^2) \tag{4}$$

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 can be used to determine the characteristic value of the limit of proportionality (LOP) (cfr. bending test) :

$$\mathbf{f_{fctk,fl} = f_{fctms,fl} - \frac{s_p\ t_{10}}{\sqrt{n}} - 1.645 s_p \left(1 + \frac{s_p\ t_{10}}{f_{fctms,fl}\ \sqrt{n}}\right)} \tag{5}$$

with

$f_{fctk,fl}$: characteristic value of LOP (N/mm²)
$f_{fctms,fl}$: mean value of test series of LOP (N/mm²)
$f_{fctm,fl}$: mean value of LOP (N/mm²)

$$\mathbf{f_{fctm,fl} = f_{fctms,fl} - \frac{s_p\ t_{10}}{\sqrt{n}}}$$

s_p : standard deviation (N/mm²)

$$\mathbf{s_p = \sqrt{\frac{\sum\left(f_{fctms,fl} - f_{fct,fl}\right)^2}{(n-1)}}} \tag{6}$$

n: number of specimens
t_{10} : value for the "student distribution" at 10% fractile (dependent on the number of specimens); some values are given in Table 2.

Table 2							
n	3	4	5	6	8	10	12	15
t_{10}	1.89	1.64	1.53	1.48	1.42	1.38	1.36	1.34

The maximum value of expression (4) and (5) can be taken as the flexural tensile strength of the SFR-concrete.

The mean value of the secant modulus E_{fcm} in kN/mm² is also given in Table 1.

2.3. Equivalent flexural tensile strength

The equivalent flexural tensile strength, which is an important parameter characterizing the post-cracking behaviour of steel fibre reinforced concrete, is determined by a deformation-controlled bending test.

Two different values are defined:

$$\mathbf{f_{eq,2} = \frac{3}{2}\left(\frac{D^f_{BZ,2,I}}{0.65} + \frac{D^f_{BZ,2,II}}{0.50}\right)\frac{L}{b\,h_{sp}^2}} \quad (N/mm^2) \tag{7}$$

$$\mathbf{f_{eq,3} = \frac{3}{2}\left(\frac{D^f_{BZ,3,I}}{2.65} + \frac{D^f_{BZ,3,II}}{2.50}\right)\frac{L}{b\,h_{sp}^2}} \quad (N/mm^2) \tag{8}$$
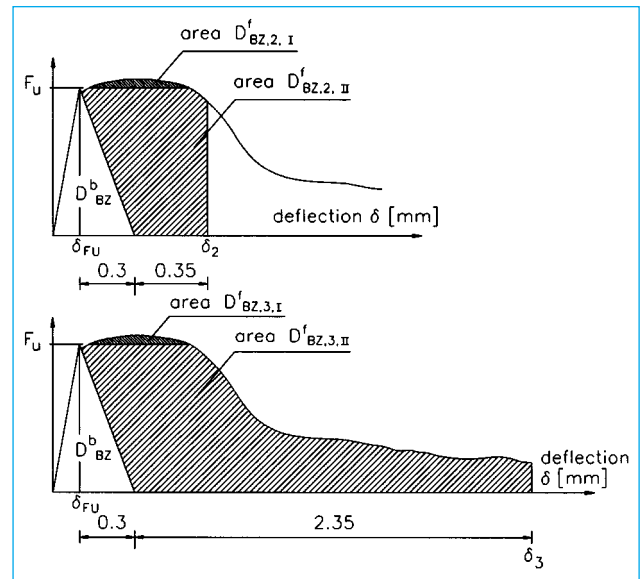
where
b = width of the specimen (mm)

Fig. 1 – Load-deflection diagram.

h_{sp} = distance between tip of the notch and top of cross section (mm)
L = span of the specimen (mm)
$D^f_{BZ,2,I}$, $D^f_{BZ,2,II}$ ($D^f_{BZ,3,I}$, $D^f_{BZ,3,II}$) = contribution of steel fibres to the energy absorption capacity (Nmm) (see Fig. 1).

