

DESIGN GUIDE FOR **FIBRE-REINFORCED CONCRETE STRUCTURES**

TO SINGAPORE STANDARD

SS 674:2021

Practical Examples and Design Aids

By

TAN KIANG HWEI

Published by



ASSOCIATION OF
CONSULTING ENGINEERS
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In collaboration with



Building and Construction Authority

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Preface

This book is intended to supplement the Design Guide (2022) for Fibre-Concrete Structures to Singapore Standard SS 674:2021, by providing practical design examples and design aids to assist practitioners.

Chapter 1 of the book summarizes the design requirements for fibre concrete members with updates on the requirement of the 2nd Generation Eurocode 2, EN 1992-1-1:2023. Chapter 2 gives practical design examples which include slab-on-grade, basement and foundation beams as well as slabs, tunnel segmental linings, pile caps, box culverts, and piles. Further examples on crack width calculations and use of macro-synthetic (polymeric) fibres are also included.

Design aids are given in Appendices A to F. These include charts to determine elastic, cracked section properties of FRC sections, design charts for bending, design charts for bending with axial load for both fibre concrete with or without conventional bar reinforcement, and charts for crack width determination. Also, a table and a chart are provided for the determination of shear capacity of fibre concrete, and a conversion table for fibre volume fraction in percentage to fibre content/dosage in kg/m^3 is included.

Last, a supplementary note on the quality assurance/control (QA/QC), highlighting the correct equipment and analysis of test results, in the testing for fibre concrete properties, is included for general information.

October 2025

The Author

Tan Kiang Hwee is Emeritus Professor at the National University of Singapore (NUS). He obtained his doctorate degree from the University of Tokyo, Japan in 1985. He specializes in the area of structural concrete and composites and has taught courses in structural concrete design at both undergraduate and postgraduate levels. He has carried out extensive research on concrete beams with transverse openings, external prestressing, fibre-reinforced concrete, and fibre-reinforced polymer (FRP) systems.

He has published more than 300 refereed technical papers, a book entitled “Beams with Openings: Analysis & Design” (CRC Press LLC, USA, 1999), “Design Guide for Fibre-Reinforced Concrete Structures to Singapore Standard SS 674:2021” (Association of Consulting Engineers Singapore, 2022), and edited the proceedings of the 6th International Symposium on FRP Reinforcement for Concrete Structures (World Scientific, 2003). He was awarded the Frederick Palmer Prize 2021, by the Institution of Civil Engineers (ICE), UK, for the best paper in structural design.

Professor Tan was Honorary Secretary of the Institution of Engineers, Singapore from 2004 to 2006. He was also Vice-Chairman of the International Committee on Concrete Model Code for Asia from 2002 to 2012. He is a Fellow of the Institution of Engineers, Singapore (IES), Japan Concrete Institute (JCI), and Japan Society of Civil Engineers (JSCE). He is also a registered professional engineer in Singapore. He serves as the Chairman for Singapore Standards Council’s Technical Committee on Structures and Sub-Structures and is a member of the Investigation Panel for Professional Engineers Board, Singapore.

Foreword by ACES

In 2022, ACES, in collaboration with the Building and Construction Authority (BCA), launched a pivotal initiative to publish the *Design Guide on Fibre Reinforced Concrete (FRC) Structures*, authored by Emeritus Professor Tan Kiang Hwee. This guide was developed with the vision of promoting the strategic use of fibre reinforced concrete to enhance productivity across suitable applications. Over recent years, the use of FRC, particularly in compression dominant flexural elements, has gained widespread acceptance and is almost becoming an industry norm. This progress is both encouraging and indicative of the growing recognition of the potential of FRC which evidently encouraged further research and expansion of usage into other structural components.

Building on this momentum, ACES and BCA are proud to advance this initiative with the release of a second design guide. This latest edition focuses predominantly on practical applications of FRC in various structural elements supported by design examples derived from actual case studies and meticulously curated design aids by Emeritus Professor Tan in forms charts to support daily practices. Additionally, a comprehensive chapter covering test regimes, quality control, and quality assurance has been incorporated to provide a holistic framework for implementation. ACES hopes this second guide will serve as a catalyst for further innovation and adoption of FRC, empowering consultants, designers, contractors, and all stakeholders in the built environment to harness its full potential. The continued use of FRC can significantly enhance productivity, improve resource efficiency, and contribute to a more sustainable and resilient construction industry, especially in the face of manpower and material constraints.

We would like to express our deepest appreciation to Emeritus Professor Tan for his authorship and invaluable contributions, our generous sponsors, dedicated workgroup members, BCA officers, and industry partners. Their collective support and commitment have been instrumental in making this publication possible. We are confident this effort will serve as a valuable resource that benefits of the entire industry.

Er. Chuck Kho
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Association of Consulting Engineers Singapore 2025

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Association of Consulting Engineers Singapore 2025

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https://www.cnsteelfiber.com/	
ZUBLIN Precast Industries Sdn Bhd	<i>FRC Precaster</i>
https://www.zublin.com.sg/databases/internet/_public/content30.nsf/web30?Openagent&id=33BDD42431D16C14C12581B50043F558	

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NOTE *All symbols used in this book can be found in SS EN 1992-1-1:2008 and SS 674:2021. Otherwise, they are defined where they first appear in this book.*

Chapter 1

General

1.1 Fibre-Reinforced Concrete

Fibre reinforced concrete (FRC), or fibre concrete, refers to concrete that incorporates short, discrete fibres to improve the performance of concrete, particularly in its post-cracking behavior. In Singapore, the design of fibre-reinforced concrete structures is based on SS 674:2021, which 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. A design guide (Tan 2022) was published by the Association of Consulting Engineers Singapore to provide guidance on the use of SS 674:2021.

NOTE In general applications, fibre dosage varies from 0.25 to 1.0% by volume of concrete. Appendix A shows the corresponding dosage in kg/m³ for common steel and polymer fibres. Also, for an effective application, fibre length should generally be taken as about 3 times the maximum aggregate size. The Singapore Standard SS 674:2021 does not deal with micro-fibres such as monofilament polypropylene fibres which are useful in limiting spalling effects in fibre concrete.

This book is intended to further supplement the design guide with practical design examples and design aids. The requirements for material strengths, and design for ultimate limit states (ULS) and serviceability limit states (SLS) are summarized with updates on the provisions of the 2nd Generation Eurocode 2 (EN 1992-1-1:2003), where applicable. In addition, the application of strut-and-tie models for FRC members with discontinuous regions is described.

1.2 Material Strengths

As described in the Design Guide 2022, fibre concrete is classified by strength classes, R₁, R₃ and R₄, with specified values of residual flexural (tensile) strengths, $f_{R,1}$, $f_{R,3}$ and $f_{R,4}$, respectively. For serviceability limit state (SLS) verifications, Class R₁ must be specified. For ultimate limit state (ULS) verifications, either Class R₃ alone or both Class R₁ and Class R₃ must be specified.

NOTE In the Second Generation Eurocode 2 (EN 1992-1-1:2023, Annex L), steel fibre reinforced concrete shall be classified into both the strength classes SC (1.0 to 8.0) and ductility classes (a to e).

1.2.1 Design Residual Tensile Strength

For ULS, the design residual tensile strengths are defined as:

$$f_{ftd,R1} = \eta_f \cdot \eta_{det} \cdot \frac{0.45 \cdot f_{R,1}}{\gamma_f} \quad (1)$$

$$f_{ftd,R3} = \eta_f \cdot \eta_{det} \cdot \frac{0.37 \cdot f_{R,3}}{\gamma_f} \quad (2)$$

For SLS, it is defined as:

$$f_{ftd,R1} = \eta_f \cdot 0.45 \cdot f_{R,1} \quad (3)$$

where $f_{R,1}$ and $f_{R,3}$ are the respective characteristic residual flexural (tensile) strengths; η_f is a factor accounting for fibre orientation; and η_{det} is a magnification factor accounting for the degree of structural redundancy in the system. Also, γ_f is the partial factor taken as equal to 1.5 (SS 674:2021).

For bending moments, shear and punching shear forces, torsion, crack width control and deflections in slabs and beams made of concrete with consistency classes S2-S4 in accordance

with SS EN206:2014(2024)+A1:2024, $\eta_f = 1.0$ may be used. Otherwise, $\eta_f = 0.5$ should be adopted unless verified by testing. Also, 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*.

