DESIGN GUIDE FOR

FIBRE-REINFORCED CONCRETE STRUCTURES

TO SINGAPORE STANDARD SS 674:2021

By

TAN KIANG HWEE

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Preface

This Design Guide is intended to supplement the Singapore Standard SS 674:2021 on Design of Fibre Concrete Structures, and deals with design requirements. The applications of fibre reinforced concrete are described and relevant design standards are given. The basis of design, material properties, and design for both ultimate and serviceability limit states are explained and illustrated with examples, and compared to the provisions of the Draft Annex L, EN 1992-1-1 on Steel Fibre Reinforced Concrete.

Design charts for flexural design of fibre-reinforced concrete members and procedure to determine steel stresses in fibre concrete members with bar reinforcement are given in the Appendix.

Tan Kiang Hwee

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Foreword by the President of ACES

Formed in 1971, the Association of Consulting Engineers Singapore (ACES) is a non-profit making association representing the independent consulting engineering profession in Singapore. As part of the association's objective, ACES constantly strives to promote the engineering advancement of fellow practitioners in the built environment. Such efforts include continuous exploration of engineering techniques to address the challenges of productivity and sustainability in collaboration with industry partners.

One such initiative is the use and design of Fibre Reinforced Concrete (FRC) structures. In this effort, ACES in collaboration with the Building and Construction Authority (BCA) is driving productivity through promoting the use of fibre reinforced concrete structures in appropriate applications. In conjunction with the publication of the Singapore Standard SS 674:2021 in 2021 which was curated to local applications and national requirements, ACES with BCA has initiated an effort to develop a design guide book for fibre reinforced concrete structures design.

This design guide is intended to raise awareness among designers in the usage of the relatively new standard on FRC, provide guidance and familiarity on the adoption of the design methodology as well as to facilitate safe adoption of FRC in the appropriate applications by understanding the fundamentals in the design considerations.

ACES hopes that this design guide will achieve its purpose for the betterment of all consultants, designers, contractors and all stakeholders in the built environment. The innovative use of FRC can further expand to reach its greater potential in enhancing productivity and resource efficiency to mitigate manpower and resource constraints as well as to achieve a more sustainable built industry.

I would like to express my gratitude to the workgroup members, author of the guidebook, officers of the BCA and friends from the building industry for their contributions and support in making this publication a success for the benefit of the industry.

Er. Chuck Kho
President
Association of Consulting Engineers 2022

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Table of Contents

Chapter One

Introduction

1.1. Fibre Reinforced Concrete

Fibre reinforced concrete (FRC) refers to concrete that contains short, discrete fibres in its mix proportion. It is sometimes referred to as fibre concrete, and they are taken as interchangeable and the same in this design guide. The fibres may be introduced during mixing of concrete (*pre-mix concrete*), or in shotcreting (*sprayed concrete*). The purpose of the fibre addition is to improve the performance of concrete, in particular, in its post-cracking behaviour.

FRC may exhibit softening or hardening behavior after cracking in tension and this depends on the fibre content (*Naaman 2017*). In strain-softening FRC, the post-crack stress resistance decreases with larger deformation, whereas in strain-hardening FRC, it is the reverse. It should however be noted that softening behavior in tension may result in hardening behaviour in bending (*Model Code 2010*), depending for example, on the presence of bar reinforcement. Likewise, softening behaviour in bending may result in hardening in structures depending on the support constraints and others.

In ultra-high performance concrete (UHPC), defined as concrete with a minimum compressive strength of 120$\sim$150 MPa, fibres are usually incorporated which improve the toughness and ductility of the composite tremendously. Such a concrete, termed ultra-high performance fibre-reinforced concrete (UHP-FRC), has a strain-hardening behavior after cracking, and is beyond the scope of the Standard SS 674:2021 as well as this Guide.

1.2. Types of Fibres

There are many types of fibres. Depending on their dimension, fibres may be classified into micro- and macro-fibres. Micro-fibres are defined as having a diameter or equivalent diameter of 0.3 mm or less (PRC-544.10-21), and typically 0.05 mm or less (Naaman 2017). Their aspect ratio (length to diameter ratio) is relatively high, and may exceed 1000. Examples of micro-fibres include those of carbon, polypropylene and PVA. When added to concrete, they lead to very large numbers of fibres per unit volume of concrete, which help to induce micro-toughness and reduce the size of microcracks. Also, they are known to improve the bond of macro-fibres when used in

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combination. The presence of monofilament polypropylene fibres is also useful in limiting spalling effects in fibre concrete (ACI 544.5) in the event of fire.

Macro-fibres include steel fibres and polymeric fibres, also known as macro-synthetic fibres. Fibres must conform to EN 14889-1 for steel fibres, and EN 14889-2 for polymeric fibres. Steel fibres are suitable reinforcement material for concrete because they possess a thermal expansion coefficient equal to that of concrete, and Young's modulus that is at least five times higher than that of concrete. Also, creep of regular carbon steel fibres can occur only above 370°C (EN 14488-3). Polymeric fibres with properties significantly affected by time and/or thermo-hygrometrical phenomena may only be used if due consideration has been given to the environment and condition in which they are applied.

Steel fibres come in different longitudinal profiles, with the hooked end steel fibres being more popularly in use. They typically measure 40 to 60 mm in length and 0.5 to 0.8 mm in diameter, with an aspect ratio of about 80. They are typically used with a dosage of 25 kg/m^3 to 60 kg/m^3 of concrete, and possibly up to 120 kg/m^3. Macro-synthetic fibres typically have a diameter of 0.3 to 1.0 mm, 19 to 65 mm length, much smaller density and used with a dosage of 2.5 to 7.5 kg/m^3 of concrete, and possibly up to 9 kg/m^3.

1.3. Applications

Fibres may be incorporated to increase the resistance of concrete members against flexure, shear, torsion and axial tension. They also assist in crack control and enhancement in ductility and durability of concrete members. Fibre-reinforced concrete is also more resistant to abrasion, impact and blast actions.

Fibre-reinforced concrete can be used with or without conventional bar reinforcement, depending on the structural system, as well as the requirement for flexural moment resistance.

For slabs-on-grade, slab-on-pile systems, and tunnel segmental linings, fibre reinforced concrete may be used without conventional steel bars. It may be used in axially loaded members such as bored piles, pile caps, foundation beams and raft foundation (Oslejs, 2008) with or without bar reinforcement.

It can also be used in flexural members such as suspended slabs and beams, for which a minimum amount of conventional steel bar reinforcement should be incorporated. In structural members, it can be used to replace transverse links required to resist shear or punching shear.

1.4. Design Standards and Technical Reports

Several standards and technical reports are available for the design of fibre-reinforced concrete and specific applications of fibre-reinforced concrete. These include:

General Aspects

- The Concrete Society (2007). *Guidance for the Design of Steel-Fibre-Reinforced Concrete*. Technical Report No. 67.
- ACI Technical Committee 544 (2018). Guide for Design with Fiber-Reinforced Concrete. *ACI 544.4R-18*. American Concrete Institute.
- International Federation for Structural Concrete (2013). *fib* Model Code for Concrete Structures 2020. *Chaps. 5.6 & 7.7*. Ernst & Sohn.

Segmental Tunnel Linings

- International Tunnelling and Underground Space Association (2019). *Guidelines for the design of segmental tunnel linings*. ITA Working Group 2 – Research, ITA Report No. 22, April 2019.
- The British Standards Institution (2016). *Tunnel Design – Design of concrete segmental tunnel linings – Code of practice, PAS8810:2016*. BSI Standards Limited 2016.
- ACI Technical Committee 544 (2016). *Report on Design and Construction of Steel Fiber-Reinforced Precast Concrete Tunnel Segments*. ACI 544.7R-16. American Concrete Institute.

Slabs-on-Grade and Slab-on-Piles Systems

- The Concrete Society (2016). *Concrete Industrial Ground Floors*. Technical Report No. 34, 4th Edition.
- ACI Technical Committee 544 (2015). *Report on Design and Construction of Steel Fiber-Reinforced Concrete Elevated Slabs*. ACI 544.6R-15. American Concrete Institute.

1.5. About the Singapore Standard

The Singapore Standard SS 674:2021 is a modified adoption of the Swedish Standard SS 812310:2014, "Fibre Concrete – Design of Fibre Concrete Structures", published by Swedish Standards Institute. It is applicable to the design of buildings and civil engineering works in concrete with steel or polymer fibres satisfying the requirements of BS EN 14889-1 or BS EN

14889-2. Where a particular fibre or its intended performance is not covered by, or the fibre deviates from an existing European Standard, its suitability shall be established with a European Technical Approval (ETA) referring to its use in fibre concrete conforming to this Singapore Standard SS 674:2021.

The Standard is intended to be used in conjunction with SS EN 1992-1-1 Eurocode 2: Design of concrete structures – Part 1-1: General rules and rules for buildings. It is not a Eurocode. Currently, the Eurocode EN1992-1-1 is undergoing revision with a view to include an informative Annex L, on Steel Fibre Reinforced Concrete. The Singapore Standard has been published as an interim measure, to facilitate the industry in the use of fibre reinforced concrete. Both the Swedish Standard SS 812310 and draft Annex L of prEN1992-1-1:2018 (herein referred to as "*Draft Annex L*") make references to the *fib* Model Code for Concrete 2010 (herein referred to as "*MC 2010*") (see Section 1.4).

1.6. About this Guide

This Design Guide is intended to provide the reader with guidance on the use of SS 674:2021 in designing fibre-reinforced concrete members. Comparisons with the *Draft Annex L* and *MC 2010* are made where relevant, and recommendations are given for the interpretation of various clauses. The corresponding clauses in *Draft Annex L* and other relevant information are given in the Commentary, shown in italics.

1.7. Execution Classes and Quality Assurance

The Standard provides guidance for the specification of execution classes for different types of fibre concrete structures. The execution classes depend on the consequences of a non-conforming fibre concrete. They are Class 1 (non-structural), Class 2 (part-structural) and Class 3 (structural), as listed in Table Q.1 in Annex Q of SS 674:2021. Examples of various execution classes are shown in Table 1.1, which differ from those in Table Q.1 in view of the local practice and the requirements for production and conformity control as well as execution control. In particular, this Guide recommends that slabs on piles, walls under compressive loading, elevated slabs with fibre concrete and bar reinforcement, and tunnel segmental linings, be considered as Class 3 (Structural) systems.

Table 1.1. Execution Classes for Fibre Concrete

Execution Class	Class 1	Class 2	Class 3
Function of fibres	Non-structural	Part-Structural	Structural
Examples of structures	• Slabs on ground not designed for external loading, • Overlays only exposed to restraint stresses	• Slabs on ground designed for significant loadings or high demands on crack control, etc.	• Slabs on piles, • Walls under compressive loading, • Elevated slabs with bar reinforcement, • Tunnel segmental linings, etc.

Tables 1.2 and 1.3 list the requirements for production and conformity control, and execution control of fibre concrete, respectively. The production and conformity control requirements are the same for all execution classes. Whereas, the execution control requirements are more stringent in Execution Class 3 structures, followed by Class 2 and Class 1 structures.

Table 1.2. Production and Conformity Control

Test item	Test method	No. of samples per test occasion
Residual flexural tensile strength at 28 days	BS EN 14651	At least **12**
Fibre distribution in fresh concrete	BS EN 14721 (steel) BS EN 14488-7 (polymer)	At least **3**

Table 1.3. Execution Control

Test item	Class 1	Class 2	Class 3
Fibre content	Not req'd	1 per **100** m^3 or **3** per pour	**3** per 100 m^3 or **3** per pour
Residual flexural tensile strength	Not req'd	Not req'd	**2** per 100 m^3 or **2** per pour

Chapter Two

Basis of Design

2.1. Material and Product Properties

The design of fibre concrete structures shall be in accordance with the general rules given in SS EN 1990 and the supplementary provisions given in SS EN 1992-1-1. Structural components designed with SS 674:2021 shall possess structural stability in ultimate limit state. This requires one of the following conditions to be fulfilled:

(a) Stress redistribution is possible as in a statically indeterminate system.
(b) Conventional steel bar reinforcement or pre-tensioned steel reinforcement is used in combination with fibre concrete.
(c) Equilibrium is maintained by external normal forces.

Commentary

This constitutes a requirement for structural robustness.

Shrinkage and Creep

Shrinkage and creep are generally considered for the verification of SLS. The effects of shrinkage and creep should be considered at ULS when their effects are significant.

For fibre concrete, the effects of shrinkage and creep at ULS can be evaluated based on elastic or plastic theories. In the elastic method, stresses caused by restrained shrinkage and creep, are added to those due to mechanical loading. Whereas in the plastic method, the strength is considered at an increased strain value considering the increased ductility demand caused by cracking due to shrinkage and creep. For FRC without bar reinforcement, the values of $f_{R,1}$ and $f_{R,3}$ should be replaced by $f_{R,3}$ and $f_{R,4}$, respectively.

Commentary

Steel fibres do not usually affect the creep and shrinkage properties significantly. Draft Annex L specifies creep and shrinkage properties to follow those for non-fibrous concrete. However, values of creep and shrinkage may be subjected to a higher

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variation, and they should be determined by testing if they have an impact on the action effects.

Where polymer fibres are used, the Standard requires long-term tests to be carried out to evaluate the creep properties. Also, if the fibre concrete structure is to be subjected to elevated temperature, long-term tests should be conducted under elevated temperature simulating the actual site conditions.

Deformations of Concrete

The effects of temperature and shrinkage shall be considered as additional action effects, or as increased ductility demand on the moment capacity. This is particularly important in cases where restraint is present.

Commentary

Steel fibres do not significantly affect the modulus of elasticity, Poisson's ratio, or coefficient of thermal expansion of concrete, and the values given for non-fibrous concrete may be used. The values of modulus of elasticity and Poisson's ratio may be subject to higher variations than for ordinary concrete, and should be determined by testing if they form a significant component of the action effects.

2.2. Partial Factors for Fibre Concrete

The partial factor, γ_f, for fibre concrete shall be taken as 1.5 for persistent and transient design situations, 1.2 for accidental design situation, and 1.0 for serviceability limit states.

Commentary

The partial factors for fibre concrete are the same as for conventional concrete.

Chapter Three

Material Properties

The specific properties of fibre concrete that are required for the design of fibre-reinforced concrete include the characteristic residual flexural (tensile) strengths, $f_{R,1}$, $f_{R,3}$ and $f_{R,4}$, at a crack-mouth-opening displacement of 0.5, 2.5 and 3.5 mm (see Fig. 3.1), respectively, determined from beam tests according to BS EN 14651 at the age of 28 days. For sprayed FRC, these values may be obtained from three-point bending tests on square panel with a notch following prEN 14488-3:2021 Method B.

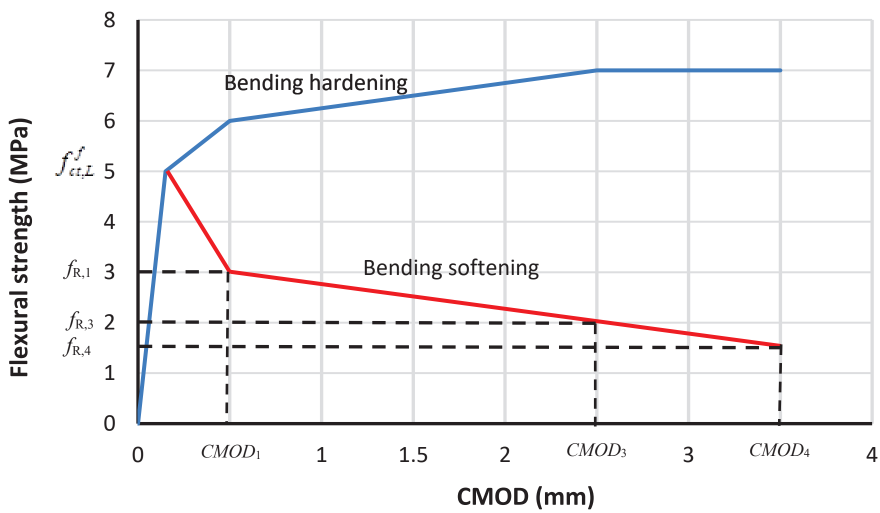

Figure 3.1. Relation between flexural strengths and CMOD (BS EN 14651).

In general, twelve notched prism specimens are required to establish the residual flexural strengths. In cases where preliminary results are desired, or where previous results are available, a smaller number of specimens may be adequate. The characteristic strengths may be obtained from:

$$f_{R,ik} = f_{R,im} - k_n\sigma(i = 1, 2, 3, 4) \tag{3.1}$$

where $f_{R,ik}$ is the characteristic value, $f_{R,im}$ is the mean value, σ is the standard deviation, and k_n is a statistical factor depending on the number

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of specimens, n in the test series, as given in Table 3.1 (SS 674:2021, Annex P; Bekaert 2020).

Table 3.1. Values of k_n for

n	3	4	5	6	8	9	10	12	15	20	30	100
k_n	1.89	1.83	1.80	1.77	1.74	1.73	1.72	1.71	1.70	1.68	1.67	1.64

It is noted that other test methods to determine the residual flexural tensile strengths can be accepted if correlation factors with EN 14651 are proven (MC 2010). Such tests include double punch tests and round panel tests (Tan et al. 2021).

3.1. Residual Flexural Tensile Strength Classes

Fibre concrete is classified by strength classes, R_1, R_3 and R_4, according to the values of residual flexural (tensile) strengths, $f_{R,1}$, $f_{R,3}$ and $f_{R,4}$, respectively (see Table 3.2). For SLS design, Class R_1 needs to be specified. On the other hand, ULS design requires both Class R_1 and Class R_3 to be specified in general, but may be based on Class R_3 alone.

Table 3.2. R-classes for fibre concrete.