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

3. ULTIMATE LIMIT STATES

3.1. Ultimate limit states for bending and axial force

3.1.1. General

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;
• 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 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 3/7 of the height of the section from the most compressed face;
• for steel fibre reinforced concrete which is additionally reinforced with bars, the strain is limited to 10‰ 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 1.5 mm. It follows from Fig. 1 that
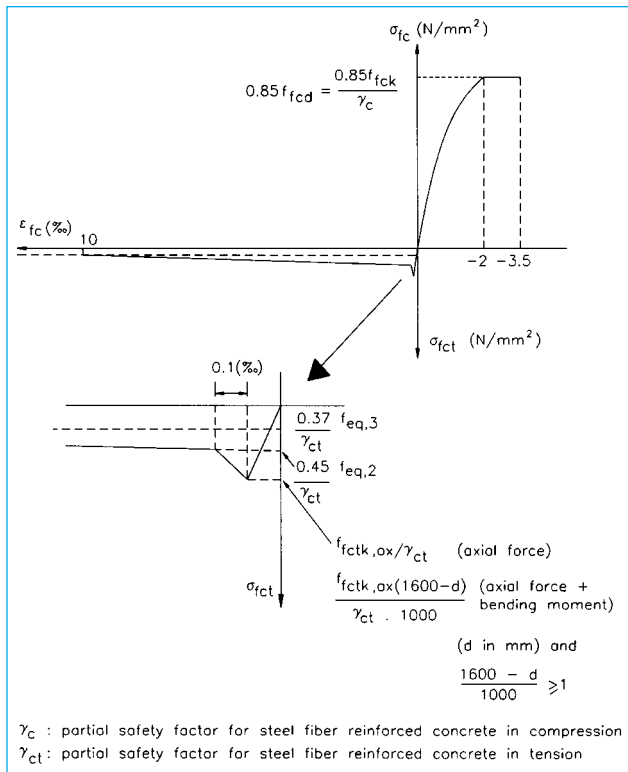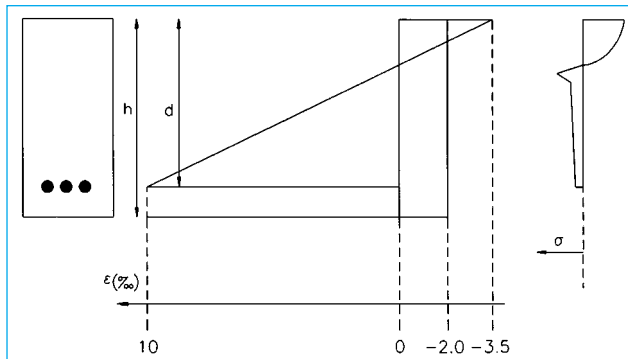
Fig. 2 – Stress-strain diagram.

Fig. 3 – Strain- and stress-distribution in cross section.

"w = 1.5 mm" corresponds approximately to the force

$$F_3 \left(= \frac{D^f_{BZ,3,I}}{2.65} + \frac{D^f_{BZ,3,II}}{2.50} \ (N) \right)$$ from which the equiva-

lent flexural tensile strength $f_{eq,3}$ has been calculated. If crack widths larger than 1.5 mm are used, the post-cracking 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, 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 1): 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 15 mm. This rule is only applicable in the ultimate limit state.
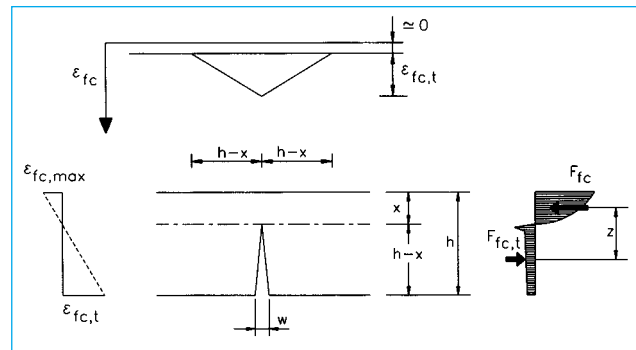

Fig. 4 – Element without ordinary reinforcement.

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 $\in_{fc,max}$ of concrete,
* determination of an idealized tensile strain $\in_{fc,t}$ taking into account the Bernoulli - hypothesis:

$$\in_{fc,t} = \in_{fc,max} \frac{h - x}{x} \tag{9}$$

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

$$w \approx \in_{fc,t} (h - x) \tag{10}$$

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.