NOTE *Refer to Design Guide 2022, Section 3.2, Table C3.2 for further recommendations on the values of η_{det} .

1.2.2 Stress-Strain Relations

The design stress-strain relation for fibre reinforced concrete in compression¹ can be taken to be the same as that for the parent concrete without fibres. In tension, the stress-strain relation² may assume either a constant-drop relationship [Fig. 1(a)] or a linear descending-drop relationship post the peak stress [Fig. 1(b)].

NOTE 1 For normal strength concrete, the design stress-strain relation in compression is a parabolic curve up to the design compression strength with a strain of 0.002, followed by a constant strength up to an ultimate strain of 0.0035.

NOTE 2 As a conservative simplification, the value of ϵ_{ftu} may be taken as $2.5/(0.8h)$ for members with conventional bar reinforcement, and $2.5/h$ for members without conventional bar reinforcement, where h is the member thickness in mm.

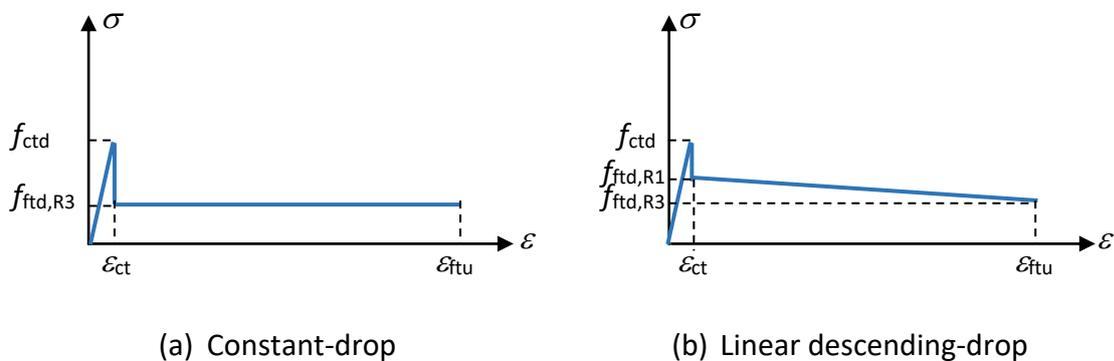


Fig. 1 Design stress-strain relations for FRC in tension

1.3 Design for Bending with or without Axial Force

The strain and stress distributions for fibre-reinforced sections at ultimate flexural limit state are shown in Fig. 2. For simplicity, a constant stress equal to $f_{ftd,R3}$ (or $f_{ftd,R1}$ if $f_{ftd,R3} > f_{ftd,R1}$) over the height of the tension zone may be assumed (Fig. 2c). Design charts for flexural design of under-reinforced sections with fibres contributing up to 50% of the moment capacity are shown in Appendix C. Design charts for sections subjected to bending with axial force are given in Appendix D.

NOTE 1 EN 1992-1-1:2023, Annex L permits the use of constant stress distribution for classes a to c only. For classes d and e, this should only be used to determine the ULS moment capacity at the design tensile strain limit of 0.02.

NOTE 2 In general, the contribution of fibres to the moment capacity should be limited to 30%.

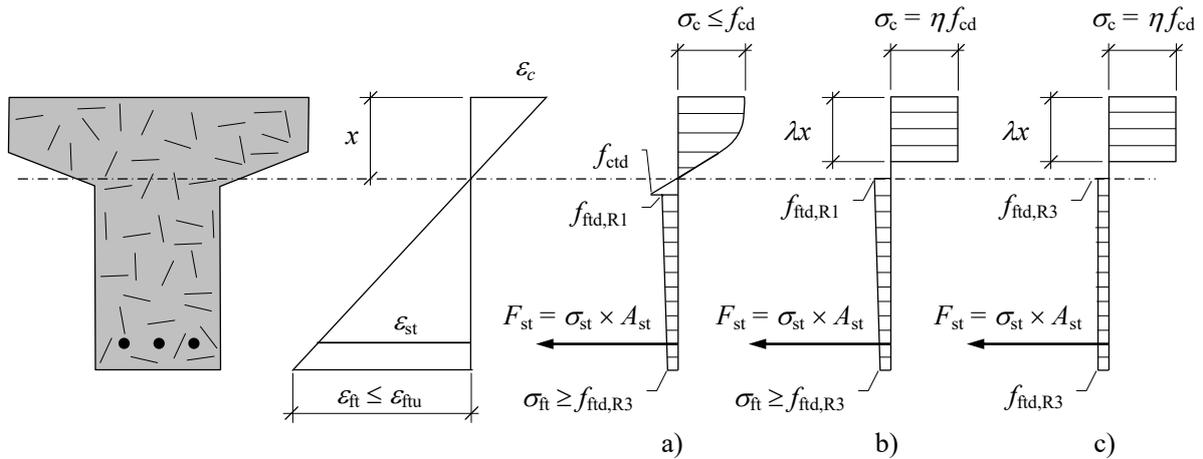


Fig. 2 Strain and stress distributions at flexural ultimate limit state

1.4 Design for Shear and Punching

The design shear resistance (in MPa) of fibre reinforced concrete without shear reinforcement is given by:

$$v_{Rd,cf} = \{(0.18/\gamma_c) \cdot k \cdot [100\rho(1+7.5f_{ft,R3}/f_{ctk}) \cdot f_{ck}]^{1/3} + 0.15\sigma_{cp}\} \\ \geq v_{Rd,cf(\min)} = \{0.035k^{3/2}f_{ck}^{1/2} + 0.15\sigma_{cp}\} \quad (4)$$

where

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

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

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

The values of $v_{Rd,cf}$ are tabulated in Appendix E, Table E-1. The values can also be determined from Chart E-1.

NOTE 1 For slabs without conventional steel bars (e.g., slabs-on-grade), the shear capacity may be conservatively evaluated from Eq. (4) with $\rho_l = 0$ and $d = 0.75h$ where h is the slab thickness. In such cases, the shear capacity is given by the lower bound value $v_{Rd,cf(min)}$.

NOTE 2 The punching shear capacity for ground-supported slabs can be determined as: $v_{Rd,cf} = v_{Rd,f} = (k/2) \cdot C \cdot f_{R3}/\gamma_f$ (SS 674, Eq. 6.4) with $C = 0.45$; $\gamma_f = 1.5$; and $d = 0.75h$.

For members requiring shear reinforcement, that is, where $v_{Ed} > v_{Rd,cf}$, the shear resistance may be taken as (Design Guide 2022):

$$v_{Rd} = 0.75v_{Rd,s} + v_{Rd,cf} \leq v_{Rd,max} \quad (5)$$

where $v_{Rd,s} = (A_{sw}/s) z f_{wd} \cot \vartheta / b_wd$ and $v_{Rd,max}$ is the maximum shear resistance.

1.5 Serviceability Limit States

The calculations of concrete and steel stresses, crack widths and deflections require either or both the elastic uncracked and elastic cracked section properties. The fibre concrete can be considered as plain concrete under elastic uncracked behaviour. For cracked section properties, Appendix B gives design charts for the determination of neutral axis depth, x , and section

curvature, $1/r$, for a given moment M_{Ed} , and steel ratio, ρ . The corresponding concrete and steel stresses at a level y from the neutral axis are, respectively:

$$\sigma_c = E_c \cdot \frac{1}{r} \cdot y \quad (6a)$$

$$\sigma_s = E_s \cdot \frac{1}{r} \cdot y \quad (6a)$$

where E_c and E_s are the modulus of elasticity of concrete and steel, respectively.

1.5.1 Calculation of Crack Widths

For fibre concrete members **with bar reinforcement**, the crack width may be calculated from:

$$w_k = s_{r,max} \cdot (\varepsilon_{sm} - \varepsilon_{cm}) \quad (7)$$

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

The strain difference ($\varepsilon_{sm} - \varepsilon_{cm}$) can be determined either from:

$$(\varepsilon_{sm} - \varepsilon_{cm}) = \frac{\sigma_s - (k_t + (1 - k_t) \cdot k_f) \cdot \frac{f_{ct,eff}}{\rho_{p,eff}} \cdot (1 + \alpha_e \cdot \rho_{p,eff})}{E_s} \geq 0.6 \cdot \frac{\sigma_s}{E_s} \quad (8)$$

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

$$(\varepsilon_{sm} - \varepsilon_{cm}) = \frac{(1 - k_f) \cdot \left(\sigma_{s,fict} - k_t \cdot \frac{f_{ct,eff}}{\rho_{p,eff}} \cdot (1 + \alpha_e \cdot \rho_{p,eff}) \right)}{E_s} \geq 0.6 \cdot (1 - k_f) \cdot \frac{\sigma_{s,fict}}{E_s} \quad (9)$$

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

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:

$$s_{r,\text{max}} = k_3 \cdot c + k_1 \cdot k_2 \cdot k_4 \cdot (1 - k_f) \cdot \frac{\phi}{\rho_{p,\text{eff}}} \quad (10)$$

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

$$s_{r,\text{max}} = 1.3 \cdot (h - x) \cdot (1 - k_f) \quad (11)$$

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

$$w_{\text{max}} = \varepsilon_{\text{ft}} \cdot 2(h - x) \quad (12)$$

where ε_{ft} is the maximum tensile strain for the relevant load combination.