Class R_1	$f_{R,1}$ (MPa)	Class R_3	$f_{R,3}$ (MPa)	Class R_4	$f_{R,4}$ (MPa)
$R_1 1$	1.0	$R_3 1$	1.0	$R_4 1$	1.0
$R_1 2$	2.0	$R_3 2$	2.0	$R_4 2$	2.0
$R_1 3$	3.0	$R_3 3$	3.0	$R_4 3$	3.0
$R_1 4$	4.0	$R_3 4$	4.0	$R_4 4$	4.0
$R_1 5$	5.0	$R_3 5$	5.0	$R_4 5$	5.0
$R_1 6$	6.0	$R_3 6$	6.0	$R_4 6$	6.0

In most cases, bending softening behavior prevails with $f^{\mathrm{f}}_{\mathrm{ct,L}} \geq f_{\mathrm{R,1}} \geq f_{\mathrm{R,3}}$, where $f^{\mathrm{f}}_{\mathrm{ct,L}}$ is the limit of proportionality. In such cases, to ensure a minimum degree of ductility of the fibre concrete, the following conditions must be fulfilled:

$$f_{\mathrm{R,1}} / f_{\mathrm{ctk,0.05}} \geq 0.5 \tag{3.2a}$$

and

$$f_{\mathrm{R,3}} / f_{\mathrm{R,1}} \geq 0.5 \tag{3.2b}$$

It is noted that higher residual flexural tensile strength of more than 6 MPa may be used if the specified value is verified by test results.

Commentary

Fibre concrete can be specified as, for example, C30/37 − $R_1 3/R_3 2$ or C30/37 − $R_1 3$ or C30/37 − $R_3 2$, depending on the requirement.

Draft Annex L denotes the characteristic values as $f_{R,1k}$ and $f_{R,3k}$, and classifies steel fibre reinforced concrete into residual strength classes according to the values of $f_{R,1k}$ and $f_{R,3k}/f_{R,1k}$. The residual strength classes are denoted by a numeral (1 to 8), followed by an alphabet (a to e), where the numeral refers to the value of $f_{R,1k}$ in MPa, and the alphabets, a, b, c, d and e denote the ductility class in term of the ratio of $f_{R,3k}/f_{R,1k}$ of 0.5, 0.7, 0.9, 1.1 and 1.3, respectively.

The ductility class with respect to values of $f_{R,1k}$ and $f_{R,3k}$ is shown in Fig. C3.1. The corresponding residual strength classes for R-classes are shown in Table C3.1.

Table C3.1. R- and Residual Strength Class.

R-Class	Residual Strength Class	R-Class	Residual Strength Class
$R_1 1/R_3 1$	1c	$R_1 5/R_3 3$	5a
$R_1 1/R_3 2$	1e	$R_1 5/R_3 4$	5b
$R_1 2/R_3 1$	2a	$R_1 5/R_3 5$	5c
$R_1 2/R_3 2$	2c	$R_1 5/R_3 6$	5d
$R_1 2/R_3 3$	2e	$R_1 6/R_3 3$	6a
$R_1 3/R_3 2$	3a	$R_1 6/R_3 5$	6b
$R_1 3/R_3 3$	3c	$R_1 6/R_3 6$	6c
$R_1 3/R_3 4$	3e	–	7a
$R_1 4/R_3 2$	4a	–	.
$R_1 4/R_3 3$	4b	–	.
$R_1 4/R_3 4$	4c	–	.
$R_1 4/R_3 5$	4d	–	.
$R_1 4/R_3 6$	4e	–	8e

A class 3c (or $R_1 3/R_3$ 3) can be obtained with a concrete C30/37 and an amount of steel fibres ranging between 30 and 40 kg/m^3 (also see Venkateshwaran, Tan and Li, 2018).

Draft Annex L also requires (3.2a) to be satisfied. It is introduced to provide a direct link to the parent concrete specification and facilitate the ease of design. The value of $f_{ctk,0.05}$ is to be taken in accordance to Table 3.1, SS EN1992-1-1.

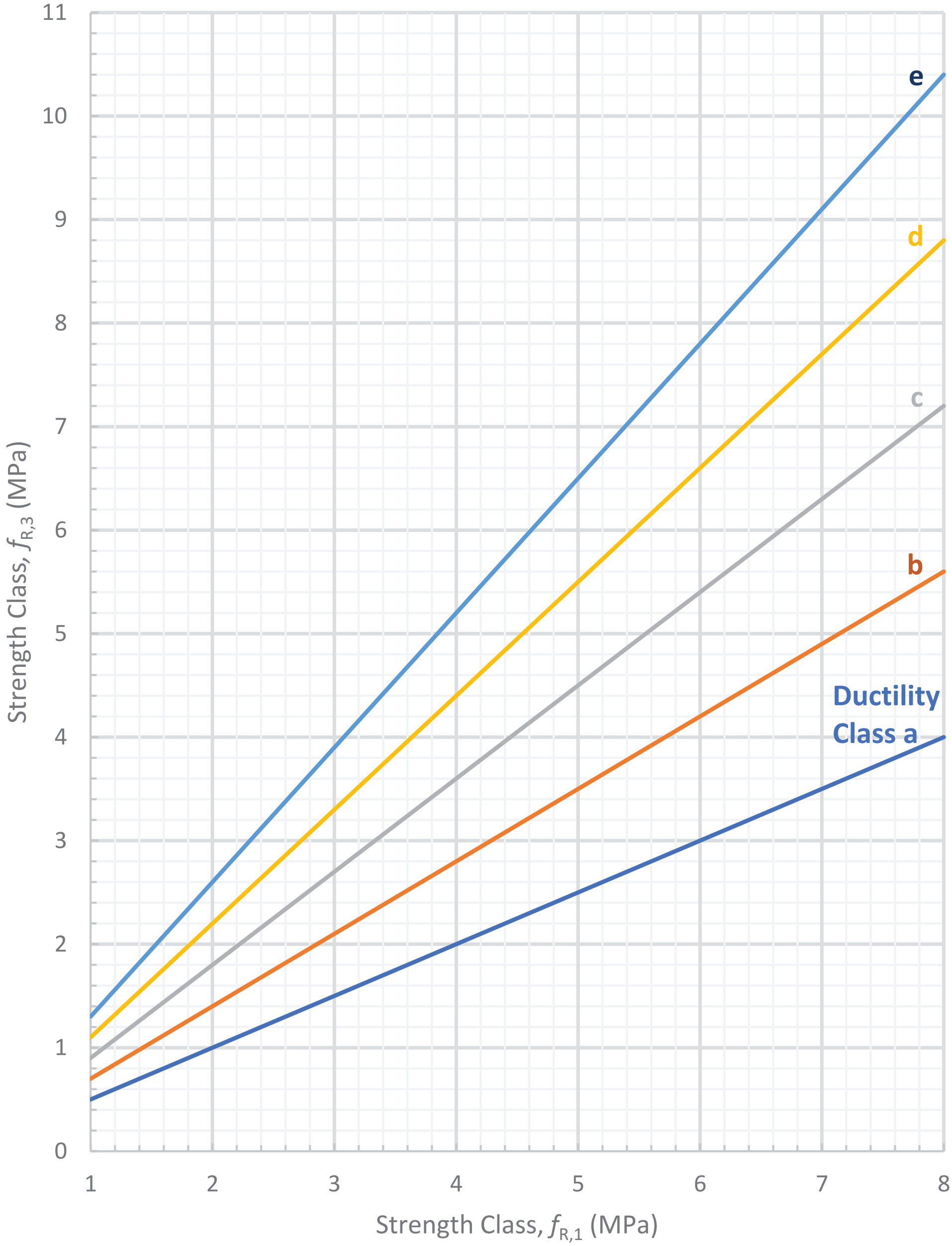

Figure C3.1. Residual Strength Classes

Equation (3.2a) implies that the lowest residual strength classes are: 1.0 for C12/15-C30/37, 1.5 for C35/45-C55/67 and 2.0 for C60/75-C90/105.

3.2. Design Residual Tensile Strength

The characteristic residual tensile strength of fibre reinforced concrete is taken as:

$$f_{\mathrm{ft,R1}} = 0.45 f_{\mathrm{R,1}} \tag{3.3}$$

$$f_{\mathrm{ft,R3}} = 0.37 f_{\mathrm{R,3}} \tag{3.4}$$

The design residual tensile strength in ULS is defined as:

$$f_{\mathrm{ftd,R1}} = \eta_f \cdot \eta_{\mathrm{det}} \cdot \frac{f_{\mathrm{ft,R1}}}{\gamma_{\mathrm{f}}} \tag{3.5}$$

$$f_{\mathrm{ftd,R3}} = \eta_{\mathrm{f}} \cdot \eta_{\mathrm{det}} \cdot \frac{f_{\mathrm{ft,R3}}}{\gamma_{\mathrm{f}}} \tag{3.6}$$

For SLS, it is defined as:

$$f_{\mathrm{ftd,R1}} = \eta_{\mathrm{f}} \cdot f_{\mathrm{ft,R1}} \tag{3.7}$$

where η_{f} is a factor accounting for fibre orientation; and η_{det} is a magnification (conversion) factor accounting for the degree of structural redundancy in the system.

The value of η_{f} is generally taken as 0.5. For horizontally cast concrete members with width-to-thickness ratio of more than 5 (i.e, slabs), the factor may be set to 1.0. For other members, a value higher than 0.5 may be selected after considering member dimensions, fibre length, and casting procedure.

The value of η_{det} takes into account favourable effects due to load redistribution in statically indeterminate structures. The proposed η_{det} values for different structural cases in Table 3.2 of SS 674:2021 are based on the analysis given in Annex S. In view of insufficient information on the value of η_{det}, SS 674:2021 recommends that the values of η_{det} be taken as equal to 1.0 for all cases.

However, considering the recommendations given in *Draft Annex L* and *fib* Model Code 2010, intermediate values for η_{det} shown in Table C3.2 may be justified.

Table C3.2. Suggested Values of η_{det}

Case No.	Type of members	η_{det}
1	Statically determinate beams	1
2	Statically indeterminate beams	1.2
3	Rect. slabs w 2 opp. sides simply-supported	1
4	S.s. circular slabs; Rect. slabs w 3 or 4 s.s. edges	1.2
5	See SS674:2021, Table 3.2	1.4

Commentary

Draft Annex L similarly considers the characteristic residual tensile strength f_{Ftsk} (for crack widths at SLS) and f_{Ftuk} (for ULS based on a rigid plastic approach) as:

$$f_{\mathrm{Ftsk}} = 0.40 f_{R,1k} \tag{C3.1}$$

$$f_{\mathrm{Ftuk}} = 0.37 f_{R,3k} \tag{C3.2}$$

The effective residual tensile strength $f_{\mathrm{Fts,ef}}$ (for crack widths at SLS) and $f_{\mathrm{Ftu,ef}}$ (for ULS), accounting for fibre orientation are:

$$f_{\mathrm{Fts,ef}} = \kappa_{\mathrm{O}} \cdot f_{\mathrm{Ftsk}} \tag{C3.3}$$

$$f_{\mathrm{Ftu,ef}} = \kappa_{\mathrm{O}} \cdot f_{\mathrm{Ftuk}} \tag{C3.4}$$

where κ_{O} is the factor according for fibre orientation (i.e., same as η_{f} in SS 674:2021).

Correspondingly, the design residual tensile strengths for ULS and SLS are respectively:

$$f_{\mathrm{Ftsd}} = f_{\mathrm{Fts,ef}} / \gamma_{\mathrm{SF}} \tag{C3.5}$$

$$f_{\mathrm{Ftud}} = f_{\mathrm{Ftu,ef}} / \gamma_{\mathrm{SF}} \tag{C3.6}$$

where γ_{SF} is the partial factor for steel fibre reinforced concrete (equal to 1.0 for SLS and 1.5 for ULS).

The factor κ_{O} shall be taken as 0.5 unless otherwise specified or verified by testing. For bending moments, shear forces and torsion in slabs and beams made of concrete with consistency classes S2-S4 in accordance with EN206, $\kappa_{\mathrm{O}} = 1.0$ may be used. This is because long relevant experience has shown that the "in-situ-strength" and fibre orientation in these cases are comparable to the EN14651 beam test (fib Model Code 2010).

Example 3.1. Determination of Characteristic Residual Flexural Strengths

Twelve notched prism specimens of a C80/95 FRC with 60 kg/m^3 of hooked-end steel fibres were tested according to BS EN 14651 and the test results are tabulated below. Determine the characteristic strengths, $f_{\mathrm{R},1}$ and $f_{\mathrm{R},3}$.

Specimen No.	1	2	3	4	5	6	7	8	9	10	11	12
$f_{R,1}$ (MPa)	8.5	9.6	10.0	9.8	8.3	7.9	8.9	10.9	9.2	8.2	9.9	6.8
$f_{R,3}$ (MPa)	11.0	9.5	9.3	10.8	10.3	8.9	11.3	9.8	10.2	9.7	10.2	9.4

Solution 3.1.
Mean value, $f_{\mathrm{R,1m}} = 9.0$ MPa with a standard variation $\sigma = 1.13$ MPa and a COV $= 0.125$.

Mean value of $f_{\mathrm{R,3m}} = 10.0$ MPa with a standard variation $\sigma = 0.73$ MPa and a COV $= 0.073$

The characteristic values of $f_{\mathrm{R},1}$ and $f_{\mathrm{R},3}$ follow from (refer to Annex P, SS 674:2021):

$f_{\mathrm{R1,k}} = f_{\mathrm{R1,m}} - 1.71 \cdot \sigma = 9.0 - 1.71(1.13) = 7.1$ MPa

$f_{\mathrm{R3,k}} = f_{\mathrm{R3,m}} - 1.71 \cdot \sigma = 10.0 - 1.71(0.73) = 8.7$ MPa

$f_{\mathrm{R3,k}}/f_{\mathrm{R1,k}} = 8.7/7.1 = 1.22$

The R-class (or residual strength class) of the FRC is therefore $R_1 7/R_3 8$ or Class 7d.

Example 3.2. Determination of Characteristic Residual Flexural Strengths

Five notched prism specimens of a C30/37 FRC with 6 kg/m^3 of macro-synthetic fibres were tested according to BS EN 14651 and the test results are tabulated below. Determine the characteristic strengths, $f_{\mathrm{R},1}$ and $f_{\mathrm{R},3}$.

Solution 3.2.
Mean value, $f_{\mathrm{R,1m}} = 1.78$ MPa with a standard variation $\sigma = 0.31$ MPa and a COV $= 0.176$

Mean value of $f_{\mathrm{R,3m}} = 2.13$ MPa with a standard variation $\sigma = 0.60$ MPa and a COV $= 0.280$

Specimen No.	1	2	3	4	5
$f_{R,1}$ (MPa)	1.50	1.42	2.15	1.98	1.87
$f_{R,3}$ (MPa)	1.60	1.47	2.89	2.49	2.21

The characteristic values of $f_{\mathrm{R},1}$ and $f_{\mathrm{R},3}$ follow from (refer to Bakaert, 2020):

$f_{\mathrm{R1,k}} = f_{\mathrm{R1,m}} - 1.80 \cdot \sigma = 1.78 - 1.80(0.31) = 1.22$ MPa

$f_{\mathrm{R3,k}} = f_{\mathrm{R3,m}} - 1.80 \cdot \sigma = 2.13 - 1.80(0.60) = 1.06$ MPa

$f_{\mathrm{R3,k}}/f_{\mathrm{R1,k}} = 1.06/1.22 = 0.86$

The R-class (or residual strength class) of the FRC is therefore $R_1 1/R_3 1$ or Class 1c.

It should be noted that in general, 12 specimens are recommended for the determination of characteristic values according to EN 14651.

3.3. Design Stress-Strain Relations for Fibre Reinforced Concrete

Different stress-strain relations for fibre reinforced concrete as shown in Figs. 3.2 and 3.3 can be used depending on the type of analysis and the required accuracy.

The "constant-drop" stress-strain relation in Fig. 3.2 is characterized by the design tensile strength f_{ctd} and the design residual tensile strength $f_{\mathrm{ftd,R3}}$, whereas the "linear descending-drop" stress strain relation is additionally characterized by $f_{\mathrm{ftd,R1}}$.

The values of f_{cd} and f_{ctd} in Figs. 3.2 and 3.3 are based on the parent concrete and calculated from Sect. 3.1.6, SS EN 1992-1-1, as:

$$f_{\mathrm{cd}} = 0.85 f_{\mathrm{ck}}/1.5 \tag{3.8}$$

$$f_{\mathrm{ctd}} = f_{\mathrm{ctk,0.05}}/1.5 \tag{3.9}$$

It should be noted that **the value of f_{ctd} should not be derived from the splitting tensile strength** of concrete.