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
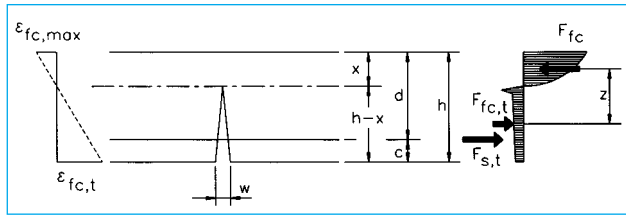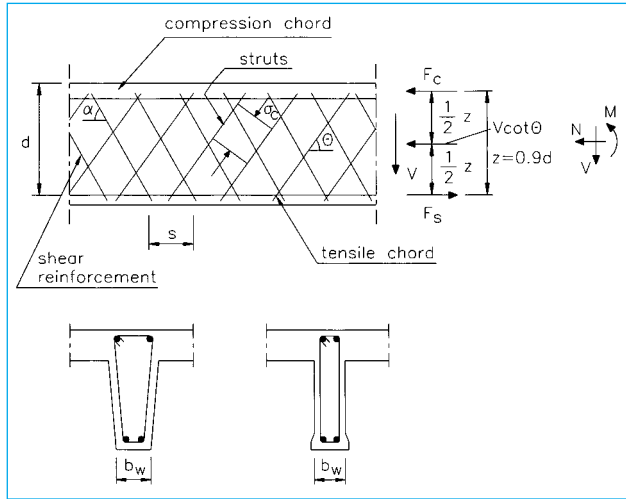
Fig. 5 – Element with ordinary reinforcement.

Fig. 6 – Strut and tie model.

Fig. 7 – ρ_ℓ for V_{cd}.

elements and columns subjected to significant shear forces. 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 maxi-

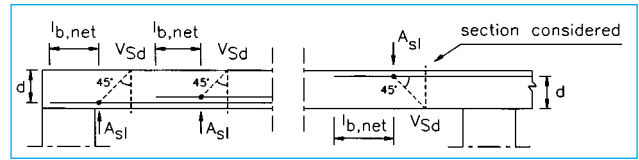mum bending moment in the element under consideration (mm) in a member with constant depth. In the shear analysis, an approximate value z = 0.9 d can normally be used.

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

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:

$$V_{Rd3} = V_{cd} + V_{fd} + V_{wd} \tag{11}$$

with:

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

$$V_{cd} = \left[0.12\,k \left(100\,\rho_\ell\,f_{fck}\right)^{\frac{1}{3}} + 0.15\,\sigma_{cp} \right] b_w\,d \quad \text{(N)} \tag{12}$$

where:

$$k = 1 + \sqrt{\frac{200}{d}} \quad \text{(d in mm) and } k \leq 2 \tag{13}$$

$$\rho_\ell = \frac{A_s}{b_w\,d} \leq 2\% \tag{14}$$

A_s = area of tension reinforcement extending not less than "d + anchorage length" beyond the section considered (Fig. 7) (mm²)
b_w = minimum width of the section over the effective depth d (mm)

$$\sigma_{cp} = \frac{N_{Sd}}{A_c} \quad \text{(N/mm}^2) \tag{15}$$

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 (12).

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

$$V_{fd} = k_f\,k_l\,\tau_{fd}\,b_w\,d \quad \text{(N)} \tag{16}$$

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

$$k_f = 1 + n \left(\frac{h_f}{b_w}\right)\left(\frac{h_f}{d}\right) \tag{17}$$

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)

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

$$k_1 = \frac{1600 - d}{1000} \quad \text{(d in mm) and } k \ge 1 \qquad (18)$$

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

$$\tau_{fd} = 0.12\, f_{eqk,3} \quad (N/mm^2) \qquad (19)$$

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

$$V_{wd} = \frac{A_{sw}}{s} 0.9\, d\, f_{ywd} \left(1 + \cot g\,\alpha\right) \sin\alpha \quad (N) \qquad (20)$$

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:

$$V_{Rd2} = \left(\frac{1}{2} \nu f_{fcd}\right) b_w\, 0.9\, d\, (1 + \cot g\,\alpha) \quad (N) \qquad (21)$$

with:

$$\nu = 0.7 - \frac{f_{fck}}{200} \ge 0.5 \quad \left(f_{fck} \quad (N/mm^2)\right) \qquad (22)$$

For vertical stirrups, or for vertical stirrups combined with inclined shear reinforcement, cotg α is taken as zero.

4. 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 $f_{eq,2}$.

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 1), may be assumed.