Appendix F, Chart F-1 facilitates the determination of maximum crack width using the rigorous approach, that is, Eqs. (7), (8) and (10).

1.6 Design with Strut-and-Tie Models

Strut-and-tie models may be used for the design of regions or members with force or geometrical discontinuity. These included knee joints, dapped ends, corbels, deep beams and pile caps. The flow of forces through such regions or members is facilitated by strut and tie

members, which are connected at nodes or nodal zones. The struts, ties and nodal zones are checked for strength capacities, and the method constitutes a lower-bound and hence a conservative design approach.

The Singapore Standard SS 674:2021 does not provide additional recommendations further to SS EN 1992-1-1:2008. However, the Second-Generation Eurocode 2 (EN 1992-1-1:2023), Annex L (Informative), gives some recommendations as follows:

- (a) Fibres may be used in ties in place of transverse reinforcement.
- (b) Fibres may substitute up to 30% of the total required main longitudinal reinforcement in ties.
- (c) In walls and deep beams, vertical and horizontal minimum reinforcement may be fully replaced by steel fibres.

References

- EN 1992-1-1:2023 (2023). *Eurocode 2: Design of concrete structures – Part 1-1: General rules, rules for buildings, bridges and civil engineering structures.*
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Chapter 2

Design Examples

This chapter gives practical design examples which include slab-on-grade, basement and foundation beams as well as slabs, tunnel segmental linings, box culverts, pile caps and piles. They have been modified from actual field applications or from existing RC design for such members to illustrate the use of the Singapore Standard SS 674:2021 and the design aids given in this supplementary guide.

Table 2-1 lists the various topics treated under each of the ten practical examples presented herein. While each example is meant to cover the full design requirements in FRC, some aspects have been omitted in some examples due to repetitiveness.

Table 2-1 Topics Dealt with in the Examples

Examples Topics	1	2	3	4-1	4-2	5-1	5-2	6	7	8
	Slab-on-Grade	Foundation Slabs	Foundation Beams	Thick Base-ment Slabs	Thick Base-ment Slabs	Tunnel Segments	Tunnel Segments	Pile Caps	Box Culvert	Circular Bored Piles
FRC Members <u>without</u> Reinforcement Bars										
Bending	✓					✓	✓			
Axial Load						✓	✓			
Shear	✓					✓				
Punching	✓									
Cracking							✓			
FRC Members <u>with</u> Reinforcement Bars										
Bending		✓	✓		✓			✓	✓	✓
Axial Load			✓						✓	✓
Shear		✓	✓					✓	✓	✓
Punching		✓								
Cracking		✓	✓	✓	✓				✓	✓
Deflection			✓							
Special Topics										
Strut-and-Tie Models								✓		
MSFRC ^a							✓			

Note: ^a Macro-Synthetic (polymeric) Fibre Reinforced Concrete

2.1 Slab-on-Grade

The design of fibre-reinforced concrete slabs-on-grade involves several steps:

1. Define the use of the slab (e.g., industrial floor, warehouse, parking lot) and expected loads (e.g., equipment, shelving, vehicles, foot traffic).
2. Choose the type and amount of fibre reinforcement (synthetic or steel).
3. Select the concrete mix design and strength (20-25 MPa).
4. Calculate the slab thickness and reinforcement requirements. Approximate guidelines for slab thickness are:
 - Light loads (e.g., foot traffic, light vehicles): 100-125 mm
 - Medium loads (e.g., cars, light trucks): 125-150 mm
 - Heavy loads (e.g., forklifts, heavy trucks): 150-200 mm
 - Very heavy loads (e.g., containers): more than 200 mmTypical fibre dosages (0.5-1.5%) by volume:
 - Steel fibres: 20-40 kg/m³
 - Synthetic fibres: 0.9-2.7 kg/m³
5. Consider shrinkage and thermal cracking.
6. Design for load distribution and stress concentrations.
7. Check for punching shear and edge stresses.
8. Specify joints and detailing (joint spacing: typically, 24-36 times the slab thickness; or 3-4.5 m).

References

- ACI 360R-10: Guide to Design of Slabs-on-Ground. American Concrete Institute, USA.
- ACI 544.4R-18: Guide for Design with Fiber-Reinforced Concrete. American Concrete Institute, USA.
- TR 34: Concrete Industrial Ground Floors – A guide to design and construction. 4th Edition. The Concrete Society, UK.
- TR 63: Guidance for the Design of Steel-Fibre-Reinforced Concrete. The Concrete Society, UK.

Example 1 – Port Pavement

This example deals with the design of a container port pavement slab supported on ground. The pavement is to carry point loads due to stacks of up to six containers, and a reach stacker. The point loads of the containers and the reach stacker can be positioned both in the slab centre as well as at the dowelled joints. The design is based on yield line theory.

Loads

Containers: Six containers, each with gross weight of 30480 kg (i.e., approximately 304.8 kN), stacked on top of each other. The total load per footplate is thus $F_{fp} = 6 \times 304.8 / 4 = 457.2$ kN.

Referring to PIANC Report 165*, a reduction factor depending on the stacking height, may be applied to account for the fact that the probability of all containers being completely full is very unlikely. For 6-container high, the proposed reduction factor is 40%, which leads to the following load per footplate:

$$F_{fp} = 0.6 \times 457.2 \text{ kN} = 274.3 \text{ kN}$$

NOTE: The World Association for Waterborne Transport Infrastructure, “Design and Maintenance of Container Terminal Pavements”, PIANC Report No. 165, 2015, Maritime Navigation Commission, 158 pp.

Load Parameters

LOAD F_1 ⓘ reset default value ↻

kN 1 3000

DISTANCE A ⓘ reset default value ↻

mm 178 7620

DISTANCE B ⓘ reset default value ↻

mm 178 7620

SIZE X ⓘ reset default value ↻

mm 25 3050

SIZE Y ⓘ reset default value ↻

mm 25 3050

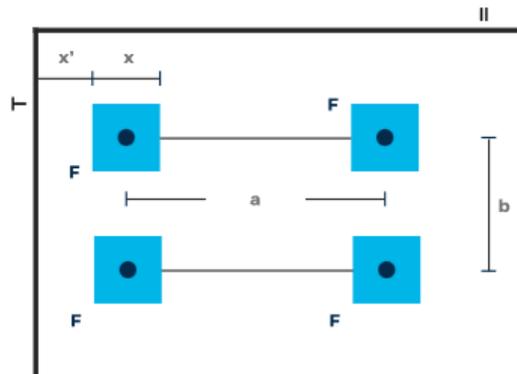


Fig. E1-1 Input load parameters – Containers

A reach stacker with an axle load of 101,900 kg = 1019 kN, divided over 4 wheels as follows:

Load Parameters

WHEEL LOAD F1 ⓘ reset default value ↻

kN 1 900

DISTANCE A ⓘ reset default value ↻

mm 50 3000

DISTANCE B ⓘ reset default value ↻

mm 50 4500

WHEEL PRESSURE P ⓘ reset default value ↻

N/mm² 0.1 45

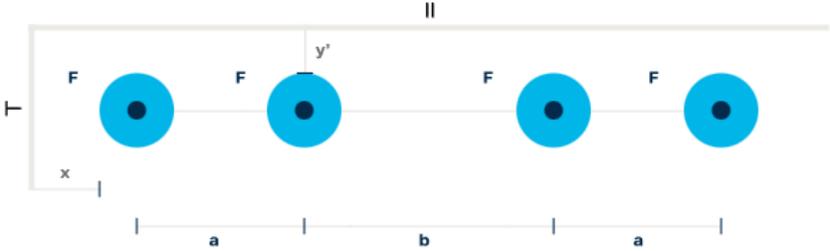


Fig. E1-2 Input load parameters – Reach stacker

Modulus of subgrade reaction

The following design example was based on the Modulus of Subgrade Reaction or k Westergaard, assuming a value $k = 0.10 \text{ N/mm}^3$.