The characteristic length, l_{cs} is taken as:

$$l_{\mathrm{cs}} = \min\{s_{\mathrm{rm}}, y\} \tag{3.10}$$

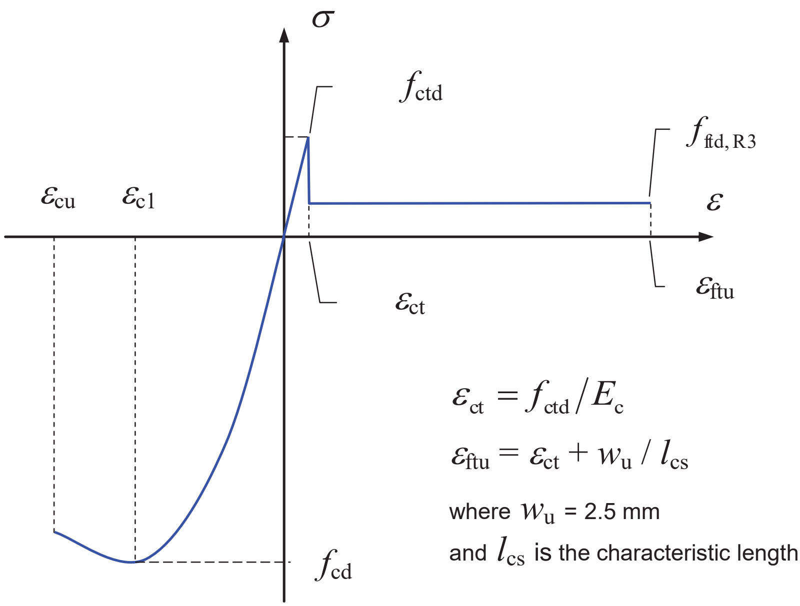

Figure 3.2. Stress-strain relation with a constant-drop relationship in tension

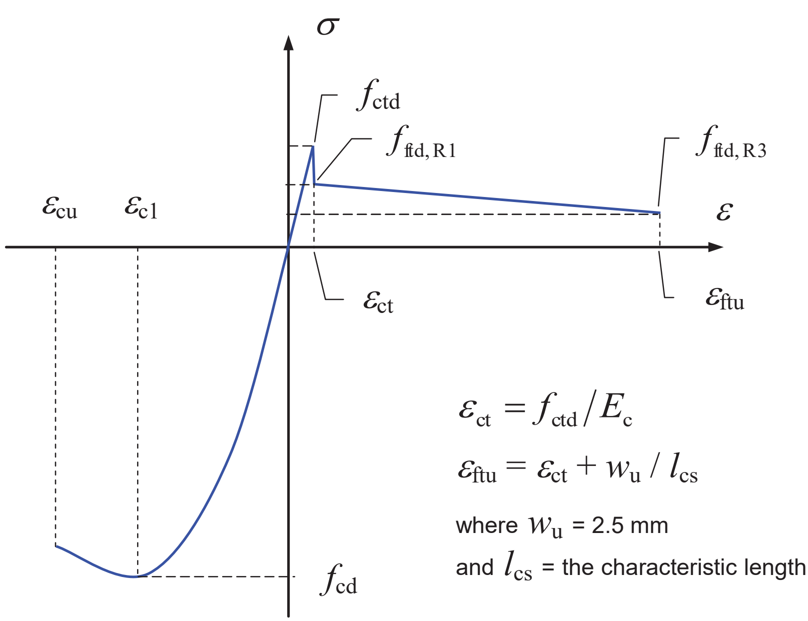

Figure 3.3. Stress-strain relation with a linear descending-drop relationship in tension

where s_{rm} is the mean cracking and y is the distance between the neutral axis and the extreme tensile fibre of the section. Conservatively, $l_{\mathrm{cs}} = 0.8h$ may be assumed for members with conventional bar reinforcement.

For sections without conventional bar reinforcement where crack dominates the behavior, and for slabs, $y = h$ is assumed.

As a simplification, $\varepsilon_{\mathrm{Ftu}}$ may be taken as 0.02.

Commentary

For structural analysis, Draft Annex L gives the following constitutive law for steel fibre reinforced concrete (SFRC):

$$f_{\mathrm{Ft1,ef}} = \kappa_O \cdot 0.37 f_{\mathrm{R,1k}} \tag{C3.7}$$

$$f_{\mathrm{Ft3,ef}} = \kappa_O \cdot (0.57 f_{\mathrm{R,3k}} - 0.26 f_{\mathrm{R,1k}}) \tag{C3.8}$$

$$\varepsilon_{\mathrm{Ftu}} = w_u / l_{\mathrm{cs}} \leq 2.5 \;\; mm \;\; \leq \varepsilon_{\mathrm{Ftud}} \tag{C3.9}$$

$$l_{\mathrm{cs}} = \min\{h; s_{\mathrm{rm}}\} \tag{C3.10}$$

$$s_{\mathrm{rm}} = 0.75 S_{\mathrm{r,max,cal}} \tag{C3.11}$$

$$\varepsilon_{\mathrm{ctm}} = f_{\mathrm{ctm}} / E_{\mathrm{cm}} \quad \text{and} \quad \varepsilon_{F,0} = 2\varepsilon_{\mathrm{ctm}} \tag{C3.12}$$

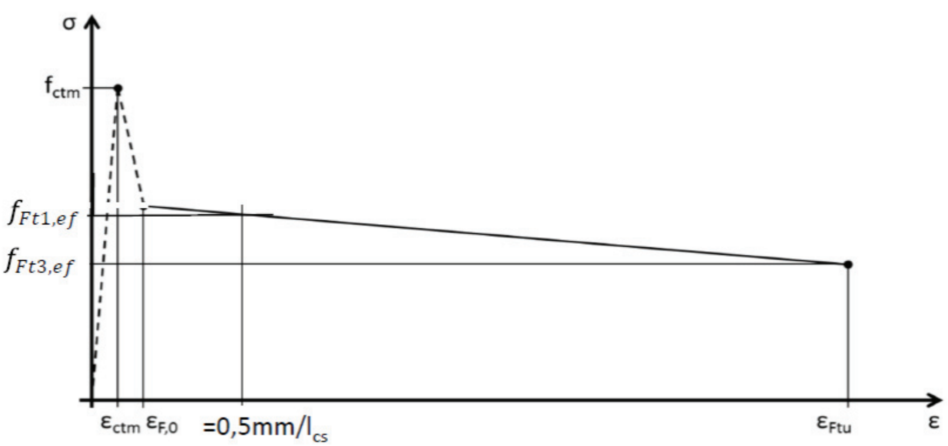

Figure C3.2. Constitutive Law

As a simplification, $l_{cs} = 125$ mm may be used. Also, the recommended value of ε_{Ftud} is 0.02. It is noted that the above does not apply to polymeric fibres.

The relation between σ_c and ε_c in compression, following prEN 1992-1-1 Eq. (5.6), is taken as:

$$\varepsilon_{c1} = 0.7 f_{cm}^{1/3}(1 + 0.03 f_{R1,k}) \tag{C3.13}$$

$$\varepsilon_{cu1} = k\varepsilon_{c1} \tag{C3.14}$$

$$k = 1 + \frac{20}{\sqrt{82 - 2.2 f_{R1,k}}} \tag{C3.15}$$

where f_{cm} and $f_{R1,k}$ are in MPa.

3.4. Design Compressive Strength for FRC without bar reinforcement

For fibre reinforced concrete without bar reinforcement, the design compressive strength may be taken as:

$$f_{cd} = \alpha_{cc,f} f_{ck} / \gamma_f \tag{3.11}$$

where $\alpha_{cc,f}$ is a coefficient accounting for long term effects and of unfavourable effects due to the way in which the load is applied.

The value of $\alpha_{cc,f}$ may be taken as:

$$\alpha_{cc,f} = 0.30 + 0.5 f_{R,3} / f_{R,1} \tag{3.12}$$

where $0.60 \leq \alpha_{cc,f} \leq 0.85$.

Commentary

The NA to SS EN 1992-1-1 specifies a value of α_{cc} equal to 0.85 for reinforced concrete, and a value of $\alpha_{cc,pl}$ equal to 0.6 for plain concrete. Since fibre concrete is more ductile than plain concrete, and in view that a minimum degree of ductility is ensured by requiring $f_{R,3}/f_{R,1}$ to be at least equal to 0.5 [see Eq. (3.3)], it is suggested that the value of $\alpha_{cc,f}$ would depend on the value of $f_{R,3}/f_{R,1}$, for which Eq. (3.10) is proposed. That is, fibre concrete would be treated like plain concrete if $f_{R,3}/f_{R,1} \leq 0.6$ and similar to reinforced concrete if $f_{R,3}/f_{R,1} \geq 1.1$.

The values of $\alpha_{cc,f}$ for ductility classes "a" to "e" are summarized in Table C3.3.

Table C3.3. Values of $\alpha_{cc,f}$

Ductility Class	$f_{\mathrm{R},3}/f_{\mathrm{R},1}$	$\alpha_{\mathrm{cc,f}}$
a	0.5	0.60
b	0.7	0.65
c	0.9	0.75
d	1.1	0.80
e	1.3	0.85

Chapter Four

Durability and Cover to Reinforcement

Steel fibres close to the surfaces of concrete members may be subjected to corrosion, which may potentially cause rust stains. However, due to their relatively small diameter, no spalling from the corrosion of the fibres will occur, and hence the durability of fibre concrete will not be affected. The Standard does not however give any provisions regarding durability of FRC.

For SFRC, the provisions in the Draft Annex L, prEN 1992-1-1, may be followed:

(1) The concrete cover due to durability requirements $c_{\text{min,dur}}$ according to Sect. 6.5 of SS EN 1992-1-1 shall only apply to the embedded reinforcement, not to the steel fibres;

(2) To avoid fibre accumulation, a minimum cover of $c_{\text{min}} = 20$ mm to embedded reinforcement shall be used for all SFRC members;

(3) For design of SFRC in exposure classes XC2-XC4, XD1-XD3, and XS1-XS3, the tensile strength in the outer most tensile fibres if designed to be uncracked, or residual tensile strength if designed to be cracked, shall be disregarded, within a depth of $c_{f,\text{dur}}$ from the exposed surface.

(4) The residual tensile strength of the outermost tensile fibres within depth $c_{\text{f,dur}}$ may be used if protective measures are applied to the steel fibres to protect them for the design life of the structure, or in temporary situations such as during the construction phase.

Commentary

The value of $c_{\text{f,dur}}$ can be taken as 10 mm if the SFRC is designed to be uncracked; or

$$c_{\text{f,dur}} = 10 \cdot \frac{w_{\text{k,cal}}}{w_{\text{lim}}} \geq 10\ mm \tag{C4.1}$$

where $w_{\text{k,cal}}$ and $w_{\text{k,lim}}$ are, respectively, the calculated and limiting crack widths, if it is designed to be cracked.

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Chapter Five

Structural Analysis

5.1. Plastic Analysis

For beams, frames and slabs, the required ductility may be deemed to have been satisfied if all of the following conditions are fulfilled.

(1) For fibre concrete without conventional steel bars:

$$f_{\mathrm{R.1}}/f_{\mathrm{ctk,0.05}} \geq 0.75 \tag{5.1}$$

(2) For fibre concrete subjected to both external loads and restrained shrinkage or thermal movement:

$$f_{\mathrm{R.1}}/f_{\mathrm{ctk,0.05}} \geq 0.75 \tag{5.1}$$

$$f_{\mathrm{R.3}}/f_{\mathrm{ctk,0.05}} \geq 0.65 \tag{5.2}$$

(3) For fibre concrete with conventional steel bars exceeding 50% of the amount needed for concrete without fibres, conditions (2)(i–iii) in Section 5.6.2, SS EN 1992-1-1 shall be satisfied. In cases when the conventional steel bars is less than 50% of that required for concrete without fibres, either conditions (2)(i–iii) in Section 5.6.2, SS EN 199-1-1, or conditions (5.1) and (5.2) indicated above together with condition (2)(iii) of Section 5.6.2 shall be satisfied.

Commentary

Conditions (5.1) and (5.2) are used for members with a nominal thickness not exceeding 400 mm. For thicker members, a special investigation is needed.

Note: Conditions (2)(i)–(iii) of Section 5.6.2, SS EN 1992-1-1 are as follows.

(i) the area of tensile reinforcement is such that at any section, $x_{\mathrm{u}}/d \leq 0.25$ for C50/60 and below, and, $x_{\mathrm{u}}/d \leq 0.15$ for C55/67 and above;
(ii) reinforcing steel is either Class B or C;
(iii) the ratio of moments at intermediate supports to the moments in the span should be between 0.5 and 2.

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5.2. Rotation Capacity

For fibre concrete members with a nominal thickness not exceeding 400 mm, the basic allowable plastic rotation, $\theta_{\mathrm{pl,d}}$, is taken as 10 mrad. For thicker members, a special investigation is needed.

Commentary

Draft Annex L states that:

(1) *Plastic analysis of SFRC structures without any direct check of rotation capacity may be used for the ultimate limit state analysis of the following structure types:*

   a. *Foundations and slabs supported directly on ground.*
   b. *For statically indeterminate rafts and slabs on piles subject to the following conditions:*
      - *the ductility class is at least c; and*
      - *if the member is needed for structural stability* $f_{\mathrm{R,3k}} / f_{\mathrm{ct,flm}} \geq 1.0$.
   c. *For statically indeterminate elevated slabs subject to the following conditions:*
      - *the ductility class is at least c; and*
      - $f_{\mathrm{R,3k}} / f_{\mathrm{ct,flm}} \geq 1.0$.

      where $f_{\mathrm{ct,flm}}$ *is the mean flexural tensile strength.*

(2) *For members not fulfilling the requirements of (1), methods based on plastic analysis, or linear analysis with limited redistribution, shall only be applied where the deformation capacity of the critical sections is demonstrated to be sufficient by calculation for the envisaged failure mechanisms to be formed.*

(3) *Methods to be used for verification of plastic deformation capacity shall take local variations in residual tensile strength into account.*

Chapter Six

Design for Ultimate Limit State

6.1. Bending With or Without Axial Force

The ultimate moment resistance of fibre-reinforced concrete (with/without conventional bar reinforcement) and pre-stressed concrete (fibre concrete and prestressing tendons) sections is determined based on the method of strain compatibility and force equilibrium, same as that for conventional reinforced and prestressed concrete sections.

The strain and stress distributions for fibre-reinforced concrete sections with or without bar reinforcement) at ultimate flexural limit state are shown in Fig. 6.1.

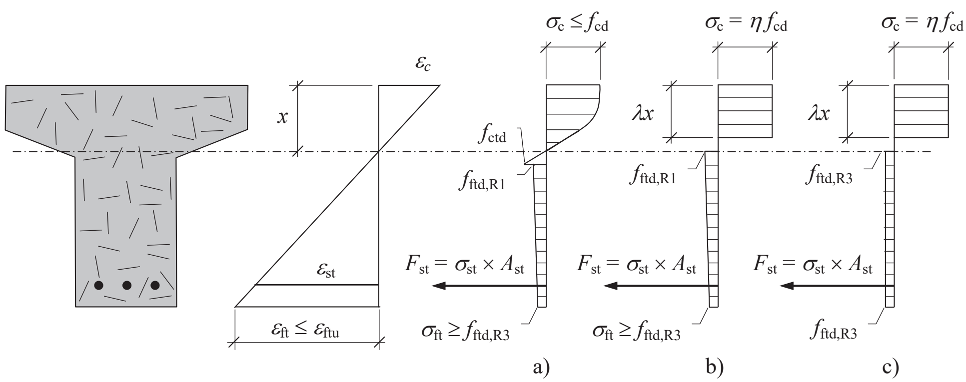

Figure 6.1. Strain and stress distributions at flexural ultimate limit state

The tensile stress distribution in concrete (that is, below the neutral axis) may be taken as one of the following:

(a) according to the stress-section relation with a linear descending-drop relationship in tension as shown in Fig. 3.2, for the general case (Fig. 6.1a);

(b) a stress value equal to $f_{ftd,R1}$ at the neutral axis reducing linearly to $\sigma_{ft}(\geq f_{ftd,R3})$ at the extreme tensile fibre, as a simplification (Fig. 6.1b); or

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(c) a constant stress equal to $f_{\mathrm{ftd,R3}}$ over the height of the tension zone, as a further simplification (Fig. 6.1c).

Based on a constant tensile stress distribution, design charts for flexural design of under-reinforced sections with fibres contributing up to 50% of the moment capacity, are developed and presented in Appendix A.

Commentary

Draft Annex L provides two simplified stress distribution for SFRC in tension, that are similar to those of Figs. 6.1b) and c).

(a) Simplified Rigid-Plastic Approach

This is shown in Fig. C6.1. It may be used for ULS design of a member subjected to bending with or without axial compression, for ductility classes, a, b and c.

It may also be used to determine the ULS moment at the design tensile limit $\varepsilon_{\mathrm{Ftud}}\,(= 0.002)$ *for classes d and e.*

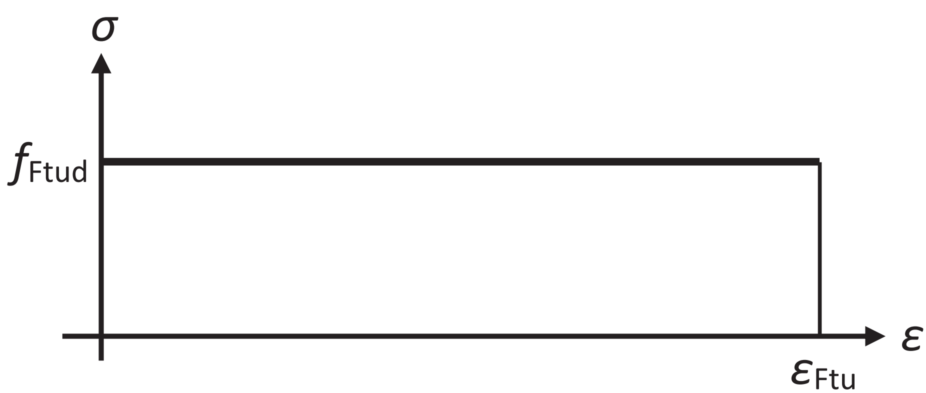

Figure C6.1. Plastic Distribution

(b) Bi-linear residual tensile stress distribution

This is shown in Fig. C6.2.

The values of f_{Ft1d} *and* f_{Ft3d} *are as follows:*

$$f_{\mathrm{Ft1d}} = f_{\mathrm{Ft1,ef}} / \gamma_{\mathrm{SF}} \tag{C6.1}$$

$$f_{\mathrm{Ft3d}} = f_{\mathrm{Ft3,ef}} / \gamma_{\mathrm{SF}} \tag{C6.2}$$

where f_{Ft1d} *and* f_{Ft3d} *are given by (C3.7) and (C3.8) respectively, and* $\gamma_{\mathrm{SF}} = 1.5$.