4.3. Minimum reinforcement

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

$$A_s = \left(k_c\, k\, k_p\, f_{fct,ef} - 0.45\, f_{eqm,2}\right)\frac{A_{ct}}{\sigma_s} \quad (mm^2) \qquad (23)$$

where:

$f_{eqm,2}$ = the average equivalent 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 = ∞)

In the range between e = 0 and e = ∞;

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				

Table 3 – Criteria for crack width

*): see Appendix 1

(**): 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.

* $e/h < 0.4$

$$k_c = \frac{1 + \dfrac{e}{0.4\,h}}{1 + \dfrac{6\,e}{h}} \qquad (24)$$

* $e/h \geq 0.4$

$$k_c = \frac{1 + \dfrac{0.4\,h}{3\,e}}{2.5\left(1 + \dfrac{h}{6\,e}\right)} \qquad (25)$$

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 :

$$k_p = 1 - \frac{\alpha}{k_c}\left(1 - k_c + 2.4\frac{e_v}{h} - \frac{6\,e_v\,k_c}{h}\right) \qquad (26)$$

where:

α = the ratio of prestressing => $\dfrac{\sigma_{cp}}{f_{ct,ef}}$

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

N_{Sd} = prestressing force (N)
A_c = cross section of concrete (mm^2)
e_v = the eccentricity of the prestressing force (mm)
if $e_v = 0$

$$k_p = 1 - \frac{\alpha}{k_c}\left(1 - k_c\right) \qquad (27)$$

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

$$k_p = 1 - 1.5\,\alpha \qquad (28)$$

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 $0.45\,f_{eq,2}$ (constant over the cracked part of the cross section).

Formula (23) 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:

$$\frac{A_{sr}}{A_{ct}} = \frac{k_c\,k_p\,k\,f_{ct,ef} - \dfrac{0.45}{1.4}\,f_{eqm,2}}{\dfrac{f_{yk}}{1.4}} \qquad (29)$$

In ordinary reinforced concrete, the following formula is used:

$$w_k = \beta\,s_{rm}\,\epsilon_{sm} \qquad (30)$$

where:

w_k = the design crack width (mm)
s_{rm} = the average final crack spacing (mm)
ϵ_{sm} = the mean strain 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.

• The value of β
= 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 :

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

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
β_2 = a coefficient which takes account of the duration of the loading or of repeated loading
= 1.0 a 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:

$$s_m = 50 + 0.25\,k_1\,k_2\,\frac{\phi}{\rho_r} \quad (mm) \qquad (32)$$

where:

ϕ = 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 con-

crete 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].

Equation (32) is identical to that in ENV 1992-1-1 [1]. Since the beneficial effect of the steel fibres has not been taken into account, *i.e.* the crack spacing is smaller due to fibre addition, formula (32) can be considered as conservative.

For steel fibre reinforced concrete, formula (30) is also used, but with σ_s and σ_{sr} in (31) calculated taking into account the postcracking tensile strength of the steel fibre reinforced concrete, *i.e.* $0.45 f_{eq,2}$.

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

The minimum shear reinforcement (stirrups + steel fibres) must be such that their shear resistance is at least equal to the shear resistance of the plain concrete.

The minimum values for:

$$k_f \, 0.15 \, f_{eqk,3} + \frac{A_{sw}}{s \, b_w \sin\alpha} \, 0.9 \, f_{ywd} = \tau_{min} \qquad (33)$$

where:
k_f = factor taking into account the contribution of the flanges in a T-section, see formula (17)
$f_{eqk,3}$ = equivalent flexural tensile strength, see 2.3 (N/mm^2).
A_{sw} = area of the shear reinforcement (stirrups) within length s (mm^2)

Table 4 – Minimum values for shear reinforcement

SFR-Concrete class	τ_{min} (N/mm^2)
C20/25	0.312
C25/30	0.36
C30/37	0.408
C35/45	0.444
C40/50	0.492

s = spacing of the shear reinforcement (stirrups) (mm)
b_w = width of the web of the member (mm)
α = angle between the shear reinforcement and the main steel (*i.e.* for vertical stirrups : $\alpha = 90°$ and $\sin\alpha = 1$)
f_{ywd} = design yield strength of the stirrups (N/mm^2).
are given in Table 4.

REFERENCES

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

APPENDIX 1

(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 A3G, A3L, A3S.

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)