Proposed solution

- *Slab thickness:* 390 mm
- *Concrete strength class:* C32/40
- *Joint concept:* The pavement is designed with joints at every 8 m in orthogonal directions. Given the large point loads and wheel loads, the joints are assumed to be provided with dowels having a sufficient load transfer between adjacent panels. A joint efficiency factor = 70% is assumed.

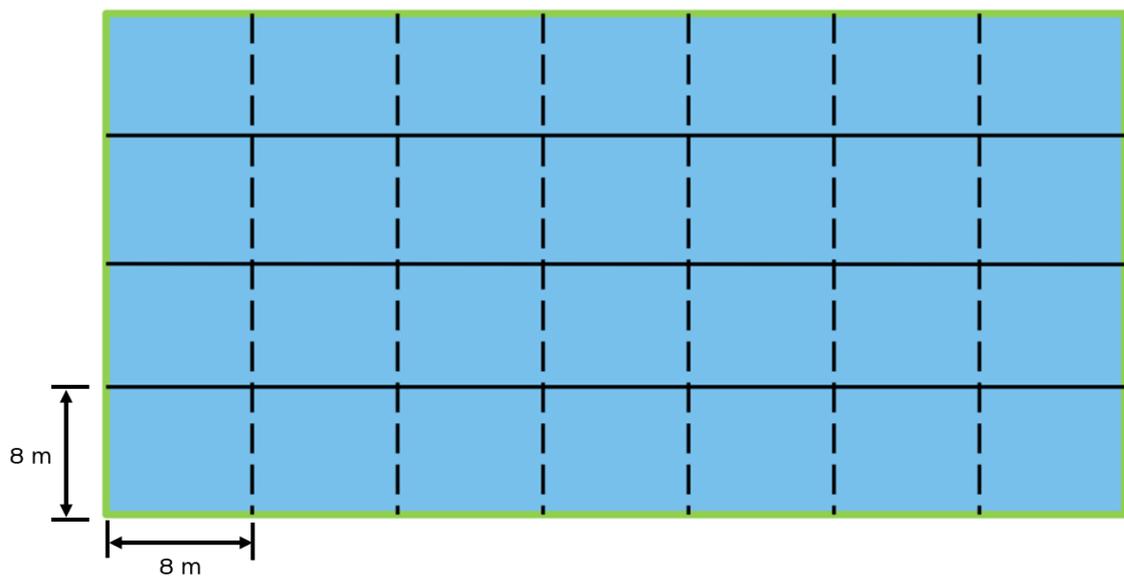


Fig. E1-3 Joint arrangement

Load cases

In this design example, it is assumed that the point loads of the containers and the reach stacker can be positioned both in the slab centre as well as at the dowelled joints. The design is based on yield line theory.

Design for bending

Following the yield line theory, occurring system bending moments $m + m'$ can be determined both at the internal slab and at the joints as follows:

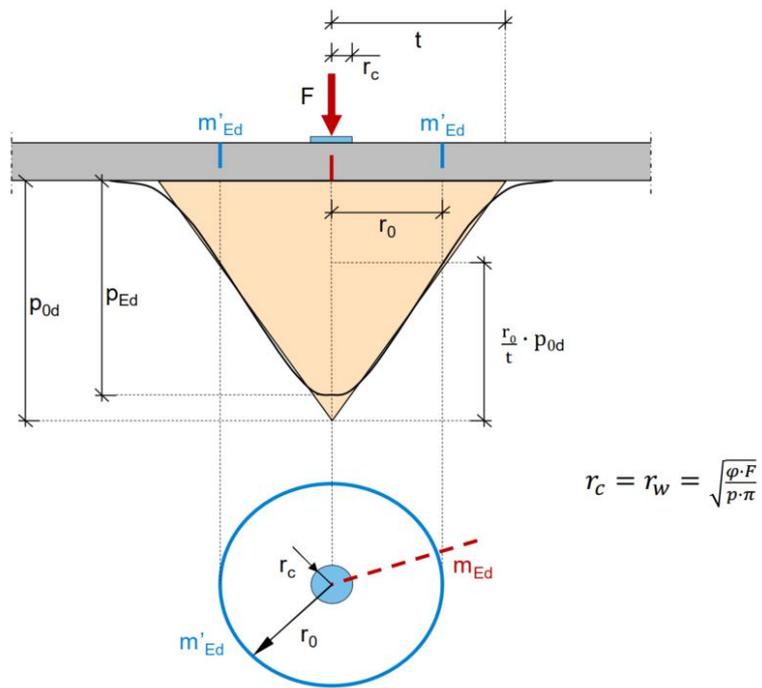


Fig. E1-4 Yield line pattern - Point or wheel loads at the interior of the slab

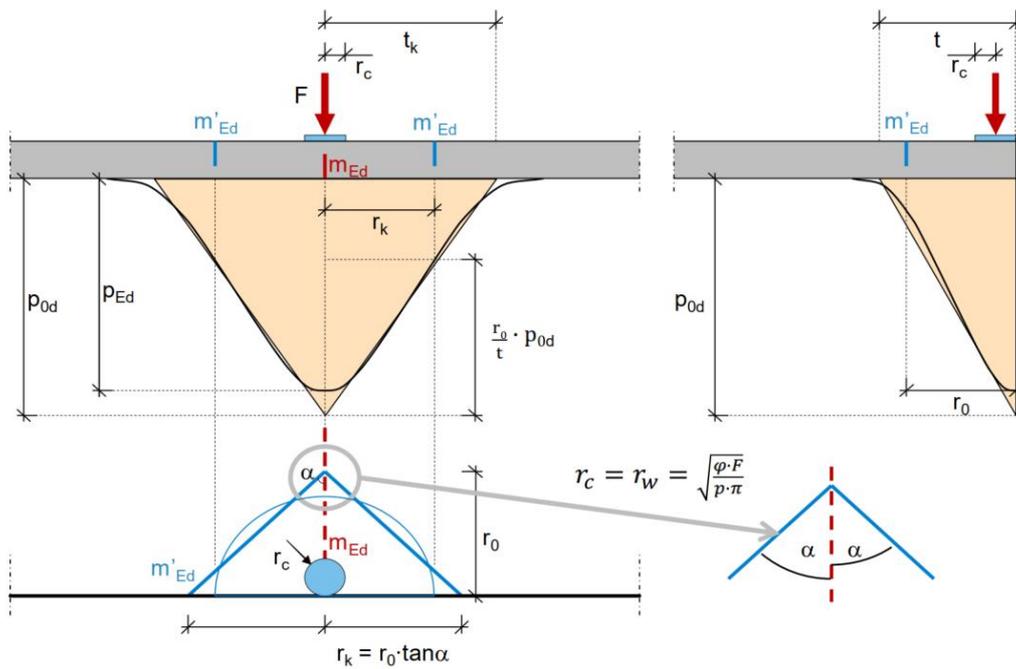


Fig. E1-5: Yield line pattern - Point or wheel loads at free edge or joint of the slab

The yield line analysis for the slab is not dealt with here. Readers may refer to TR63 for information. The design system bending moments in this case are finally obtained as:

- At slab centre: $(m+m')_{Ed} = 124.7 \text{ kNm/m}$
- At joint: $(m+m')_{Ed} = 174.3 \text{ kNm/m}$

Moment capacity of the system

The moment capacity for the system can be determined as:

$$m_{sys} = m'_{Rd} + m_{Rd}$$

The characteristic residual flexural tensile strengths are:

- $f_{R1} = 3.3 \text{ MPa}$
- $f_{R3} = 3.7 \text{ MPa}$

from which the characteristic residual tensile strengths are obtained as:

- $f_{ft,R1} = 0.45 \cdot f_{R1} = 1.49 \text{ MPa}$ (SS 674, Eq. 3.1)

- $f_{ft,R3} = 0.37 \cdot f_{R3} = 1.37 \text{ MPa}$ (SS 674, Eq. 3.2)