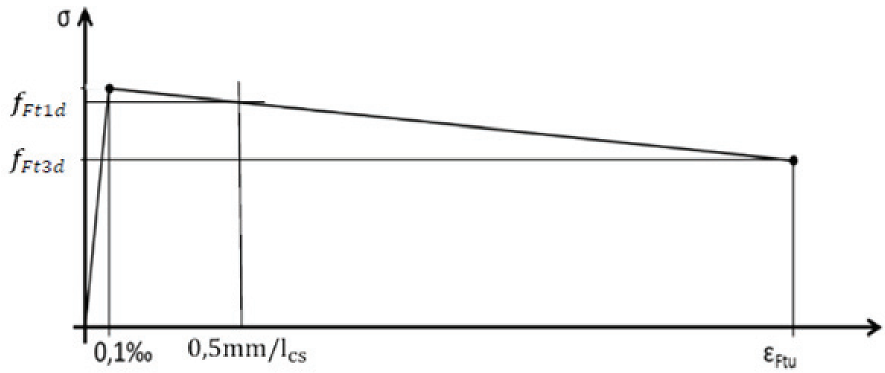

Figure C6.2. Bi-Linear Distribution

For the stress distribution in compression, the parabolic-rectangular stress block is assumed with corresponding values of $\varepsilon_{c2} = 0.0025$ *and* $\varepsilon_{cu} = 0.006$.

For statically indeterminate slabs, the strengths, f_{Ftud} *(C3.5 and C3.6),* f_{Ft1d} *(C6.1) and* f_{Ft3d} *(C6.2) may be increased by a factor*

$$\kappa_G = 1.0 + 0.5A_{ct} \leq 1.5 \tag{C6.3}$$

where A_{ct} *is the area of the tension zone* (m^2) *of the cross-section involved in the failure of an equilibrium system.*

The stress-strain diagrams in Figs. C6.1 and C6.2 may be applied to SFRC structures not satisfying minimum reinforcement requirements (see Section 9.5).

Example 6.1. Determination of Ultimate Moment of Resistance

Figure E6.1(a) shows the cross-section of a 150 mm thick simply-supported, 1-way SFRC slab of concrete class C30/37-$R_1$3/$R_3$2. The tensile steel reinforcement consists of H10 bars placed at 100 mm spacing with an effective depth of 120 mm. Determine the ultimate moment of resistance of the slab given that $f_{yk} = 500$ MPa, $E_s = 200$ GPa, $\gamma_s = 1.15$ and $\gamma_c = 1.5$.

Solution 6.1.
Material Properties:

Characteristic tensile strength of concrete,

$f_{ctk} = 0.21 f_{ck}^{2/3} = 0.21 \times 30^{2/3} = 2.0$ MPa;

Secant modulus of concrete,

$E_c = 22[(f_{ck} + 8)/10]^{0.3} = 22[(30 + 8)/10]^{0.3} = 32.8$ GPa;

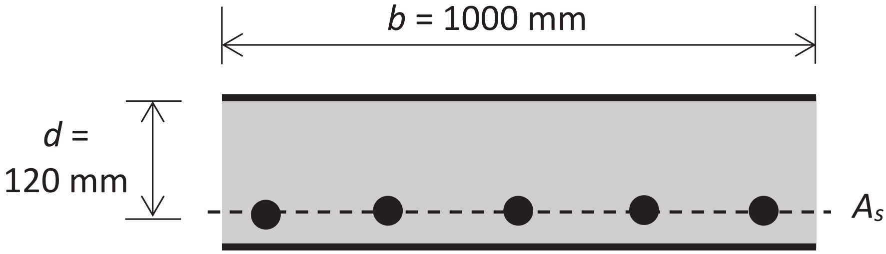

Figure E6.1. (a)

Design tensile strength of concrete,

$f_{\text{ctd}} = f_{\text{ctk}}/\gamma_{\text{c}} = 2.0/1.5 = 1.33$ MPa

Characteristic residual flexural strengths,

$f_{\text{R1}} = 3$ MPa; $f_{\text{R3}} = 2$ MPa;

Characteristic residual tensile strengths,

$$f_{\text{ft,R1}} = 0.45 f_{\text{R1}} = 0.45 \times 3 = 1.35\text{MPa};$$

$$f_{\text{ft,R3}} = 0.37 f_{\text{R3}} = 0.37 \times 2 = 0.74\text{MPa};$$

Design residual tensile strength,
$\eta_{\text{f}} = 1$ (since width $> 5 \times$ thickness);

$\eta_{\text{det}} = 1$ (for simply-supported slabs);

$$\therefore \quad f_{\text{ftd,R1}} = \eta_{\text{f}} \eta_{\text{det}} f_{\text{ft,R1}}/\gamma_{\text{c}} = 1 \times 1 \times 1.35/1.5 = 0.9 \text{ MPa};$$

$$f_{\text{ftd,R3}} = \eta_{\text{f}} \eta_{\text{det}} f_{\text{ft,R3}}/\gamma_{\text{c}} = 1 \times 1 \times 0.74/1.5 = 0.493 \text{ MPa}.$$

Assuming $l_{\text{cs}} = 0.8\,h$, the ultimate tensile strain in SFRC is

$$\varepsilon_{\text{ftu}} = f_{\text{ctd}}/E_{\text{c}} + 2.5/(0.8\,h) = 1.33/32800 + 2.5/(0.8 \times 150) = 0.0209$$

By trial and error, $x = 31.8$ mm.
Steel strain,

$$\varepsilon_{\text{st}} = (d/x - 1) \times 0.0035 = (120/31.8 - 1) \times 0.0035 = 0.0097$$

$$> (f_{\text{yk}}/\gamma_{\text{s}})/E_{\text{s}} = (500/1.15)/(200 \times 10^3) = 0.00217$$

$\therefore$ the steel bars yield at ultimate limit state.

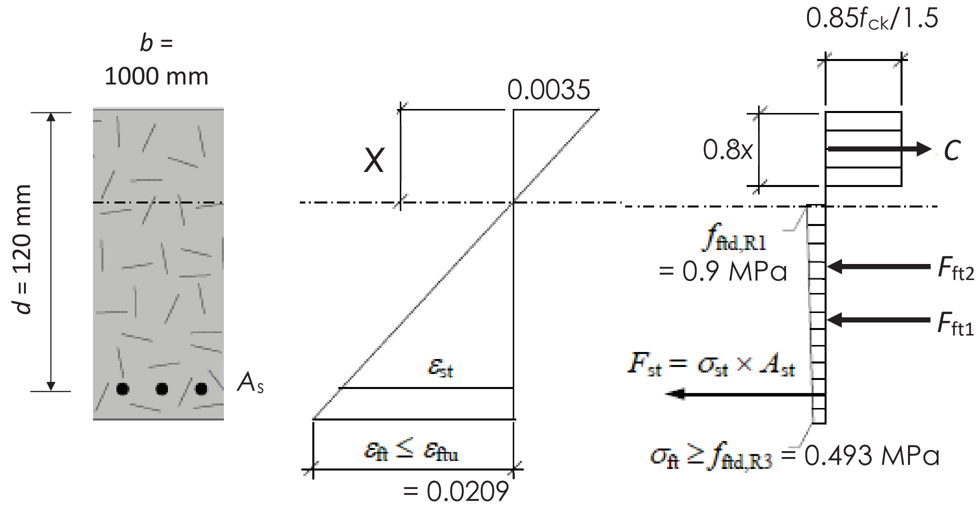

Figure E6.1. (b)

Extreme tensile strain in SFRC,

$$\varepsilon_{ft} = (h/x - 1) \times 0.0035 = (150/31.8 - 1) \times 0.0035 = 0.0130 < \varepsilon_{ftu} \quad (\because \text{ok})$$

The internal forces and their distances from the neutral axis, with the respective moments about the neutral axis are calculated and tabulated as follows.

	Forces (kN/m)	Dist from n.a. (mm)	M (kNm/m)
F_{st}	341.3	88.2	30.1
F_{ft1}(rectangular stress distribution)	76.4	59.1	4.5
F_{ft2}(triangular stress distribution)	15.0	39.4	0.6
C	432.7	19.1	8.3
Sum	0.0	–	43.5

The *ultimate moment of resistance* of the SFRC slab is 43.5 kNm/m. It may be noted that for an identical slab without steel fibres (*see* solution to Example 6.2), the ultimate moment of resistance is 37.6 kNm/m.

The solution can also be obtained using the *design charts in Appendix A*. Noting that $d/h = 120/150 = 0.8$, $A_s f_{yd}/bdf_{ck} = 78.5 \times (500/1.15)/(100 \times 120 \times 30) = 0.095$; and $f_{ft,R3}/f_{ck} = 0.94/30 = 0.025$, Chart A-1 gives

$M_{Ed}/bd^2 f_{ck} = 0.10$, or $M_{Ed} = 0.10 \times 1000 \times 120^2 \times 30 \times 10^{-6} = 43.2$ kNm, and $x = 0.26\ d = 0.26 \times 120 = 31.2$ mm.

Example 6.2. Partial Replacement of Flexural Reinforcement by Steel Fibres

The tensile steel reinforcement of a 150 mm thick C30/37 concrete slab consists of H10 bars placed at 100 mm spacing with an effective depth of 120 mm. Given that $f_{yk} = 500$ MPa, $E_s = 200$ GPa, $\gamma_s = 1.15$ and $\gamma_c = 1.5$, $\eta_f = 1$ and $\eta_{det} = 1$. Determine the required characteristic residual flexural tensile strength, $f_{R,3k}$, for the slab section to provide the same moment capacity if the bar spacing is increased to 125 mm, assuming a constant stress distribution in FRC as shown in Fig. 6.1(c).

Solution 6.2.

(a) *Moment capacity of section with H10 bars bar at 100 mm spacing and without fibres:*
Assuming neutral axis depth at flexural ULS, $x < 0.45$d, the tension steel bars would have yielded. Therefore, with $b = 1000$ mm, $A_s = 786$ mm^2, $d = 120$ mm, equilibrium of internal forces on the section gives:

$$b(0.8x)(0.85 f_{ck}/\gamma_c) = A_s f_{yk}/\gamma_s$$

or

$$x = 786 \times (500/1.15)/(1000 \times 0.8 \times 0.85 \times 30/1.5)$$

$$= 25.1 \text{ mm } < 0.45d = 54 \text{ mm.}$$

Therefore, the moment capacity is:

$$M_{Rd} = A_s f_{yd}(d - 0.4x) = 786 \times (500/1.15) \times (120 - 0.4 \times 25.1) \times 10^{-6}$$

$$= 37.6 \text{ kNm/m}$$

(b) *Section with H10 bars bar at 125 mm spacing and with fibres:*
In this case, $A_s = 628$ mm^2. Referring to Fig. E6.2, equilibrium of internal forces on the section gives:

$$F_f = b(0.8x)(0.85 f_{ck}/\gamma_c) - A_s f_{yk}/\gamma_s$$

$$= [1000(0.8x)(0.85 \times 30/1.5) - 628 \times 500/1.15] \times 10^{-3}$$

$$\therefore \quad F_f = 13.6x - 273 \quad \text{(kN)} \tag{E6.2a}$$

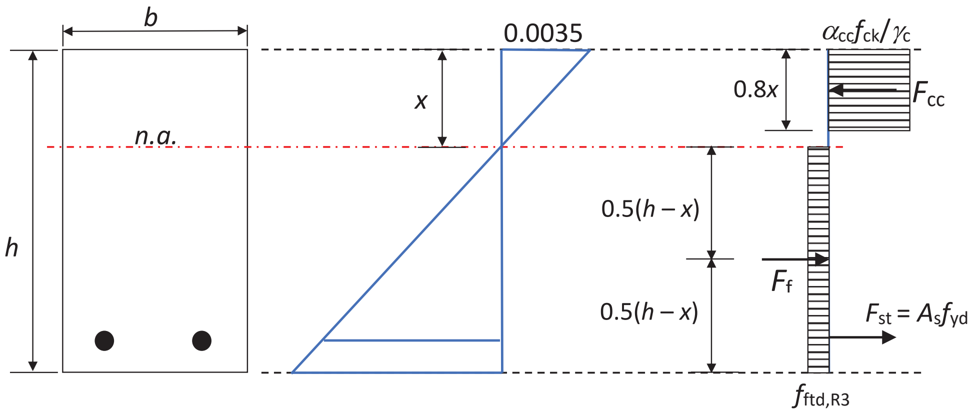

Figure E6.2.

Moment equilibrium gives:

$$M_{Ed} = F_f[0.6x + 1/2(h - x)] + A_s f_{yd}(d - 0.4x)$$

$$= F_{ft}(0.5h + 0.1x) + A_s f_{yd}(d - 0.4x)$$

Substituting $M_{Ed} = 37.6$ kNm, $d = 120$ mm and $h = 150$ mm into the equation gives:

$$37.6 \times 10^3 = F_f(75 + 0.1x) + 273.0(120 - 0.4x) \tag{E6.2b}$$

Solving (E6.2a) and (E6.2b) gives:

$$x = 27.5 \text{ mm and } F_f = 101.0 \text{ kN}$$

Check: $\varepsilon_{ft} = (h/x - 1) \times 0.0035 = (150/27.5 - 1) \times 0.0035 = 0.0156 < \varepsilon_{ftu} = 0.0209 \quad (\therefore \text{ ok})$

Referring to Fig. E6.2, $F_{ft1} = b(h - x)f_{ftd,R3}$, therefore:

$$f_{ftd,R3} = F_{ft1}/[b(h - x)] = 101.0 \times 10^3/[1000 \times (150 - 27.5)] = 0.82 \text{ MPa}$$

Hence,

$$f_{R,3} = 1.5 f_{ftd,R3}/0.37 = 3.3 \text{ MPa}$$

Therefore, a minimum class corresponding to 3d or 4c is required. The same value can be obtained from the *design chart given in Appendix A*, with

$d/h = 120/150 = 0.8$. For the section with H10-100 bars and without fibres:

$$A_s f_{yd}/bdf_{ck} = 786 \times (500/1.15)/(1000 \times 120 \times 30) = 0.095; K_f/K = 0;$$

Therefore, from Chart A-1,

$$K = M_{Ed}/bd^2 f_{ck} = 0.087; x/d = 0.21$$

or $M_{Ed} = 0.0870 \times 1000 \times 120^2 \times 30 \times 10^{-6} = 37.6$ kNm;

$$x = 0.21 \times 120 = 25.2 \text{ mm}$$

If the bar spacing is reduced to 125 mm, then:

$$A_s f_{yd}/bdf_{ck} = 628 \times (500/1.15)/(1000 \times 120 \times 30) = 0.076$$

From Chart A-1, with $M_{Ed}/bd^2 f_{ck} = 0.087$, one obtains:

$$K_f/K \approx 0.2; x/d \approx 0.23; f_{ftd,R3}/f_{ck} \approx 0.03$$

Therefore, $x = 0.23 \times 120 = 27.5$ mm, and $f_{ftd,R3} = 0.03 \times 30 = 0.9$ MPa.
Hence, $f_{R,3} = 1.5 \times 0.9/0.37 = 3.6$ MPa.

6.2. Shear

The design shear resistance of fibre reinforced concrete without shear reinforcement is taken as:

$$V_{Rd,cf} = \{(0.18/\gamma_c) \cdot k \cdot [100\rho_l(1 + 7.5 f_{ft,R3}/f_{ctk}) \cdot f_{ck}]^{1/3} + 0.15\sigma_{cp}\} \cdot b_w d$$

$$\geq \{0.035\mathrm{k}^{3/2} f_{ck}^{1/2} + 0.15\sigma_{cp}\} \cdot b_w d \qquad (6.1)$$

where

$k = 1 + (200/d)^{1/2} \leq 2$;

ρ_l = tensile steel ratio ($= A_s/b_w d$); and

σ_{cp} = axial stress due to loading or prestress.

Equation (6.1) follows the provision in MC2010, which applies to SFRC. Limited studies at NUS have indicated that Eq. (6.1) is equally applicable to macro-synthetic fibre-reinforced concrete beams.

The standard does not provide information on the contribution of fibres towards the shear resistance of fibre-reinforced concrete members with conventional shear reinforcement.

Following the requirements of SS EN 1992-1-1, no shear reinforcement is required if $V_{Ed} \leq V_{Rd,cf}$, except in beams where a minimum shear reinforcement ratio as follows, $\rho_{w,min}$, is to be provided.

$$\rho_{w,min} = A_{sw,min}/(s \cdot b_w \cdot \sin\alpha)$$

$$= (0.08\sqrt{f_{ck}})/f_{yk} \tag{6.2}$$

in which

$A_{sw,min} =$ area of shear reinforcement within s;

$s =$ spacing of shear reinforcement;

$b_w =$ breadth of web of the member; and

$\alpha =$ angle between shear reinforcement and the longitudinal axis.

For members requiring shear reinforcement using both transverse steel reinforcement (i.e., links) and steel fibres, *MC2010* considers the shear resistance to be equal to the sum of $V_{Rd,s}$ and $V_{Rd,cf}$, where $V_{Rd,s}$ is the design value of the shear force which can be sustained by the yielding shear reinforcement. However, in line with *Draft Annex L* [see Eq. (C6.7)], it is recommended by this Guide to take:

$$V_{Rd} = 0.75V_{Rd,s} + V_{Rd,cf} \leq V_{Rd,max} \tag{6.3}$$

where $V_{Rd,s} = (A_{sw}/s)zf_{wd} \cot\vartheta$ and $V_{Rd,max}$ is the shear resistance based on the capacity of struts in the truss model (See SS EN 1991-1-1: 2008 for definitions of terms.)

Commentary

For SFRC with longitudinal bars in the tensile zone, Draft Annex L gives the equivalent value of $V_{Rd,cf}$ as $\tau_{Rd,cF}$ (in MPa) where

$$\tau_{Rd,cF} = \eta(0.6/\gamma_c)(100\rho_l f_{ck} d_{dg}/d)^{1/3} + f_{Ftud}$$

$$\geq \eta\tau_{Rd,cF} + f_{Ftud} \tag{C6.4}$$

in which

$$\eta = \max\{(1/(1 + 0.43f_{Ftuk}{}^{2.85})); 0.4\} \tag{C6.5}$$

Shear reinforcement may be omitted where the design shear stress, τ_{Ed}, is less than or equal to $\tau_{Rd,cF}$. However, minimum shear reinforcement should be provided where needed. The minimum shear reinforcement ratio $\rho_{Fw,min}$, is taken as:

$$\rho_{Fw,min} = \rho_{w,min} - f_{Ftu,ef}/f_{yk} \geq 0 \tag{C6.6}$$

Where steel fibres are used and $f_{\mathrm{Ftu,ef}} \geq 0.08\sqrt{f_{\mathrm{ck}}}$, there is no need for minimum shear reinforcement with the value of $\rho_{\mathrm{w,min}}$ given by Eq. (6.2).

For SFRC members requiring shear reinforcement, i.e., $\tau_{\mathrm{Ed}} > \tau_{\mathrm{Rd,cF}}$, and having steel fibres and tensile longitudinal bars, the design shear resistance should be taken as:

$$\tau_{\mathrm{Rd,sF}} = \eta_\sigma \rho_w f_{\mathrm{yd}} + \eta_{\mathrm{F}} f_{\mathrm{Ftud}} \tag{C6.7}$$

where $\eta_\sigma = 0.75$ and $\eta_{\mathrm{F}} = 1.0$.

For SFRC without longitudinal bars or prestressing tendons in the tensile zone, the design shear strength should be taken as:

$$\tau_{\mathrm{Rd,cF}} = f_{\mathrm{Ftud}} \tag{C6.8}$$

Example 6.3. Determination of Shear Capacity

Determine the design shear capacity of a 400 mm wide, 600 mm deep SFRC beam with four H32 bars placed at an effective depth of 540 mm. The concrete class is C30/37-$\mathrm{R_1}$3/$\mathrm{R_3}$2. Also, $f_{\mathrm{yk}} = 500$ MPa.

Solution 6.3.

For concrete class C30/37-$\mathrm{R_1}$3/$\mathrm{R_3}$2, the values of f_{ctk} [refer SS EN 1992-1-1:2008, Table 3.1] is

$$f_{\mathrm{ctk}} = 0.21 f_{\mathrm{ck}}^{2/3} = 0.21 \times 30^{2/3} = 2.0 \text{ MPa}$$

The value of $f_{\mathrm{R,3}}$ is 2.0 MPa [refer to this Guide, Table 3.2] is 2.0 MPa; hence, from Eq. (3.4),

$$f_{\mathrm{ft,R3}} = 0.37 f_{\mathrm{R,3}} = 0.37 \times 2.0 = 0.74 \text{ MPa}.$$

Also, $A_{\mathrm{s}} = 4 \times 804 = 3216 \text{ mm}^2$; $k = 1 + \sqrt{(200/540)} = 1.61 < 2$; $\rho_{\mathrm{l}} = 3216/(400 \times 540) = 1.489\% < 2\%$; therefore, from Eq. (6.1),

$$V_{\mathrm{Rd,cf}} = (0.18/1.5) \times 1.61 \times (1.489 \times (1 + 7.5 \times 0.74/2.0) \times 30)^{1/3}$$

$$\times 400 \times 540 \times 10^{-3} = \underline{230.3} \text{ kN}$$

$$> 0.035 \times 1.61^{3/2} \times 30^{1/2} \times 400 \times 540 \times 10^{-3} = 84.5 \text{ kN}$$

It is noted that without fibres, the shear capacity of the section would have been equal to 147.9 kN, indicating that the introduction of fibres led to an increase of 55.7% in shear capacity.

Example 6.4. Determination of Shear Reinforcement

Determine the required shear reinforcement for the 400 mm wide, 600 mm deep SFRC beam in Example 6.3, if it is to carry a design shear force of 500 kN. The concrete class is C30/37-$R_1 3/R_3 2$. Also, $f_{yk} = 500$ MPa, $z = 0.9\ d$, and assume $\vartheta = 45°$.

Solution 6.4.
Maximum shear resistance based on strut capacity:

$$\begin{aligned} V_{Rd,max} &= b_w z \nu f_{cd} / (\cot\vartheta + \tan\vartheta) \\ &= [400 \times (0.9 \times 540) \times 0.6(1 - 30/250) \times (30/1.5)/2] \times 10^{-3} \\ &= 1026 \text{ kN} \end{aligned}$$

Therefore, $V_{Rd,cf}(= 230$ kN) [see Example 6.3] $< V_{Ed}(= 500$ kN) $< V_{Rd,max}$ $(= 1026$ kN). Hence, the section size is adequate, and shear reinforcement is required. The required shear reinforcement is given by Eq. (6.3) as:

$$\begin{aligned} A_{sw}/s &= [(V_{Ed} - V_{Rd,cf})/0.75]/(z f_{ywd} \cot\vartheta) \\ &= [(500 - 230) \times 10^3/0.75]/[(0.9 \times 540) \times (500/1.15) \times 1] = 1.70 \text{ mm} \end{aligned}$$

Therefore, provide H13 double-legged closed links at 150 mm spacing; $(A_{sw}/s)_{prov.} = 1.76$ mm.

6.3. Torsion

No specific clauses are given for torsion resistance of fibre-reinforced concrete. However, in Eq. (6.31) of the Standard SS 674:2021, which specifies the criterion for minimum reinforcement to be provided in approximately rectangular solid sections under combined shear and torsion, the value of $V_{Rd,cf}$ as defined in Eq. (6.1) can be used to replace $V_{Rd,c}$, that is,

$$T_{Ed}/T_{Rd,c} + V_{Ed}/V_{Rd,cf} \leq 1.0 \tag{6.3}$$

Commentary

Draft Annex L of prEN 1992-1-1 gives the torsional resistance based on yielding of shear reinforcement and longitudinal reinforcement, respectively, of SFRC members as:

$$\tau_{\text{Rd,swF}} = \eta_{\text{ss}}\, \tau_{\text{Rd,sw}} + \eta_{\text{F}} f_{\text{Ftud}} \geq \tau_{\text{Rd,sw}} \tag{C6.9a}$$

$$\tau_{\text{Rd,slF}} = \eta_{\text{ss}}\, \tau_{\text{Rd,sl}} + \eta_{\text{F}} f_{\text{Ftud}} \geq \tau_{\text{Rd,sl}} \tag{C6.9b}$$

where $\tau_{Rd,sw}$ and $\tau_{\text{Rd,sw}}$ are given by:

$$\tau_{\text{Rd,sw}} = (A_{\text{sw}}/s) f_{\text{ywd}} \cot\vartheta / t_{\text{ef}} \tag{C6.10a}$$

$$\tau_{\text{Rd,sl}} = (\Sigma A_{\text{sl}} f_{\text{yd}})/(u_{\text{k}} t_{\text{ef}} \cot\vartheta) \tag{C6.10b}$$

and $\eta_{\text{F}} = 1.0$ and $\eta_{\text{F}} = 0.75$.

Where a member is subject to torsion in combination with shear and bending, fibre contribution should be used either:

(a) to resist torsion only; or
(b) to resist tensile forces due to shear and bending, and disregarded in axial tension and torsional resistances.

Minimum reinforcement shall always be provided for beams.

6.4. Punching

For fibre concrete slabs and column bases containing conventional longitudinal bar reinforcement but without shear reinforcement, the punching shear resistance is given by:

$$\begin{aligned} v_{\text{Rd,cf}} &= (0.18/\gamma_{\text{c}}) \cdot k \cdot [100\rho_{\text{l}}(1 + 7.5 f_{\text{ft,R3}}/f_{\text{ctk}}) \cdot f_{\text{ck}}]^{1/3} + 0.15\sigma_{\text{cp}} \\ &\geq 0.035 k^{3/2} f_{\text{ck}}^{1/2} + 0.15\sigma_{\text{cp}} \end{aligned} \tag{6.4}$$

For fibre concrete *ground-supported* slabs and column bases without conventional bar reinforcement,

$$v_{\text{Rd,cf}} = v_{\text{Rd,f}} = (k/2) \cdot C \cdot f_{\text{R,3}}/\gamma_{\text{c}} \tag{6.5}$$

where k is a factor to account for size (thickness) effect as defined in Eq. (6.1) and $C = 0.45$. In applying Eq. (6.5), the value of d may be taken as equal to 0.75 times the overall slab thickness. It is noted that the value of k is equal to 2 for a value of d equal to or greater than 200 mm.

Commentary

Draft Annex L of prEN 1992-1-1 gives the design punching shear stress resistance of FRC slabs with flexural bar reinforcement but without shear reinforcement as:

$$\tau_{\mathrm{Rd,cF}} = \eta_{\mathrm{c}}\, \tau_{\mathrm{Rd,c}} + \eta_{\mathrm{F}} f_{\mathrm{Ftud}} \tag{C6.11}$$

where $\eta_{\mathrm{c}} = \tau_{\mathrm{Rd,c}} / \tau_{\mathrm{Ed}} \leq 1.0$ *and* $\eta_{\mathrm{F}} = 1.0$*, in which* $\tau_{\mathrm{Rd,c}} (\geq \tau_{\mathrm{Rdc,min}})$ *is the design punching shear stress resistance (in MPa) of RC slabs without shear reinforcement, and* τ_{Ed} *is the design shear stress.*

Equation (C6.11) is however not applicable to members subject to punching shear in combination with axial tension.

In FRC slabs with flexural reinforcement complying with detailing rules with respect to structural safety, serviceability, durability and robustness, the shear reinforcement where required may be obtained from:

$$\begin{aligned} \tau_{\mathrm{Rd,cs}} &= \eta_{\mathrm{c}}\, \tau_{\mathrm{Rd,c}} + \eta_{\sigma} \rho_{\mathrm{w}} f_{\mathrm{ywd}} + \eta_{\mathrm{F}} f_{\mathrm{Ftud}} \\ &\geq \rho_{\mathrm{w}} f_{\mathrm{ywd}} + \eta_{\mathrm{F}} f_{\mathrm{Ftud}} \end{aligned} \tag{C6.8}$$

where η_{c} *and* η_{σ} *are defined in Clause 8.4.4(1) of prEN 1992-1-1, and* $\eta_{\mathrm{F}} = 1.0$.

6.5. Others

The Standard SS 674:2021 gives additional clauses related to "6.7 Partially Loaded Areas" and "6.8 Fatigue".

- **Partially Loaded Areas [Cl. 6.7(4)]**

The Standard states that the reinforcement required for the tensile force due to the effect of the action can be designed in accordance with Cl. 6.5 Design with strut and tie models, SS EN 1992-1-1:2008. This should be under-stood as that the design should be carried out without considering the presence of fibres.

Commentary

Draft Annex L of prEN 1992-1-1 gives more specific guidance as follows: "Fibres may be used to replace the transverse reinforcement required to resist transverse tensile stresses arising from action effects in partially loaded areas."

For the design, the idealized tensile post-cracking behavior (Figs. C3.1, C6.1, C6.2) may be used. It is noted that in all cases, the fibre orientation factor $\kappa_{\mathrm{o}} = 0.5$ *shall be used. Also, other idealized stress-crack or tensile stress-strain relations may be used if they adequately represent the behavior of the SFRC considered.*

• Fatigue [Cl. 6.8.2(1) & 6.8.7]

For the purpose of fatigue verification [Cl. 6.8.2(1)], the stress calculation should account for the tensile strength of fibre concrete (see Fig. 6.1). For simiplicity, however, the contribution of fibres may be neglected.

Also, for the verification of concrete under compression or shear, although it is known that inclusion of fibres improves the fatigue resistance, this should be ignored due to limited test data and available design models.

Commentary

It is noted that steel fibres can improve the fatigue behaviour of concrete, particularly in low frequency ($\leq$ 1 Hz) compressive fatigue and flexural fatigue in the post-cracking regime, up to certain fibre content. However, due to lack of general and validated design rules, Draft Annex L to prEN 1992-1-1 recommends that the contribution of fibres should be neglected for the fatigue vertification of SFRC members.

Chapter Seven

Design for Serviceability Limit State

7.1. Stress Limitation

In general, the provisions for structural concrete as given in Clause 7.2, SS EN 1992-1-1 apply to fibre reinforced concrete. For fibre concrete members with fibres only, that is, without conventional steel bars, crack width shall be limited to avoid unacceptable cracking or deformation.

Commentary

MC 2010 further states that in structural FRC elements with a tension softening behavior after cracking, there is no need for tensile stress verification if the element is verified at the ultimate limit state (ULS). For structural elements having a tension hardening behavior after cracking, the principal tensile stress, σ_t, must satisfy the following criterion:

$$\sigma_t \leq 0.6 f_{\mathrm{Ftsk}} \tag{C7.1}$$

where f_{Ftsk} is the characteristic value of f_{Fts}, given in Eq. (C3.4).

7.2. Crack Control

For members with both fibres and conventional reinforcing or prestressing steels, the allowable crack widths follow those given for structural concrete members given in SS EN 1992-1-1. For fibre concrete members without conventional steel reinforcement, the recommended limiting calculated crack width, $w_{\max}$, for Execution Class 2 (Semi-Structural System) and Class 3 (Structural System) fibre-reinforced concrete (see Annex F of Standard) are shown in Table 7.1 with respect to durability for intended working life of L50 (at least 50 years) and L100 (at least 100 years).

The allowable crack widths are generally relaxed for Exposure Class XC and less aggressive exposure classes. They are tightened for Exposure Classes XS3 and XD3, for which corroding steel fibres must be combined conventional bar reinforcement for suspended decks, slabs or beams.

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Table 7.1. w_{max} for Fibre Concrete

Exposure class	L50 w_{max} (mm)	L100 w_{max} (mm)
X0, XC1	see comments	
XC2, XC3	0.5	0.4
XC4	0.4	0.3
XS1, XS2, XD1, XD2	0.3	0.2
XS3*, XD3*	0.2	0.1

* reinforcing bars to be provided.

Commentary

Steel fibres efficiently reduces the tensile strains and crack spacing, leading to reduced crack widths. Draft Annex L recommends stress and crack with limits for normal structural concrete members to be used for members with steel fibres.

For X0 and XC1 exposure classes, and for fibre concrete with non-metallic fibres, crack width has no influence on durability and the limiting crack width may be set to provide generally acceptable appearance.

Minimum Reinforcement Areas

The provisions of Clause 7.3.2, SS EN 1992-1-1 applies to fibre reinforced concrete with conventional bar reinforcement, with the fibre contribution to the tensile resistance taken into account, that is, the minimum reinforcement area for crack control is given by:

$$A_s \cdot \sigma_s = k_c \cdot k \cdot (1 - k_f) \cdot f_{ct,eff} \cdot A_{ct} \tag{7.1}$$

where

$$k_f = f_{ftd,R1} / f_{ctm} \leq 1.0 \tag{7.2}$$

in which $f_{ftd,R1}$ is given by Eq. (3.7), and all other symbols are as defined in SS EN 1992-1-1. For crack control, the value of σ_s should be taken as that needed to satisfy crack width limits according to maximum bar size or bar spacing.

Commentary

Draft Annex L suggested that the reinforcement areas for crack control for normal concrete members may be adjusted for fibre contribution in accord-ance to the requirement for robustness.

Crack Control Without Calculation

For fibre concrete **with bar reinforcement**, crack control may be facilitated as a simplification, by restricting the reinforcing bar diameter or spacing. The maximum bar size $\phi_{\mathrm{s,f}}$ can be obtained by multiplying the maximum bar diameter $\phi{*}_{\mathrm{s}}$ in SS EN 1992-1-1 Table 7.2N by the following factor:

$$\frac{\phi_{\mathrm{s,f}}}{\phi_{\mathrm{s}}^{*}} = \frac{\sigma_{\mathrm{s}} \cdot A_{\mathrm{s}}/b}{11.6(h-d)} \cdot \frac{1}{(1-k_{\mathrm{f}})^{2}} \leq \frac{f_{\mathrm{ct,eff}}}{2.9} \cdot \frac{1}{(1-k_{\mathrm{f}})^{2}} \tag{7.3}$$

where σ_{s} is the steel stress; A_{s} is the area of bar reinforcement within the tensile zone, h is the height, d the effective depth, and b the width of the tensile zone.

Commentary

The prEN1992-1-1 refers this to as "simplified control of cracking", where the maximum bar diameter and bar spacing are given in formula format. With the reformulation, much larger bar diameters are possible than before. Draft Annex L does not give any recommendations on the modification of the formulas for use in fibre-reinforced concrete

Calculation of Crack Widths

For fibre concrete **with bar reinforcement**, the crack width may be calculated as in SS EN 1992-1-1 from:

$$w_k = s_{\mathrm{r,max}} \cdot (\varepsilon_{\mathrm{sm}} - \varepsilon_{\mathrm{cm}}) \tag{7.4}$$

where $s_{\mathrm{r,max}}$ is the maximum crack spacing; $\varepsilon_{\mathrm{sm}}$ is the mean strain in the reinforcement; and $\varepsilon_{\mathrm{cm}}$ is the mean strain in the concrete between cracks.

The strain difference $(\varepsilon_{\mathrm{sm}} - \varepsilon_{\mathrm{cm}})$ can be determined either by: (a) calculating the steel stress assuming a cracked section with the effect of fibres $(f_{\mathrm{ftd,R1}})$ taken into account; or (b) considering a fictitious stress $\sigma_{\mathrm{s,fict}}$ in the reinforcement assuming a cracked section but neglecting the effect of fibres. Accordingly, the strain difference may be calculated from:

$$(\varepsilon_{\mathrm{sm}} - \varepsilon_{\mathrm{cm}}) = \frac{\sigma_{\mathrm{s}} - (k_{\mathrm{t}} + (1 - k_{\mathrm{t}}) \cdot k_{\mathrm{f}}) \cdot \frac{f_{\mathrm{ct,eff}}}{\rho_{\mathrm{p,eff}}} \cdot (1 + \alpha_{\mathrm{e}} \cdot \rho_{\mathrm{p,eff}})}{E_{\mathrm{s}}}$$

$$\geq 0.6 \cdot \frac{\sigma_{\mathrm{s}}}{E_{\mathrm{s}}} \tag{7.5a}$$

where σ_{s} is the steel stress assuming cracked section with the contribution of fibres $(f_{\mathrm{ftd,R1}})$ being considered; or

$$(\varepsilon_{\mathrm{sm}} - \varepsilon_{\mathrm{cm}}) = \frac{(1 - k_{\mathrm{f}}) \cdot \left(\sigma_{\mathrm{s,fict}} - k_{\mathrm{t}} \cdot \frac{f_{\mathrm{ct,eff}}}{\rho_{\mathrm{p,eff}}} \cdot (1 + \alpha_{\mathrm{e}} \cdot \rho_{\mathrm{p,eff}})\right)}{E_{\mathrm{s}}}$$

$$\geq 0.6 \cdot (1 - k_{\mathrm{f}}) \frac{\sigma_{\mathrm{s,fict}}}{E_{\mathrm{s}}} \tag{7.5b}$$

where $\sigma_{\mathrm{s,fict}}$ is the fictitious steel stress assuming cracked section with fibre contribution $(f_{\mathrm{ftd,R1}})$ being ignored.