The design residual tensile strengths in ULS are:

$$f_{ftd,R1} = \eta_f \cdot \eta_{det} \cdot \frac{f_{ft,R1}}{\gamma_f} = 1 \cdot 1.4 \cdot \frac{1.49}{1.5} = 1.39 \text{ MPa}$$
 (SS 674, Eq. 3.3)

$$f_{ftd,R3} = \eta_f \cdot \eta_{det} \cdot \frac{f_{ft,R3}}{\gamma_f} = 1 \cdot 1.4 \cdot \frac{1.37}{1.5} = 1.28 \text{ MPa}$$
 (SS 674, Eq. 3.4)

In this example, the simplified stress distribution for ULS in SS 674, Fig. 6.1(b) [Fig. 2(b) in this Guide] is used. The maximum design tensile strength at the extreme tensile fibre is:

$$\sigma_{ft} = f_{ftd,R1} - \frac{\varepsilon_{ft}}{\varepsilon_{ftu}} \cdot (f_{ftd,R1} - f_{ftd,R3})$$
 (SS 674, Eq. 6.1)

with

$$\varepsilon_{ct} = \frac{f_{ctd}}{E_c} = \frac{1.4}{33500} = 4.2 \cdot 10^{-5}$$
 (SS 674, Fig. 3.3)

$$\varepsilon_{ftu} = \varepsilon_{ct} + \frac{w_u}{l_{cs}} = 4.2 \cdot 10^{-5} + \frac{2.5}{390} = 6.45 \cdot 10^{-3} \quad (\text{SS 674, Fig. 3.3})$$

Considering the idealized parabolic-rectangular stress-strain relation for concrete in compression, with $\varepsilon_{c2} = 0.002$ and $\varepsilon_{cu2} = 0.0035$, and taking the extreme tensile fibre strain, $\varepsilon_{ft} = \varepsilon_{ftu} = 0.00645$, the following solution is obtained:

$$x = 55.7 \text{ mm}; \varepsilon_c = 0.00108 < \varepsilon_{c2} = 0.002; \varepsilon_{ft} = 0.00645 \text{ (as assumed)}$$

Force in concrete compression zone, $C = 446.2 \text{ kN/m}$; acting at $x_c = 36.1 \text{ mm}$ from neutral axis.

Forces in tension, $F_{f1} = 427.9 \text{ kN/m}$; $F_{f2} = 18.4 \text{ kN/m}$; $F_f = F_{f1} + F_{f2} = 446.3 \text{ kN/m} = C$

Moment capacity of section,

$$m_{Rd} = m'_{Rd} = [446.2 \times 36.1 + 427.9 \times (390 - 55.7)/2 + 18.4 \times (390 - 55.7)/3] \times 10^{-3} = 89.7 \text{ kNm/m}$$

Alternatively, using design chart C-1 (Appendix C) with $d/h = 0.8$, $A_s = 0$, and $f_{ftd,R3}/f_{ck} = 1.28/32 = 0.04$, one obtains:

$$m_{Rd} = 0.029bd^2f_{ck} = 0.029b(0.8h)^2 f_{ck} = 0.029 \times (0.8 \times 390)^2 \times 32 \times 10^{-6} = 90.3 \text{ kNm/m}$$

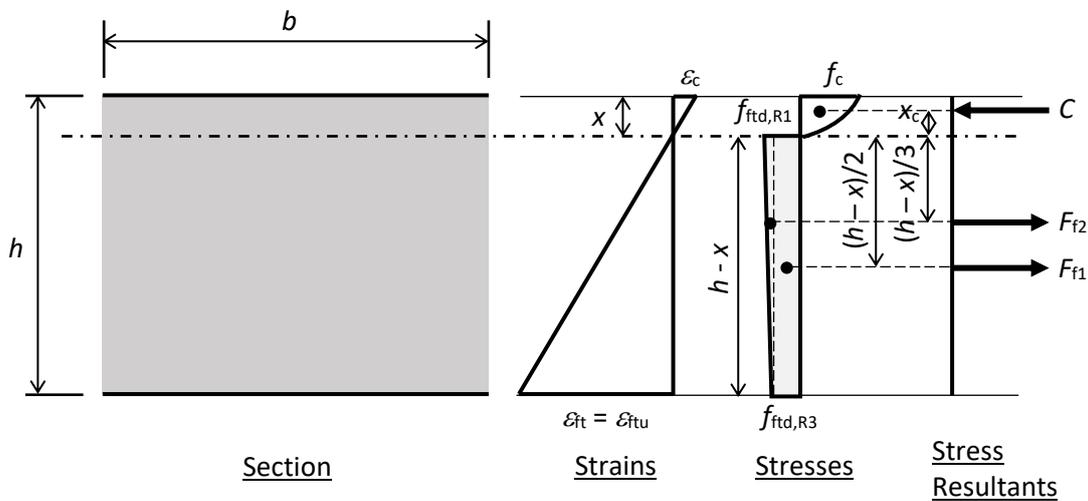


Fig. E1-6 Analysis for Ultimate Bending Moment

The system bending moment capacity is:

$$M_{sys} = m_{Rd} + m'_{Rd} = 180 \text{ kNm/m}$$

This is larger than the design bending moments at the slab centre and doweled joint, 124.7 kNm/m and 174.3 kNm/m, respectively.

Design for Shear

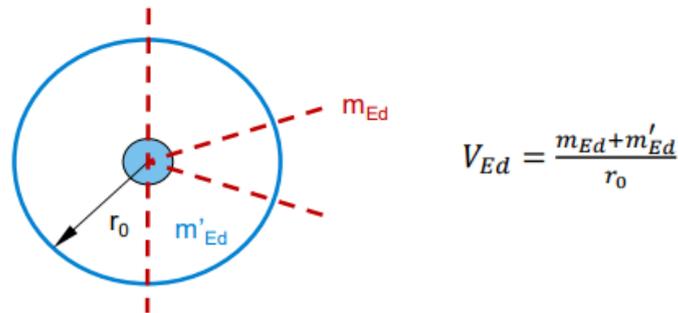


Fig. E1-7 Yield line theory - Acting shear forces for point or wheel loads

In this example, the governing shear forces occur when four container point loads are placed at the doweled joint. The design shear force in this case is $V_{Ed} = 109.7$ kN/m.

The shear capacity of steel fibre concrete section can be taken as:

$$V_{Rd,cf} = v_{Rd,cf(min)} \cdot bd = 0.75v_{min} \cdot bh \quad (\text{assuming } d = 0.75h)$$

where

$$v_{Rd,cf(min)} = 0.035k^{3/2}f_{ck}^{1/2} = 0.489 \text{ MPa}$$

in which $k = 1 + \sqrt{(200/d)} = 1 + \sqrt{(200/(0.75 \times 390))} = 1.83 < 2$. Hence,

$$V_{Rd,cf} = 0.489 \times 0.75 \times 390 = 143.0 \text{ kN/m} > V_{Ed} = 109.7 \text{ kN/m} \quad \dots \text{ ok}$$

Design for punching shear

The governing punching load case is when four container point loads are placed at the joint (refer Figs. E1-8 to E1-10). Taking the ground pressure into account, the design punching shear is $V_{Ed,red} = 1222$ kN.

The punching shear capacity for ground-supported slabs can be determined as:

$$v_{Rd,cf} = v_{Rd,f} = (k/2) \cdot C \cdot f_{R3}/\gamma_f = 1.01 \text{ MPa} \quad (\text{SS 674, Eq. 6.4})$$

where $C = 0.45$; $k = 1.83$; $f_{R3} = 3.7$ MPa; and $\gamma_f = 1.5$.