The value of k_{f} is given by Eq. (7.2), and all other symbols in Eq. (7.5a) and Eq. (7.5b) are as defined in SS EN 1992-1-1.

The maximum crack spacing may be calculated by considering a reduced bond transfer required to initiate a new crack, due to fibre contribution. In situations where bonded reinforcement is provided at spacing $\leq 5(c + \phi/2)$, where c is the cover to the longitudinal reinforcement, and ϕ is the bar diameter, the maximum final crack spacing is obtained from:

$$s_{\mathrm{r,max}} = k_3 \cdot c + k_1 \cdot k_2 \cdot k_4 \cdot (1 - k_{\mathrm{f}}) \cdot \frac{\phi}{\rho_{\mathrm{p,eff}}} \tag{7.6}$$

Where the spacing of bonded reinforcement exceeds $5(c + \phi/2)$, the maximum crack spacing is taken as:

$$s_{\mathrm{r,max}} = 1.3 \cdot (h - x) \cdot (1 - k_{\mathrm{f}}) \tag{7.7}$$

For fibre concrete member **without bar reinforcement**, the maximum surface crack width w_{max} under flexure may be estimated from:

$$w_{\mathrm{max}} = \varepsilon_{\mathrm{ft}} \cdot 2\,(h - x) \tag{7.8}$$

where $\varepsilon_{\mathrm{ft}}$ is the maximum tensile strain for the relevant load combination.

For fibre concrete members **without bar reinforcement** subjected to restraint, the maximum crack width can be estimated from:

$$w_{\text{max}} = \left(R_{\text{ax}} \cdot \varepsilon_{\text{cs}} - \frac{f_{\text{ftd,R1}}}{E_{\text{c}}} \cdot (1 + \phi_{\text{ef}}) \right) \cdot s_{\text{r,max}} \geq 0 \tag{7.9}$$

where ε_{cs} is the strain due to thermal and/or shrinkage moments, R_{ax} (where $0 \leq R_{ax} \leq 1$) is the axial restraint factor. For ground-supported slabs, $s_{r,max}$, can be assumed to be 1/2 of the distance between free-movement joints. Also, R_{ax} should not be chosen less than 0.5. Values for R_{ax} can be found in SS EN 1992-3:2010, Annex L.

Commentary

The prEN 1992-1-1:2018 refers this to as "refined control of cracking". For members reinforced with steel fibres and longitudinal bars, Draft Annex L assumes that the tension stiffening term is unaffected by the fibres, that is,

$$(\varepsilon_{\text{sm}} - \varepsilon_{\text{cm}}) = \frac{\sigma_{\text{s}} - k_{\text{t}} \cdot \frac{f_{\text{ct,ef}}}{\rho_{\text{p,ef}}} \cdot (1 + \alpha_{\text{e}} \cdot \rho_{\text{p,ef}})}{E_{\text{s}}} \geq 0.6 \cdot \frac{\sigma_{\text{s}}}{E_{\text{s}}} \tag{C7.1}$$

Also, the maximum crack spacing is calculated from:

$$s_{\text{r,max,cal}} = \left(2c + 0.28 \frac{\phi}{\rho_{\text{p,ef}}} \right) \cdot \left(1 - \frac{f_{\text{Fts,ef}}}{f_{\text{ctm}}} \right) \tag{C7.2}$$

where $f_{Fts,ef}$ [see Eq. (C3.5) with f_{Ftsk} given by Eq. (C3.3).] is the effective residual tensile strength for the serviceability limit state, and the other parameters are as defined in prEN 1992-1-1.

Draft Annex L does not address crack width in restrained members explicitly.

For SFRC members without longitudinal bars, and structural hardening behavior under bending with or without axial compression, the maximum crack spacing can be determined as:

$$s_{\text{r,max,cal}} = h \tag{C7.3}$$

Example 7.1. Calculation of Crack Width

A simply-supported one-way SFRC slab is designed to carry characteristic dead load, g_k, equal to 5.25 kN/m^2, and characteristic imposed load, q_k, equal to 7.5 kN/m^2. The slab is simply supported over a span of 4 m, and the cross-section of the slab is shown in Fig. E7.1(a). The concrete class is

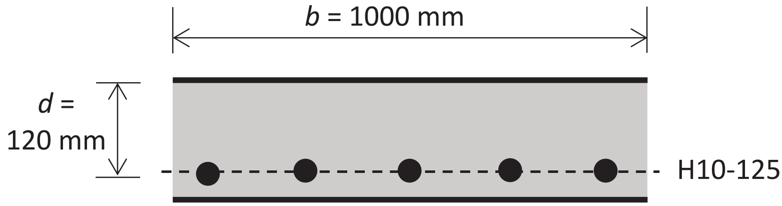

Figure E7.1. (a)

C30/37-R₁3/R₃4. Also, $f_{yk} = 500$ MPa. Calculate the maximum crack width under the total characteristic load.

Solution 7.1.
The mid-span moment due to total characteristic loads, $g_k + q_k$, of 12.75 kN/m^2, is:

$$M_s = 12.75 \times 4^2/8 = 25.5 \text{ kNm/m}$$

For concrete class C30/37-$R_1 3/R_3 4$, and assuming $\eta_f = 1$ and η_{det} =1:

$$\begin{aligned}
f_{ck} &= 30 \text{ MPa};\\
E_{cm} &= 22[(30+8)/10]^{0.3} = 32.8 \text{ GPa};\\
f_{ctm} &= 2.9 \text{ MPa};\\
f_{ftd.R1} &= \eta_f f_{ft,R1} = 1 \times (0.45 \times 3)/1\\
&= 1.35 \text{ MPa};\\
k_f &= f_{ftd.R1}/f_{ctm} = 1.35/2.9 = 0.466
\end{aligned}$$

(a) Crack width calculation using Eq. (7.5a):
Figure E7.1(b) shows the strain and stress diagrams assumed for the section under elastic cracked condition. Following the procedure given in Appendix B, the following values are obtained:
Section curvature,
$1/r = 11.8 \times 10^{-6}$/mm;

$x = 37.6$ mm;

$$\varepsilon_{cc} = (1/r)x = 11.8 \times 10^{-6} \times 37.6 \times 10^{-6}\ \text{mm/mm}$$

$$= 444 \times 10^{-6}\ \text{mm/mm}$$

$$f_{cc} = E_{cm}\varepsilon_{cc} = 32.8 \times 10^{3} \times 444 \times 10^{-6}$$

$$= 14.6\ \text{MPa}$$

$$< 0.40 f_{cm} = 0.40 \times (30 + 8)$$

$$= 15.2\ \text{MPa};$$

linear compressive stress diagram is valid.

$$\varepsilon_{ft} = \varepsilon_{cc}(h/x - 1) = 444 \times (150/37.6 - 1) \times 10^{-6} = 0.00133\ \text{mm/mm}$$

$$< \varepsilon_{ftu} = 0.02087\ \text{mm/mm (see solution to Example 6.1)} \qquad \text{ok}$$

$$\varepsilon_{st} = \varepsilon_{cc}(d/x - 1) = 444 \times (120/37.6 - 1) \times 10^{-6} = 0.000973\ \text{mm/mm}$$

$$< \varepsilon_{yd} = (500/1.15)/(200,000) = 0.002173\ \text{mm/mm}$$

$$\sigma_s = E_s\varepsilon_{st} = 200 \times 10^{3} \times 0.000973 = 195\ \text{MPa}$$

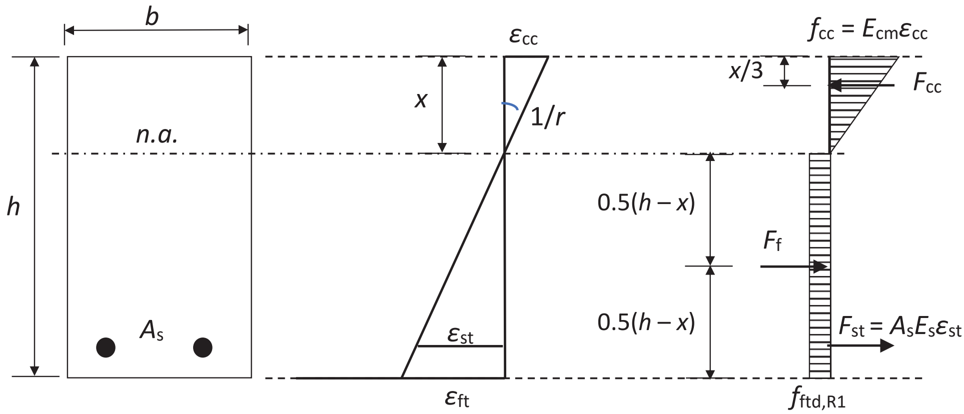

Figure E7.1.

Check for equilibrium:

$$F_{cc} = 1/2bxf_{cc} = 1/2 \times 1000 \times 37.6 \times 32.8 \times 10^3 \times 444 \times 10^{-6} \times 10^{-3}$$

$$= 274 \text{ kN};$$

$$F_f = b(h - x)f_{ftd,R1} = 1000 \times (150 - 37.6) \times 1.35 \times 10^{-3}$$

$$= 152 \text{ kN};$$

$$F_{st} = A_s\,\sigma_s = 628 \times 195 \times 10^{-3}$$

$$= 122 \text{ kN}$$

Therefore, $F_{cc} - F_f - F_{st} = 274 - 152 - 122 = 0$ kN

Check value of moment:

$$M_s = F_{cc}(2x/3) + F_f 1/2(h - x) + F_{st}(d - x)$$

$$= [274 \times 2 \times 37.6/3 + 152 \times 1/2 \times (150 - 37.6) + 122 \times (120 - 37.6)] \times 10^{-3}$$

$$= 6.9 + 8.5 + 10.1 = 25.5 \text{ kNm/m} \qquad \text{ok}$$

Equation (7.5a) gives:

$$f_{ct,eff} = f_{ctm} = 2.9 \text{ MPa};$$

$$h_{eff} = \min\cdot\{2.5(h - d), (h - x)/3, h/2\}$$

$$= \min\cdot\{2.5(150\text{-}120), (150\text{-}37.6)/3, 150/2\}$$

$$= \min\cdot(75, 37.5, 75) = 37.5 \text{ mm}$$

$$\rho_{p,eff} = A_s/bh_{eff} = 628/(1000 \times 37.5)$$

$$= 0.0168$$

$$\alpha_e = E_s/E_{cm} = 200/32.8 = 6.10$$

Therefore, considering that $k_t = 0.6$ for short-term crack width,

$$(\varepsilon_{sm} - \varepsilon_{cm}) = [\sigma_s - (k_t + (1 - k_t)k_f)(f_{ct,eff}/\rho_{p,eff})(1 + \alpha_e \rho_{p,eff})]/E_s$$

$$= [195 - (0.6 + 0.4 \times 0.466)$$

$$\times (2.9/0.0168)(1 + 6.10 \times 0.0168)]/(200 \times 10^3]$$

$$= (195 - 150)/(200 \times 10^3) = 200 \times 10^{-6}$$

$$< 0.6\sigma_s/E_s = 0.6 \times 195/(200 \times 10^3) = 585 \times 10^{-6} \text{ (governs)}$$

Since bar spacing, $s = 125$ mm $< 5(c + \phi/2) = 5 \times 50 = 250$ mm, therefore, from Eq. (7.6), with $k_3 = 3.4, k_4 = 0.425, k_1 = 0.8$ (high bond bars), $k_2 = 0.5$ (bending) (NA to SS EN1992-1-1),

$$S_{r,max} = 3.4c + 0.425 \times 0.8 \times 0.5 \times \phi/\rho_{p,eff}$$

$$= 3.4 \times 25 + 0.425 \times 0.8 \times 0.5 \times 10/0.0168 = 186 \text{ mm}$$

Therefore, from Eq. (7.4).

$$w_k = S_{r,max}(\varepsilon_{sm} - \varepsilon_{cm}) = 186 \times 585 \times 10^{-6} = 0.11 \text{ mm}$$

(b) Crack width calculation using fictitious steel stress, Eq. (7.5b):

$$\rho = A_s/bd = 628/(1000 \times 120) = 0.00523$$

$$\alpha_e \rho = 6.10 \times 0.00523 = 0.0319$$

Ignoring fibre contribution, the neutral axis depth is given by:

$$x/d = \sqrt{[(\alpha_e\rho)^2 + 2\alpha_e\rho]} - \alpha_e\rho = \sqrt{(0.0319)^2 + 2 \times 0.0319]} - 0.0319 = 0.223$$

$$x = 0.223 \times 120 = 26.7 \text{ mm}$$

The moment of inertia is given by:

$$I = bx^3/3 + \alpha_e A_s (d - x)^2 = 1000 \times 26.7^3/3 + 6.1 \times 628 \times (120 - 26.7)^2$$

$$= 39.7 \times 10^6 \text{ mm}^4/\text{m}$$

Under $M_s = 25.5$ kNm/m, the fictitious steel stress is:

$$\sigma_{s,fict} = \alpha_e M_s (d - x)/I = 6.10 \times 25.5 \times 10^6 \times (120 - 26.7)/(39.7 \times 10^6)$$

$$= 366 \text{ MPa}$$

From Eq. (7.5b):

$$\begin{aligned}
&(\varepsilon_{\mathrm{sm}} - \varepsilon_{\mathrm{cm}}) \\
&\quad = (1 - k_{\mathrm{f}})[\sigma_{\mathrm{s,fict}} - k_{\mathrm{t}}(f_{\mathrm{ct,eff}}/\rho_{\mathrm{p,eff}})(1 + \alpha_e \rho_{\mathrm{p,eff}})]/E_{\mathrm{s}} \\
&\quad = (1 - 0.466)[366 - 0.6 \times (2.9/0.0168)(1 + 6.10 \times 0.0168)]/(200 \times 10^3] \\
&\quad = 0.534 \times (366 - 114)/(200 \times 10^3) = 673 \times 10^{-6} \text{ (governs)} \\
&\quad > 0.6(1 - k_{\mathrm{f}})\sigma_{\mathrm{s}}/E_{\mathrm{s}} = 0.6 \times (1 - 0.466) \times 366/(200 \times 10^3) = 586 \times 10^{-6}
\end{aligned}$$

Since bar spacing, $s = 125$ mm $< 5(c + \phi/2) = 250$ mm, therefore, from Eq. (7.6), $S_{\mathrm{r,max}} = 186$ mm. Hence, from Eq. (7.4):

$$w_{\mathrm{k}} = S_{\mathrm{r,max}}(\varepsilon_{\mathrm{sm}} - \varepsilon_{\mathrm{cm}}) = 186 \times 673 \times 10^{-6} = 0.13 \text{ mm}$$

Example 7.2. Crack Control

Verify the serviceability limit state for cracking for the simply-supported one-way SFRC slab in Fig. E7.1 under quasi-permanent loading, equal to $(g_k, +0.5q_k)$ of 9.75 kN/m^2. Given that the creep coefficient, φ, is 1.8, and allowable crack width, w_k, is 0.2 mm.