For the control perimeter 1 at $a_{crit} = 2d = 2 \times 0.75 h = 1.5 \times 390 = 585$ mm from the loading area, $u_1 = 3840$ mm. Therefore,

$$V_{Rd,f} = v_{Rd,f} \cdot u_1 d = 1.01 \times 3840 \times 0.75 \times 390 \times 10^{-3} = 1139 \text{ kN} < V_{Ed,red} = 1222 \text{ kN}$$

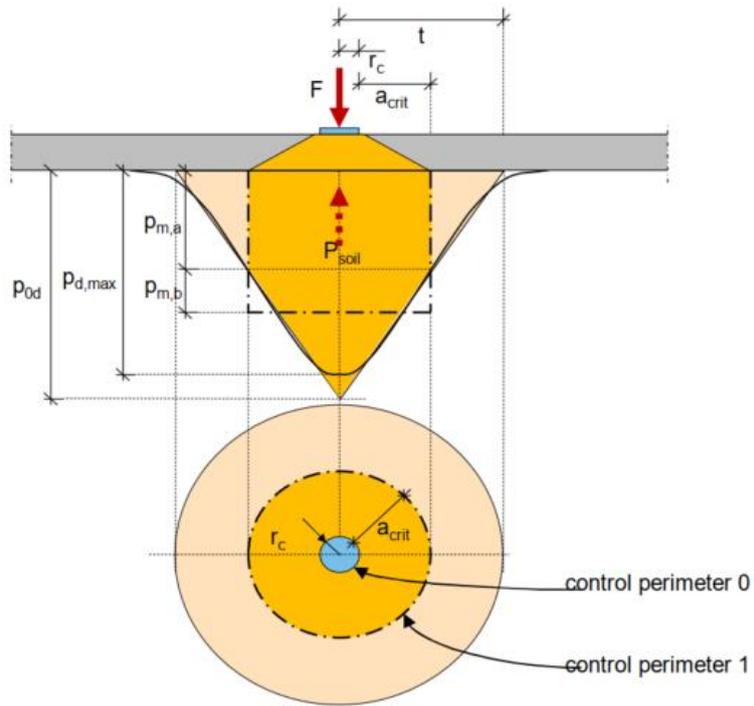


Fig. E1-8 Point load punching at the slab centre

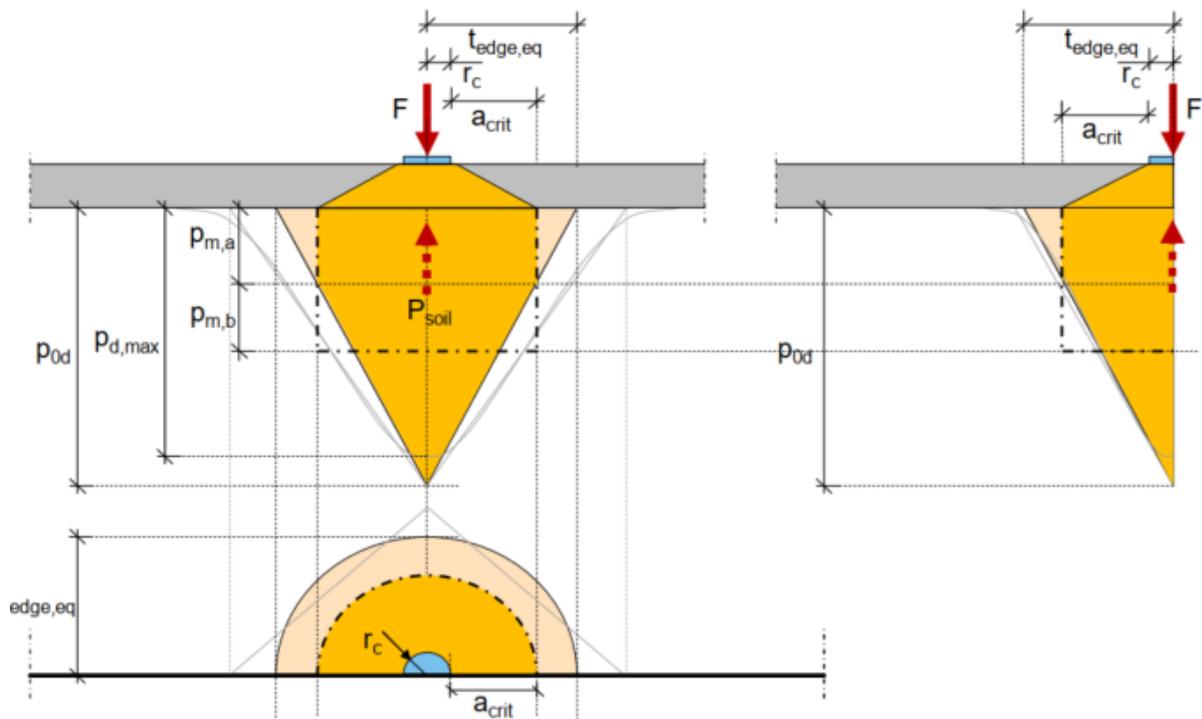


Fig. E1-9 Point load punching at the edge/joint of the slab

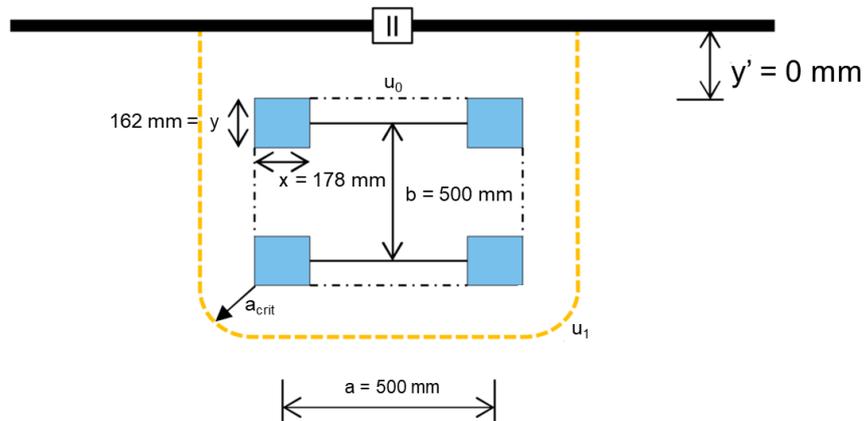


Fig. E1-10 Punching – Governing load case: 4 container point loads at the joint

Hence, to resist punching shear, it would be necessary to increase either the fibre dosage to achieve an f_{R3} value of $1222 / 1139 \times 3.7 = 4.0 \text{ MPa}$ or the slab thickness to about 420 mm.

2.2 Foundation Slabs

The procedure for the design of foundation slabs typically involves the following steps:

1. Preliminary Design:

- Determine the slab thickness based on initial calculations considering buoyancy forces and dead weight.
- Select appropriate concrete mix and reinforcement details (e.g., bar diameter, spacing, and cover).

2. Structural Analysis:

- Perform structural analysis to determine the bending moments, shear forces, and punching forces acting on the slab.
- Use finite element modelling (FEM) or other suitable methods to analyse load distribution and structural behaviour.

3. Reinforcement Design:

- Design the top and bottom reinforcement layers to resist the calculated bending moments and shear forces.
- Ensure the reinforcement spacing and cover meet the durability and structural requirements.

4. Crack Width Calculation:

- Calculate the expected crack widths due to bending moments and restraint shrinkage.
- Ensure the calculated crack widths are within the acceptable limits specified for the exposure class and intended work life.

5. Bending Moment Capacity:

- Calculate the cross-sectional bending moment capacity of the slab.
- Verify that the bending moment capacity is sufficient for the expected loads. If not, design additional reinforcement as needed.

6. **Shear and Punching Capacity:**

- Determine the shear and punching capacity of the slab, considering the contribution of steel fibres if used.
- Ensure the slab can resist the calculated shear and punching forces.

7. **Detailing and Documentation:**

- Prepare detailed drawings and specifications for the foundation slab, including reinforcement layout, concrete mix, and construction details.
- Document all calculations and design assumptions for review and approval.

This procedure ensures that the foundation slab is designed to safely support the intended loads while meeting durability and performance requirements.

References

ACI 318-25: Building Code Requirements for Structural Concrete (which includes provisions for fibre-reinforced concrete). American Concrete Institute, USA.

ACI 544.1R-96: Report on Fiber Reinforced Concrete. American Concrete Institute, USA.

ACI 544.4R-18: Guide for Design with Fiber-Reinforced Concrete. American Concrete Institute, USA.

fib Model Code 2010: International Federation for Structural Concrete (*fib*) guidelines for the design of fibre-reinforced concrete structures.

SS EN 1992-1-1:2008(2015): Eurocode 2: Design of concrete structures - Part 1-1: General rules and rules for buildings.

Example 2 – Car Park Foundation Slab

This example deals with the design of a pile-supported car park basement slab (Basement 3) of a high-rise apartment building (Fig. E2-1). The thickness of the slab is 800 mm, and the slab is subjected to a water pressure up to 7.5 m, due to the difference between buoyancy forces and dead weight of the slab.