Solution 7.2. The mid-span moment due to quasi-permanent loading is:

$$M_s = 9.75 \times 4^2/8 = 19.5 \text{ kNm/m}$$

As in Example E7.1,

$$f_{\mathrm{ck}} = 30 \text{ MPa}; E_{\mathrm{cm}} = 32.8 \text{ GPa}; f_{\mathrm{ctm}} = 2.9 \text{ MPa};$$

$$f_{\mathrm{ftd.R1}} = 1.35 \text{ MPa}; k_{\mathrm{f}} = f_{\mathrm{ftd.R1}}/f_{\mathrm{ctm}} = 0.466;$$

Also,

$$E_{\mathrm{c,eff}} = E_{\mathrm{cm}}/(1 + \phi) = 32.8/(1 + 1.8) = 11.7 \text{ GPa},$$

$$\alpha_{\mathrm{e}} = E_{\mathrm{s}}/E_{\mathrm{c,eff}} = 200/11.7 = 17.1$$

(a) Crack width calculation using Eq. (7.5a):

Following the procedure given in Appendix B, the following values are obtained:

Section curvature, $1/r = 11.2 \times 10^{-6}$ / mm; $x = 57.1$ mm;

$$\varepsilon_{cc} = (1/r)x = 11.2 \times 10^{-6} \times 57.1 \times 10^{-6} \text{ mm/mm}$$

$$= 640 \times 10^{-6} \text{ mm/mm}$$

$$f_{cc} = E_{c,eff}\varepsilon_{cc} = 10.9 \text{ MPa } < 0.40 f_{cm} = 15.2 \text{ MPa;}$$

$$\varepsilon_{ft} = \varepsilon_{cc}(h/x - 1) = 0.00104 \text{ mm/mm } < \varepsilon_{ftu} = 0.02087 \text{ mm/mm}$$

$$\varepsilon_{st} = \varepsilon_{cc}(d/x - 1) = 0.000705 \text{ mm/mm } < \varepsilon_{yd} = 0.002173 \text{ mm/mm}$$

$$\sigma_s = E_s\varepsilon_{st} = 140 \text{ MPa}$$

With $f_{ct,eff} = 2.9$ MPa; $h_{eff} = \min\cdot\{2.5(h-d), (h-x)/3, h/2\} = \min\cdot\{2.5(150\text{-}120), (150\text{-}57.1)/3, 150/2\} = \min\cdot(75, 37.5, 75) = 31.0$mm, $\rho_{p,eff} = A_s/bh_{eff} = 628/(1000 \times 31.0) = 0.0203$, and $k_t = 0.4$ for long-term crack width, Eq. (7.5a) gives:

$$(\varepsilon_{sm} - \varepsilon_{cm})$$
$$= [\sigma_s - (k_t + (1-k_t)k_f)(f_{ct,eff}/\rho_{p,eff})(1 + \alpha_e\rho_{p,eff})]/E_s$$
$$= [140 - (0.4 + 0.6 \times 0.466)(2.9/0.0203)(1 + 17.1 \times 0.0203)]/(200 \times 10^3]$$
$$= (140 - 131)/(200 \times 10^3) = 45 \times 10^{-6}$$
$$< 0.6\sigma_s/E_s = 0.6 \times 140/(200 \times 10^3) = 420 \times 10^{-6} \text{ (governs)}$$

Also, as in Example E7.1, from Eq. (7.6),

$$S_{r,max} = 3.4c + 0.425 \times 0.8 \times 0.5 \times \phi/\rho_{p,eff}$$
$$= 3.4 \times 25 + 0.425 \times 0.8 \times 0.5 \times 10/0.0203 = 169 \text{ mm}$$

Therefore, from Eq. (7.4).

$$w_k = S_{r,max}(\varepsilon_{sm} - \varepsilon_{cm}) = 169 \times 420 \times 10^{-6} = 0.07 \text{ mm } < 0.2 \text{ mm} \qquad \text{ok}$$

(b) Crack width calculation using fictitious steel stress, Eq. (7.5b):

$$\rho = A_s/bd = 0.00523; \; \alpha_e\rho = 17.1 \times 0.00523 = 0.0894$$

Ignoring fibre contribution, the neutral axis depth is given by:

$$x/d = \sqrt{[(\alpha_e\rho)^2 + 2\alpha_e\rho]} - \alpha_e\rho = \sqrt{(0.0894^2 + 2 \times 0.0894]} - 0.0894 = 0.343$$

$$x = 0.343 \times 120 = 41.1 \text{ mm}$$

The moment of inertia is given by:

$$I = bx^3/3 + \alpha_e A_s (d-x)^2 = 1000 \times 41.1^3/3 + 17.1 \times 628 \times (120 - 41.1)^2$$
$$= 90.0 \times 10^6 \text{ mm}^4/\text{m}$$

Under $M_s = 19.5$ kNm/m, the fictitious steel stress is:

$$\sigma_{s,fict} = \alpha_e M_s (d-x)/I = 17.1 \times 19.5 \times 10^6 \times (120 - 41.1)/(90.0 \times 10^6)$$
$$= 292 \text{ MPa}$$

From Eq. (7.5b):

$$(\varepsilon_{sm} - \varepsilon_{cm})$$
$$= (1-k_f)[\sigma_{s,fict} - k_t(f_{ct,eff}/\rho_{p,eff})(1+\alpha_e\rho_{p,eff})]/E_s$$
$$= (1-0.466)[292 - 0.4 \times (2.9/0.0203)(1+17.1 \times 0.0203)]/(200 \times 10^3]$$
$$= 0.534 \times (292-77)/(200 \times 10^3) = 574 \times 10^{-6} \text{ (governs)}$$
$$> 0.6(1-k_f)\sigma_s/E_s = 0.6 \times (1-0.466) \times 292/(200 \times 10^3)$$
$$= 467 \times 10^{-6}$$

From (a), $S_{r,max}$= 169 mm. Therefore, using Eq. (7.4),

$$w_k = S_{r,max}(\varepsilon_{sm} - \varepsilon_{cm}) = 169 \times 574 \times 10^{-6}$$
$$= 0.10 \text{ mm } < 0.2 \text{ mm} \qquad \text{ok}$$

It may be noted that for a reinforced concrete slab of the same cross section and moment capacity but with H10 bars at 100 mm spacing and without fibres (see Example 6.2), the steel stress under the quasi-permanent loading is 236 MPa, $(\varepsilon_{sm} - \varepsilon_{cm}) = 822 \times 10^{-6}$ mm/mm, $S_{r,max} = 161$ mm, and the maximum crack width is 0.13 mm. Hence, the maximum crack width has been reduced by more than 20 percent by partial replacement of steel bars with fibres.

(c) Crack control based on limiting bar size:

From Table 7.2N (SS NA), for a steel stress, $\sigma_s = 292$ MPa, and $w_k = 0.2$ mm, the maximum bar diameter, ϕ_s^* is 7.4 mm. Therefore, applying Eq. (7.3) gives:

$$\phi_{s,f} = \phi_s^*(\sigma_s A_s)/[11.6b(h-d)(1-k_f)^2]$$

$$= 7.4 \times 292 \times 628/[11.6 \times 1000 \times (150-120) \times (1-0.466)^2]$$

$$= 7.4 \times 1.85 = 13.7 \text{ mm (governs)}$$

$$< (f_{ct,eff}/2.9)/(1-k_f)^2 = (2.9/2.9)/(1-0.466)2 = 7.4 \times 3.51$$

$$= 26.0 \text{ mm}$$

Since actual bar diameter, $\phi = 10$ mm $< \phi_{s,f} = 13.7$ mm, the SLS is verified.

7.3. Deflection Control

For fibre concrete members with bar reinforcement, Expressions (7.16.a) and (7.16.b) of 7.4.2(2), SS EN 1992-1-1, for the values of limiting span/depth ratios may be used as a conservative simplification. For a more rigorous and accurate calculation, the same principle as given in Section 7.4.3(3) of SS EN 1992-1-1 may be used. The influence of the residual tensile strength of fibre concrete on the moment-curvature relation for a cross-section shall be considered as given in Annex O.3 of the Standard.

For fibre concrete members without bar reinforcement, the deflection of uncracked members may be determined based on elastic analysis using an effective elastic modulus to account for creep. For cracked members, the deflection may be assessed by considering the member as consisting of cracked and uncracked elastic regions, as shown in Fig. 7.1. The length of the cracked region is determined by the characteristic length l_{cs}, defined by Eq. (3.10).

Commentary

Draft Annex L does not give any specific rules on deflections of FRC members.

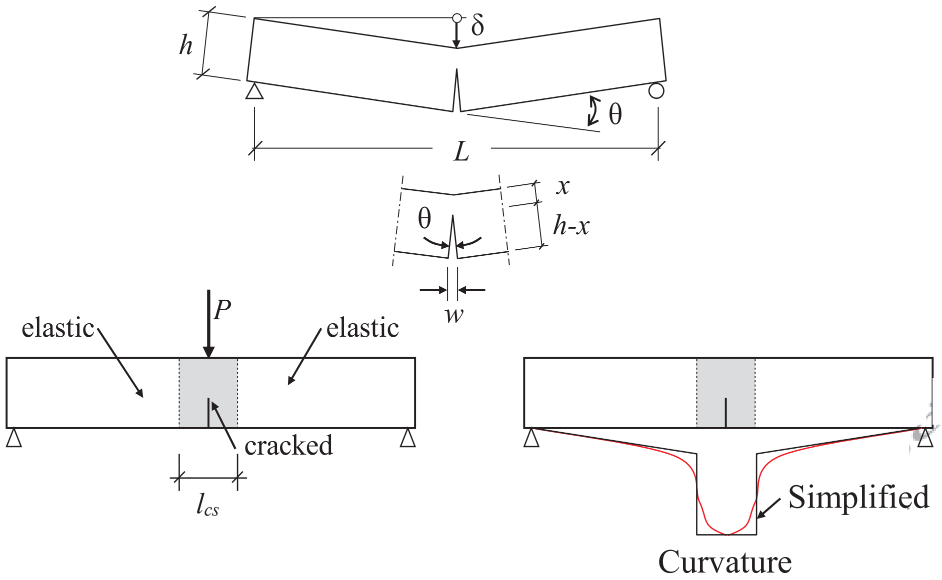

Figure 7.1. Cracked and uncracked regions of fibre concrete members without bar reinforcement

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Example 7.3. Deflection Control

Verify the serviceability limit state of deflection for the simply-supported one-way FRC slab described in Example E7.2 under a quasi-permanent loading of 9.75 kN/m^2. Neglect curvature due to shrinkage of concrete.

Solution 7.3. Uncracked section properties:

$$x_u = (bh^2/2 + \alpha_e A_s d)/(bh + \alpha_e A_s)$$

$$= (1000 \times 150^2/2 + 17.1 \times 628 \times 120)/(1000 \times 150 + 17.1 \times 628)$$

$$= 78.0 \text{ mm}$$

$$I_u = bx_u^3/12 + bh(h/2 - x_u)^2 + \alpha_e A_s(d - x)^2$$

$$= 1000 \times 78.0^3/3 + 1000 \times 150 \times (150/2 - 78.0)^2$$

$$+ 17.1 \times 628 \times (120 - 78.0)^2$$

$$= 301.5 \times 10^6 \text{mm}^4$$

The cracking moment is given by:

$$M_{cr} = 0.9 f_{ctm,fl} I_u/(h - x_u) = 0.9(1.6 - h/1000) f_{ctm} I_u/(h - x_u)$$

$$= 0.9 \times (1.6 - 150/1000) \times 2.9 \times 301.5 \times 10^6/(150 - 78.0) \times 10^{-6}$$

$$= 15.8 \text{ kNm/m}$$

Effective section curvature:

Mid-span moment due to q.p. loading, $M_s = 19.5$ kNm/m.

$$(1/r)_u = M_s/E_{c,eff} I_u = 19.5 \times 10^6/(11.7 \times 10^3 \times 301.5 \times 10^6)$$

$$= 5.53 \times 10^{-6}/\text{mm}$$

$$(1/r)_c = 11.16 \times 10^{-6}/\text{mm (from Example E7.2)}$$

For sustained loading, $\beta = 0.5$. Therefore, distribution parameter,

$$\zeta = 1 - \beta(M_{cr}/M_s)^2 = 1 - 0.5 \times (15.8/19.5)^2 = 0.671$$

Hence,

$$(1/r)_{eff} = \zeta(1/r)_c + (1 - \zeta)(1/r)_u$$

$$= [0.671 \times 11.16 + (1 - 0.671) \times 5.53] \times 10^{-6} = 9.31 \times 10^{-6}/\text{mm}$$

The mid-span deflection under the quasi-permanent loading is:

$$\Delta = 5/48(1/r)_{eff} L^2 = 5/48 \times 9.31 \times 10^{-6} \times 4000^2 = 15.5 \text{ mm}$$

$$< L/250 = 4000/250 = 16 \text{ mm}$$

It may be noted that for the RC slab without fibres but with H10 bars at 100 mm spacing and with the same moment capacity, the deflection under the same quasi-permanent loading is 20.2 mm. Thus, replacing 20 percent of the steel bars by fibres would result in 23 percent reduction in the deflection.

Deflection check using limiting span-effective depth ratio:

$$\rho = 0.00523 < \rho_0 = \sqrt{f_{ck}} \times 10^{-3} = \sqrt{30} \times 10^{-3} = 0.00548$$

For a simply supported slab, $K = 1$. Therefore, SS EN 1992-1-1:2008, Eq. (7.16.a) gives the limit span/depth ratio as:

$$L/d = K[11 + 1.5\sqrt{f_{ck}}(\rho_0/\rho) + 3.2\sqrt{f_{ck}}(\rho_0/\rho - 1)^{3/2}]$$

$$= 1 \times [11 + 1.5\sqrt{30}(0.00548/0.00523)$$
$$+ 3.2 \times \sqrt{30}(0.00548/0.00523 - 1)^{3/2}]$$

$$= 11 + 8.6 + 0.2 = 19.8$$

It is clear that neglecting fibre contribution and using the limiting span/depth ratios given by Expressions (7.16.a) and (7.16.b) would be overly conservative. Hence it is suggested to modify the L/d *ratios using the steel stress under characteristic combination of loading (PD 6687-1:2020) obtained with consideration for fibre contribution, but limiting the modification factor* $(310/\sigma_s)$ *to 1.5.*

In this example, the steel stress under characteristic loads, $g_k + q_k$, equal to 12.75 kN/m^2, considering the fibre contribution, is $\sigma_s = 218$ MPa; hence, $(310/\sigma_s) = 310/218 = 1.42 < 1.5$. Therefore, the modified limit span depth ratio may be taken as:

$$(310/\sigma_s)L/d = 1.42 \times 19.8 = 28.1 < \text{ actual } L/d = 33.3$$

Hence, the SLS of deflection is not satisfied, and a more rigorous and accurate calculation as given above would be required.

Chapter Eight

Detailing of Reinforcement and Pre-stressing Tendons – General

8.1. General

The rules for normal structural concrete members shall be followed.

Commentary

Draft Annex L states that in general:

(1) steel fibres shall not be used to replace reinforcement across a construction joint; and
(2) the residual tensile strength of SFRC shall be disregarded at construction joints.

Draft Annex L recognizes that steel fibres can enhance the anchorage of reinforcing bars and pre-tensioned tendons. Specifically, the transfer and anchorage lengths of pre-tensioned tendons may be reduced. However, general and validated design rules are currently not available.

8.2. Spacing of bars

To prevent agglomeration of fibres between reinforcing bars, the clear distance between individual parallel bars or horizontal layers of parallel bars should be not less than 1.5 times the fibre length. For thin vertical members without reinforcement bars, the clear distance between the formwork panels should be not less than 1.5 times the fibre length, unless the structural design and performance is not compromised by a two-dimensional fibre orientation due to the thinness of the elements.

Commentary

Draft Annex L specifies a bar spacing of at least 2 times the fibre length to be used.

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By Tan Kiang Hwee. ISBN: 978-981-18-5451-4

In collaboration with Building and Construction Authority, Singapore.

Chapter Nine

Detailing of Members and Particular Rules

9.1. General

Minimum areas of reinforcement are required to prevent a brittle failure, wide cracks and to resist forces due to restraints.

Commentary

Draft Annex L's minimum reinforcement rules are as follows.

(1) In members reinforced with steel fibres with or without axial force, minimum reinforcement shall be provided such that:

$$M_{R,min}\left(N_{Ed}\right) \geq k \cdot M_{cr}\left(N_{Ed}\right) \quad \text{(C9.1)}$$

where
$M_{R,min}$ *is the bending strength of section with* $A_{s,min}$ *in the presence of axial force* N_{Ed}*, calculated using the stress distribution in Fig. C6.1 or C6.2, and with the effect of fibres included by the effective residual tensile strength,* $f_{Ftu,ef}$ *(Eq. C3.7);*
M_{cr} *is the cracking moment of the section in the presence of* N_{Ed}*; and*
k *is a factor taken as 1.0 for sections with bonded reinforcement, and 1.15 for sections with unbonded prestressing tendons (internal or external).*

(2) For members subjected to axial tension, $A_{s,min}$*, shall satisfy:*

$$N_{R,min} \geq k \cdot N_{cr} \quad \text{(C9.2)}$$

where $N_{R,min}$ *and* N_{cr} *are the axial load capacity and the cracking load, respectively, of section with* $A_{s,min}$*.*

(3) The minimum shear reinforcement ratio, $\rho_{Fw,min}$*, for members reinforced with steel fibres requiring shear or torsion reinforcement may be taken as:*

$$\rho_{Fw,min} = \rho_{w,min} - f_{Ftu,ef}/f_{yk} \geq 0 \quad \text{(C9.3)}$$

where $\rho_{w,min} = 0.08\sqrt{f_{ck}}$*. The minimum amount of conventional shear reinforcement (links) is not required if* $f_{Ftu,ef} \geq 0.08\sqrt{f_{ck}}$*.*

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9.2. Beams

For fibre concrete beams with or without bar reinforcement, the effect of residual tensile strength on the moment capacity may be taken into account. To ensure sufficient capacity for flexural ductility, the beam shall be provided with a minimum reinforcement area of:

$$A_{\mathrm{s,min}} = \frac{A_{\mathrm{ct}} \cdot \left(k_{\mathrm{c}} \cdot f_{\mathrm{ctm}} - \eta_{\mathrm{f}} \cdot \eta_{\mathrm{det}} \cdot f_{ft,R3}\right)}{f_{\mathrm{yk}}} \tag{9.1}$$

where $A_{\mathrm{ct}} = b_{\mathrm{t}}h/2$; k_{c} is a coefficient which accounts for the stress distribution within the section prior to flexural cracking (refer to Clause 7.3.2(2) of SS EN 1992-1-1); and

$$k_{\mathrm{c}} \cdot f_{\mathrm{ctm}} - \eta_{\mathrm{f}} \cdot \eta_{\mathrm{det}} \cdot f_{\mathrm{ft,R3}} > 0 \tag{9.2}$$

This guide recommends further that the minimum area should also satisfy the Eq. (9.1N) of SS EN 1992-1-1, that is:

$$A_{\mathrm{s,min}} = 0.26 \cdot \frac{f_{\mathrm{ctm}}}{f_{\mathrm{yk}}} \cdot b_{\mathrm{t}}d \geq 0.0013 \cdot b_t d \tag{9.3}$$

The above requirements will preclude the use of fibre concrete members without bar reinforcement unless the fibre concrete shows hardening behavior, that is:

$$k_{\mathrm{c}} \cdot f_{\mathrm{ctm}} - \eta_{\mathrm{f}} \cdot \eta_{\mathrm{det}} \cdot f_{\mathrm{ft,R3}} < 0 \tag{9.4}$$

Commentary

Based on Draft Annex L, the minimum longitudinal reinforcement in beams should not be replaced by steel fibres. Also, the shear and torsion reinforcement in beams may be fully replaced by steel fibres if the minimum reinforcement rules (1) to (3) stated above in Section 9.1. Commentry, are fulfilled.

9.3. Solid Slabs

In general, the flexural reinforcement requirement for beams apply to solid slabs using fibre concrete with or without bar reinforcement.

Commentary

Draft Annex L allows longitudinal reinforcement in slabs to be partially replaced by steel fibres subject to the minimum reinforcement rules stated above in Section 9.1. Commentary. The replacement of minimum longitudinal tensile reinforcement should be limited to $k_{\mathrm{AS}}A_{\mathrm{s,min}}$, where k_{AS} is a nationally determined parameter

(NDP), with a recommended value of 0.5. Also, the minimum secondary tensile reinforcement in one-way slabs may be fully replaced by steel fibres.

The shear reinforcement in slabs may be fully replaced by steel fibres if $f_{\mathrm{Ftu,ef}}/f_{\mathrm{yk}} \geq \rho_{\mathrm{w,min}}$ *and/or* $f_{\mathrm{Ftu,ef}} \geq 0.08\sqrt{f_{\mathrm{ck}}}$.

9.4. Walls

The Standard does not give additional rules for walls. Walls under compressive loading should be considered as Executive Class 3 members.

Commentary

Draft Annex L allows the minimum vertical and horizontal reinforcement be fully replaced by steel fibres subject to fulfilling minimum reinforcement rule (1) stated above in Section 9.1. Commentary, with the post cracking tensile strength, $f_{\mathrm{Ftu,ef}}$, *taken in account.*

9.5. SFRC Structures Not Complying with Minimum Reinforcement Requirements

Additional rules for SFRC structural members not complying with minimum reinforcement requirements are given in *Draft Annex L*. These rules may be followed.

Commentary

Draft Annex L provides the following general rules.

(1) For members constructed with joints to avoid uncontrolled cracking, brittle failure of these members should not lead to collapse of the structure.
(2) Members using SFRC and designed in accordance with this Chapter do not preclude the provision of steel reinforcement required to satisfy serviceability, nor reinforcement in certain parts of the members. Reinforcement provided may be taken into account for local verification of ULS and check for SLS.
(3) Members subject to imposed deformations should however comply with minimum reinforcement rules.

SFRC Footing on Rock

For design of SFRC in footing on rock, the values of f_{FTsd} and f_{FTud} may be taken into account in accordance with Eqs. (C3.3) to (C3.8).

Foundations Directly on Ground

For continuously ground supported rafts and foundation beams, the minimum reinforcement can be replaced by using a ***minimum residual strength and ductility class of 1b****, corresponding to $R_1 1/R_3 1$. This is due to the intrinsic ductility of such members. For soft ground or larger loads the required residual strength and ductility class would be higher. Also, for soft ground, large or restrained slabs, the serviceability requirements will result in combined reinforcement.*

Foundations on Piles

For rafts and slabs on piles, a ***minimum residual strength and ductility class of 2c****, corresponding to $R_1 2/R_3 2$, should be used.*

For large loads, a higher residual strength and ductility class would be required. Also, for large or restrained slabs, the serviceability requirements will result in combined reinforcement.

Segmental Linings

For segmental lining without longitudinal bar reinforcement, a ***minimum residual strength and ductility class of 4c****, corresponding to $R_1 4/R_3 4$, should be used. This replaces the minimum reinforcement rule given by Eq. (C9.3).*

Chapter Ten

Conclusions

10.1. General

The Singapore Standard SS 674:2021 has been published to provide a uniform approach for the design of fibre concrete structures, in line with the principles of and as a supplement to SS EN 1992-1-1:2008. This guidebook has been written to give readers further background and information and assist them in the interpretation of the various provisions in the Standard.

Design recommendations pertaining to the properties of fibre concrete, and verifications of ultimate and serviceability limit states are given, and are related to the provisions of the *Draft Annex L* to prEN 1992-1-1:2018. Examples are given to further aid the reader in the application of the Standard.

10.2. Important Points to Note

The reader should be aware of the following in using the Singapore Standard.

(a) The Standard deals with the design of fibre concrete members. It does not deal with the analysis of fibre concrete structures, for which readers are to refer to SS EN 1992-1-1 and other specialist literature for guidance.
(b) Although the Standard is applicable for both metallic and polymeric FRC in principle, it should be noted that most of the equations have been derived based on steel FRC, as are the provisions of *Draft Annex L*.
(c) Fibre concrete should not be used in Execution Class 3 (Structural System 1) members such as beams and decks without conventional bar reinforcement, unless otherwise justified.

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Appendix A

Design Charts for Fibre Concrete Members with Bar Reinforcement in Flexure

The design charts are developed for under-reinforced rectangular sections with fibres contributing up to 50% of the moment capacity, using the method of strain compatibility and a constant stress distribution in fibre concrete in tension, as shown in Fig. A-1. The moment capacity of a section is defined as M_{Rd}, taken as equal to the design moment, M_{Ed}, and the neutral axis depth at failure is denoted by x.

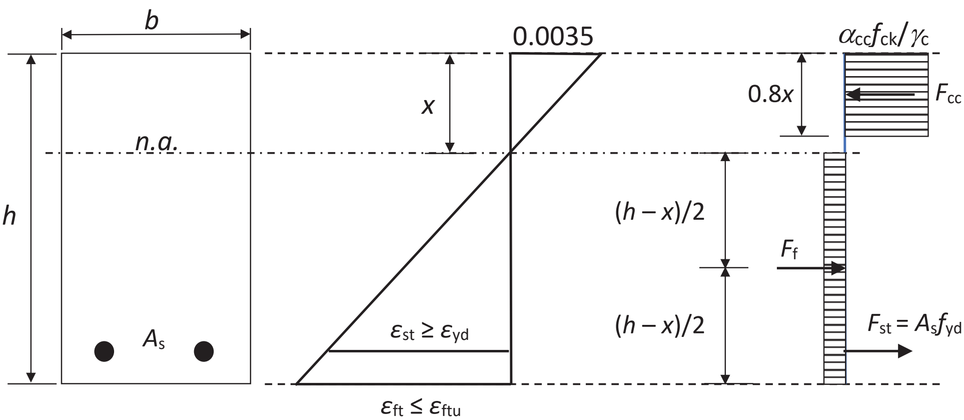

Figure A-1. Strain and stress distributions

Assuming the steel bars (with area, A_s) yield at failure (by ensuring that $x/d < 0.45$), force and moment equilibrium give, respectively:

$$F_f = F_{cc} - F_{st} = b(0.8x)(0.567f_{ck}) - A_s f_{yd} \tag{A-1}$$

$$M_{Ed} = F_f[0.6x + 1/2(h - x)] + A_s f_{yd}(d - 0.4x) \tag{A-2}$$

which simplify to:

$$F_f = 0.453bxf_{ck} - A_s f_{yd} \tag{A-3}$$

$$M_{Ed} = F_{ft}(0.5h + 0.1x) + A_s f_{yd}(d - 0.4x) \tag{A-4}$$

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Substituting the value of F_{ft} given by (A-3) into (A-4) gives:

$$M_{Ed} = (0.453bxf_{ck} - A_s f_{yd})(0.5h + 0.1x) + A_s f_{yd}(d - 0.4x) \tag{A-5}$$

or

$$\begin{aligned} M_{Ed}/bd^2 f_{ck} = {} & [0.453(x/d) - A_s f_{yd}/bdf_{ck}][0.5/(d/h) + 0.1(x/d)] \\ & + A_s f_{yd}/bdf_{ck}[1 - 0.4(x/d)] \end{aligned} \tag{A-6}$$

Also,

$$F_f = b(h - x) f_{ftd,R3} \tag{A-7}$$

From (A-3) and (A-7),

$$b(h - x) f_{ftd,R3} = 0.453bxf_{ck} - A_s f_{yd} \tag{A-8}$$

Hence,

$$A_s f_{yd}/bdf_{ck} = 0.453(x/d) - [1/(d/h) - (x/d)](f_{ftd,R3}/f_{ck}) \tag{A-9}$$

For a set of values of d/h and $f_{ftd,R3}/f_{ck}$, the value of $A_s f_{yd}/bdf_{ck}$ can be obtained from Eq. (A-9) for a specified value of x/d, and subsequently the value of $M_{Ed}/bd^2 f_{ck}$ can be calculated from Eq. (A-6). Using this procedure, Design Charts A-1 to A-4 are developed for the cases of $d/h = 0.80$, 0.85, 0.90 and 0.95, respectively, for values of $f_{ftd,R3}/f_{ck}$ from 0 to 0.25, in steps of 0.05.

In each design chart, lines corresponding to values of K_f/K ranging from 0 to 0.5, in steps of 0.1, are also shown. The parameter K_f/K indicate the fraction of the total moment resistance contributed by the fibres, with $K = M_{Ed}/bd^2 f_{ck}$. For example, $K_f/K = 0.2$ indicates that 20% of the moment capacity is due to the fibres.

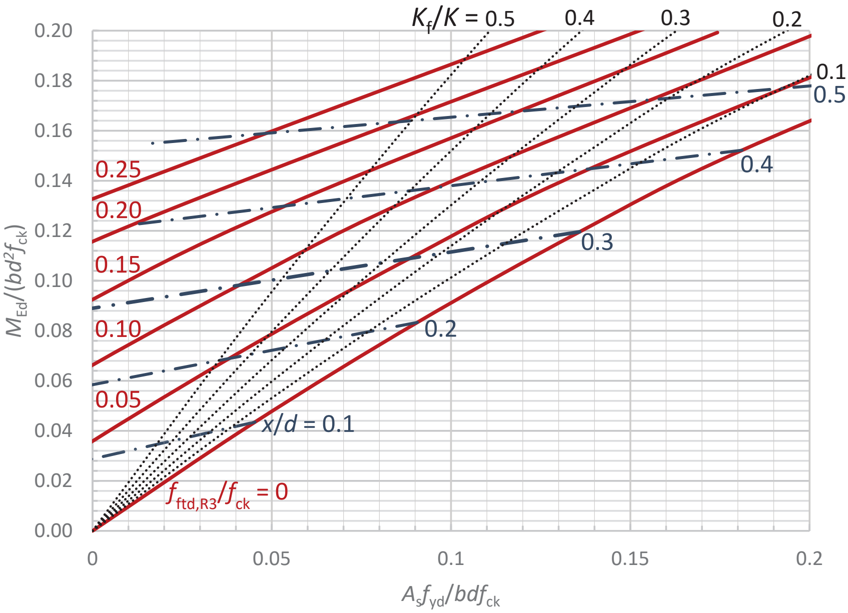

Chart A-1. (d/h = 0.80)

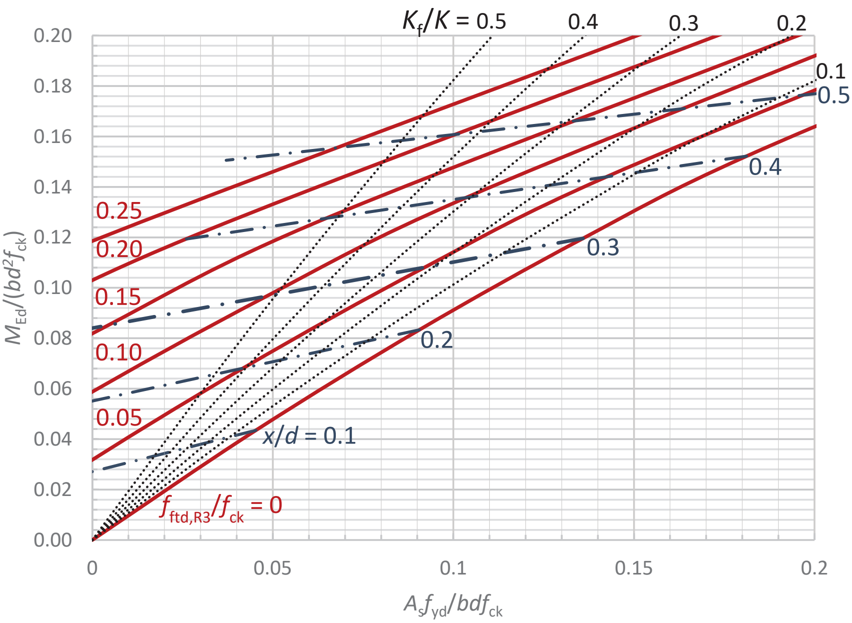

Chart A-2. (d/h = 0.85)

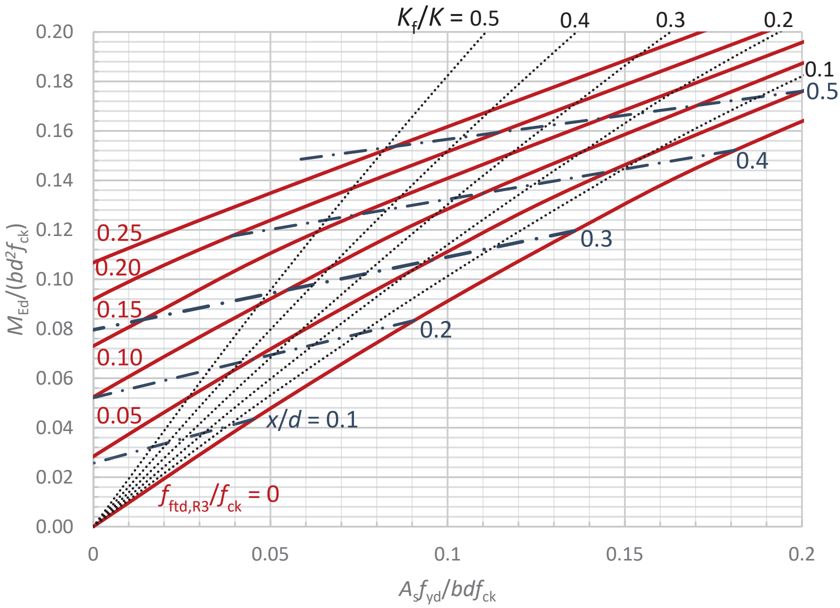

Chart A-3. (d/h = 0.90)

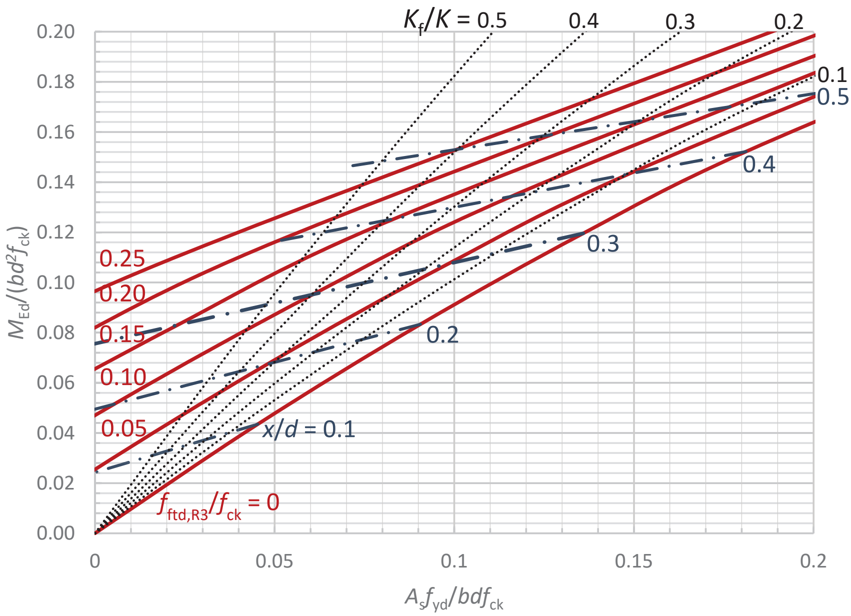

Chart A-4. (d/h = 0.95)

Appendix B

Determination of Steel Stress in Fibre Concrete Members with Bar Reinforcement in Flexure

The strain and stress diagrams for a FRC section under a moment M is shown in Fig. B-1, in which a constant tensile stress distribution, with magnitude $f_{\mathrm{ftd,R1}}$, for fibre concrete is assumed.

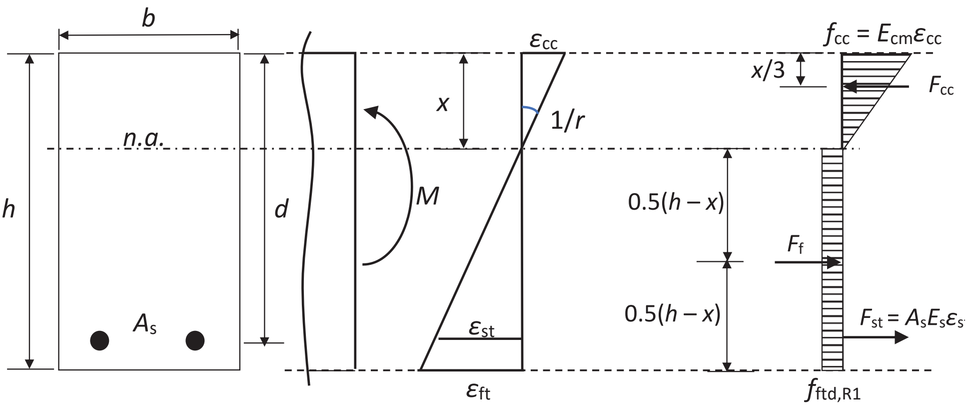

Figure B-1.

The following procedure may be used to obtain the corresponding section curvature, $1/r$, neutral axis depth, x.

Step 1: Assume a value for the section curvature, $1/r$.
Step 2: Adjust the value of x until force equilibrium is achieved, i.e.,

$$\Sigma F = F_{cc} - F_f - F_{st} = 0.$$

Step 3: Check if $M = \Sigma M = F_{cc} \cdot (2x/3) + F_f \cdot [1/2(h - x)] + F_{st} \cdot (d - x)$.
Step 4: If not, adjust the value of $1/r$ and repeat Steps 1 to 3 until $M = \Sigma M$.

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