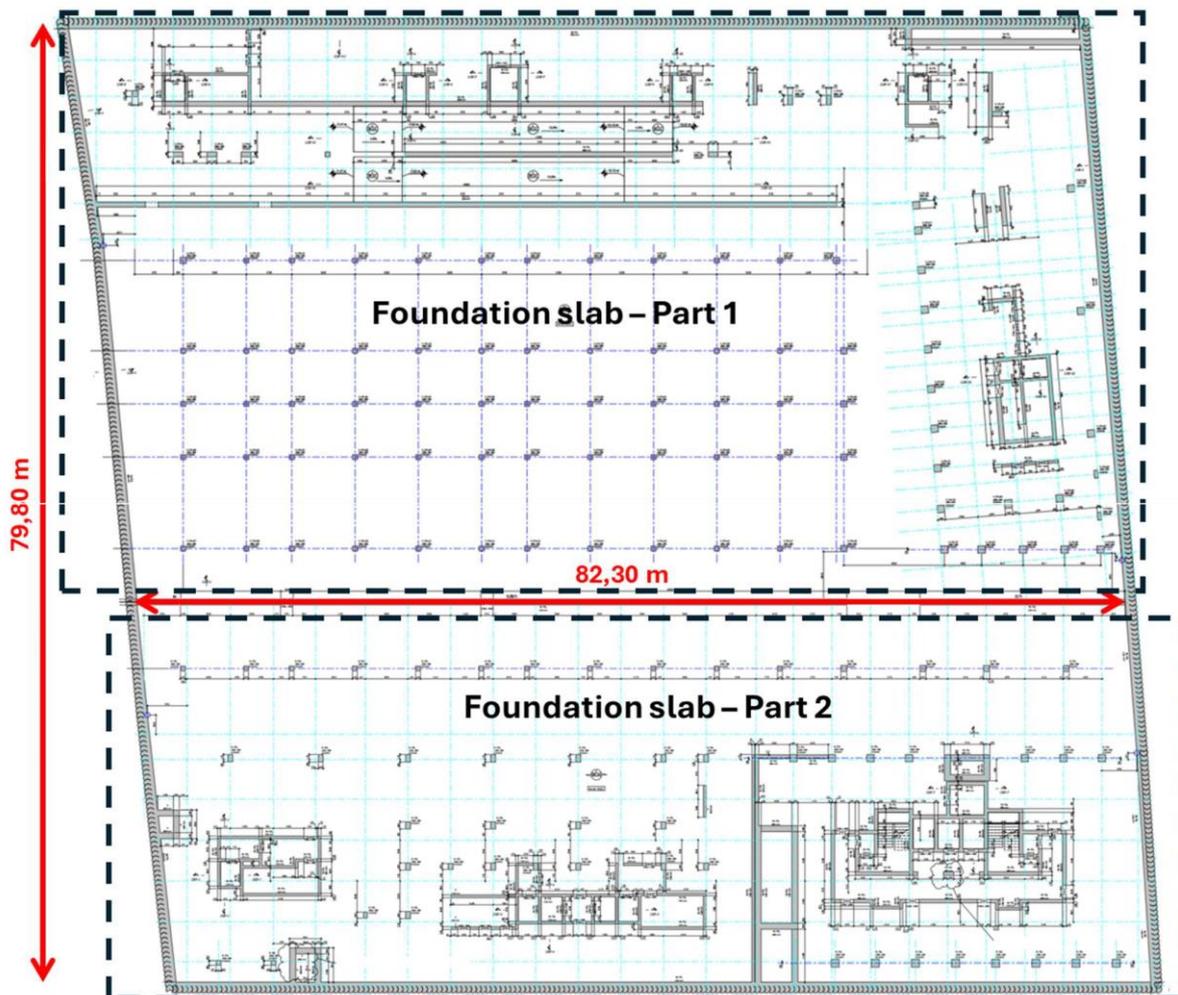


Fig. E2-1 Layout of Foundation Slab

Design Data

- Intended work life: 50 years; Exposure class: XD3 – Car park slab
- Thickness: 800 mm; Nominal concrete cover: $c_{nom} = 45$ mm

- Concrete: C35/45
- Reinforcement: top bars - H20–125 BW; bottom bars – H20–125 BW
- Characteristic residual flexural tensile strength steel fibres:

$$f_{R1} = 4.0 \text{ MPa}$$

$$f_{R3} = 4.6 \text{ MPa}$$

The characteristic residual tensile strengths are obtained as:

$$f_{ft,R1} = 0.45 \cdot f_{R1} = 1.80 \text{ MPa} \quad (\text{SS 674, Eq. 3.1})$$

$$f_{ft,R3} = 0.37 \cdot f_{R3} = 1.70 \text{ MPa} \quad (\text{SS 674, Eq. 3.2})$$

The design residual tensile strengths in ULS are:

$$f_{ftd,R1} = \eta_f \cdot \eta_{det} \cdot \frac{f_{ft,R1}}{\gamma_f} = 1 \cdot 1.4 \cdot \frac{1.80}{1.5} = 1.68 \text{ MPa} \quad (\text{SS 674, Eq. 3.3})$$

$$f_{ftd,R3} = \eta_f \cdot \eta_{det} \cdot \frac{f_{ft,R3}}{\gamma_f} = 1 \cdot 1.4 \cdot \frac{1.70}{1.5} = 1.59 \text{ MPa} \quad (\text{SS 674, Eq. 3.4})$$

The design residual tensile strength in SLS is:

$$f_{ftd,R1} = \eta_f \cdot \eta_{det} \cdot \frac{f_{ft,R1}}{\gamma_f} = 1 \cdot 1.4 \cdot \frac{1.80}{1} = 2.52 \text{ MPa} \quad (\text{SS 674, Eq. 3.3})$$

Crack width calculation

The limiting calculated crack width, w_{\max} , according to SS674:2021, Table 4 – Recommended values of w_{\max} for fibre concrete only with respect to durability, for exposure class XD3, and an intended working life of 50 years, is 0.2 mm.

Calculation of crack widths – Restraint shrinkage

A distinction must be made between the calculation of crack widths due to the bending moments in SLS and the calculation of crack widths due to restraint shrinkage. Since the bending moments in SLS are commonly not governing, in this example, the calculation for restraint shrinkage is performed. Nonetheless, the tensile stresses due to the SLS bending moments need to be checked according to the same calculation principles.

The direct crack width calculation for restraint shrinkage is as follows:

$$w_k = s_{r,max} \cdot (\varepsilon_{sm} - \varepsilon_{cm}) \quad (\text{SS 674, Eq. 7.5})$$

where

$$s_{r,max} = k_3 \cdot c + k_1 \cdot k_2 \cdot k_4 \cdot (1 - k_f) \cdot \frac{\phi}{\rho_{p,eff}} \quad (\text{SS 674, Eq. 7.8})$$

in which $k_1 = 0.8$; $k_2 = 1$ (axial restraint/pure tension); $k_3 = 3.4$; $c = c_{nom} = 45$ mm; $k_f = f_{ftd,R1}/f_{ctm} = 1.68/3.2 = 0.53$; $\phi = 20$ mm. The value of $\rho_{p,eff} = A_s/h_{c,eff}$ where $A_s = 2512$ mm²/m; $h_{c,eff} = \min \{2.5(c_{nom} + \phi/2); (h - x)/3; h/2\} = 2.5(c_{nom} + \phi/2) = 2.5 \times (45 + 20/2) = 137.5$ mm. That is, $\rho_{p,eff} = A_s/h_{c,eff} = 2512/(1000 \times 137.5) = 0.01827$. Hence,

$$s_{r,max} = 3.4 \cdot 45 + 0.8 \cdot 1 \cdot 0.425 \cdot (1 - 0.53) \cdot \frac{20}{0.01827} = 328 \text{ mm}$$

Also,

$$(\varepsilon_{sm} - \varepsilon_{cm}) = \frac{(1 - k_f) \cdot \left(\sigma_{s,fict} - k_t \cdot \frac{f_{ct,eff}}{\rho_{p,eff}} \cdot (1 + \alpha_e \cdot \rho_{p,eff}) \right)}{E_s} \geq 0.6 \cdot (1 - k_f) \frac{\sigma_{s,fict}}{E_s}$$

(SS 674, Eq. (7.7))

where $k_t = 0.4$ (long-term loading); $f_{ct,eff} = f_{ctm} = 3.2$ MPa; $\alpha_e = E_s/E_{cm} = 200/34 = 5.88$; and $\sigma_{s,fict}$ is the fictitious stress in the tension bar reinforcement assuming a cracked section but neglecting the effect of fibres, given by:

$$A_s \cdot \sigma_{s,fict} = k_c \cdot k \cdot f_{ct,eff} \cdot A_{ct} \quad (\text{SS EN 1992, Eq. 7.1})$$

where $k_c = 1.0$ (for pure tension); $k = 0.65$ (for $h = 800$ mm); and $A_{ct} = bh_{ct} = bh/2$ (for symmetrically reinforced rectangular section). With $A_s = 3140$ mm²/m; $k_f = 0.53$; and $f_{ct,eff} = 3.2$ MPa,

$$\sigma_{s,fict} = 1.0 \cdot 0.65 \cdot 3.2 \cdot 1000 \cdot 400 / 2512 = 331.2 \text{ MPa}$$

for which

$$\begin{aligned} (\varepsilon_{sm} - \varepsilon_{cm}) &= \frac{(1 - 0.53) \cdot \left(331.2 - 0.4 \cdot \frac{3.2}{0.01827} \cdot (1 + 5.88 \cdot 0.01827) \right)}{200000} = 596 \cdot 10^{-6} \\ &> 0.6 \cdot (1 - 0.53) \frac{331.2}{200000} = 467 \cdot 10^{-6} \end{aligned}$$

Therefore,

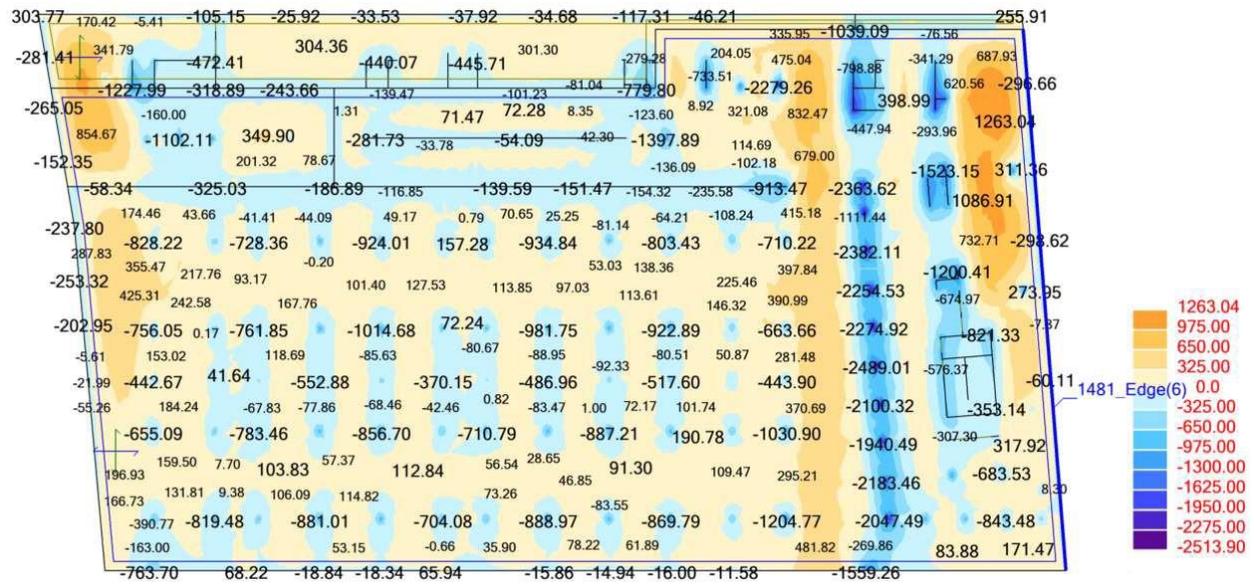
$$w_k = s_{r,max} \cdot (\varepsilon_{sm} - \varepsilon_{cm}) = 328 \cdot 596 \cdot 10^{-6} = 0.195 \text{ mm} < 0.20 \text{ mm}$$

Design for Bending

In this design example, it is not the purpose to go into detail the full load distribution of the columns, piles and other loads on the foundation slab. The design bending moments in ULS are determined by the engineering office using a (non-)linear or plastic analysis in a FEM model.

Bending moments M_{xx} [kNm/m]

Foundation slab – Part 1



Foundation slab – Part 2

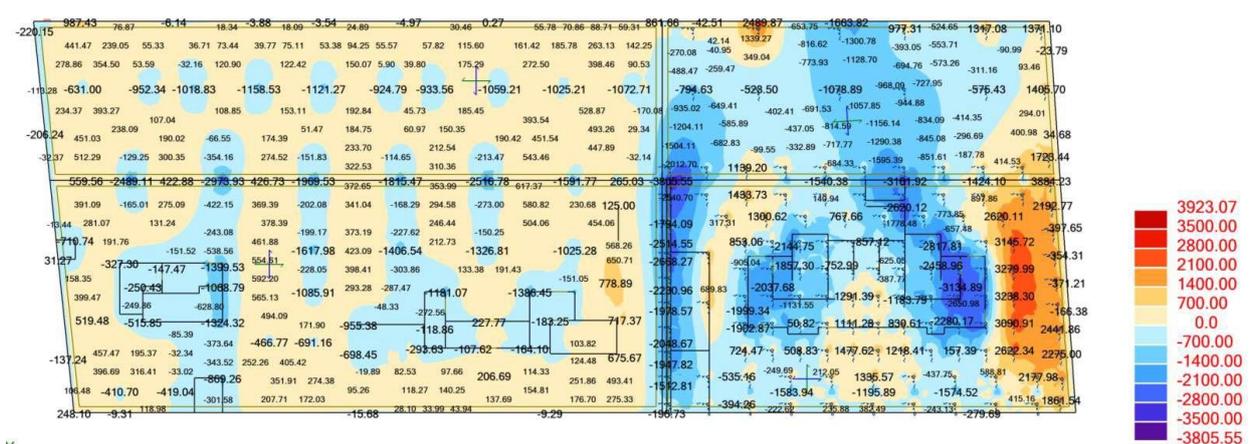
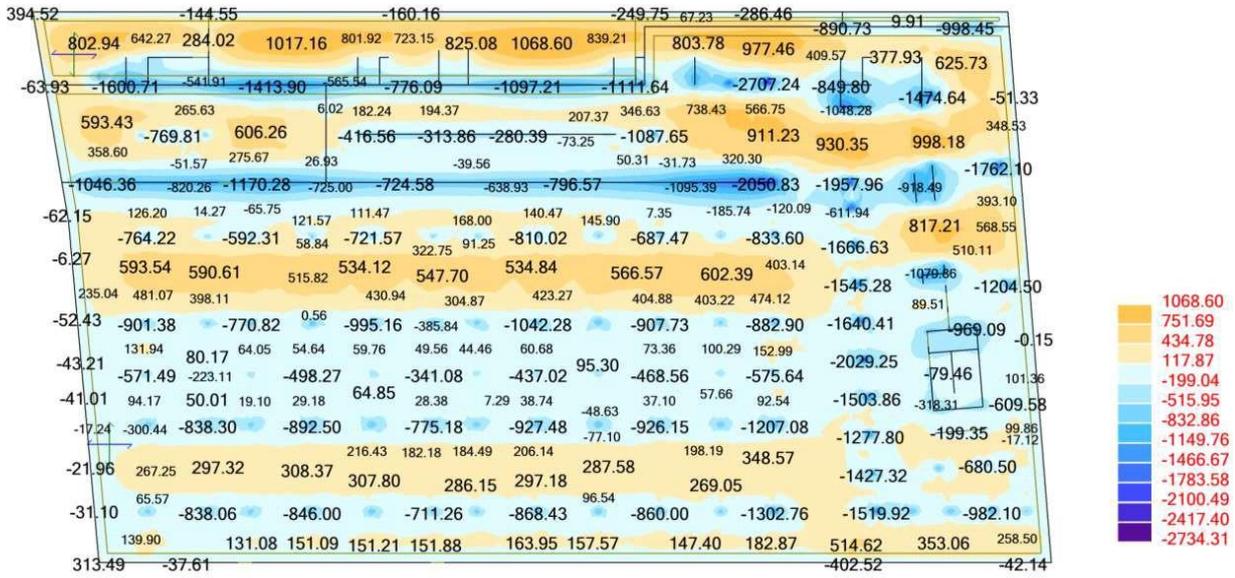


Fig. E2-2 Bending Moments M_{xx}

Bending moments M_{yy} [kNm/m]

Foundation slab – Part 1



Foundation slab – Part 2

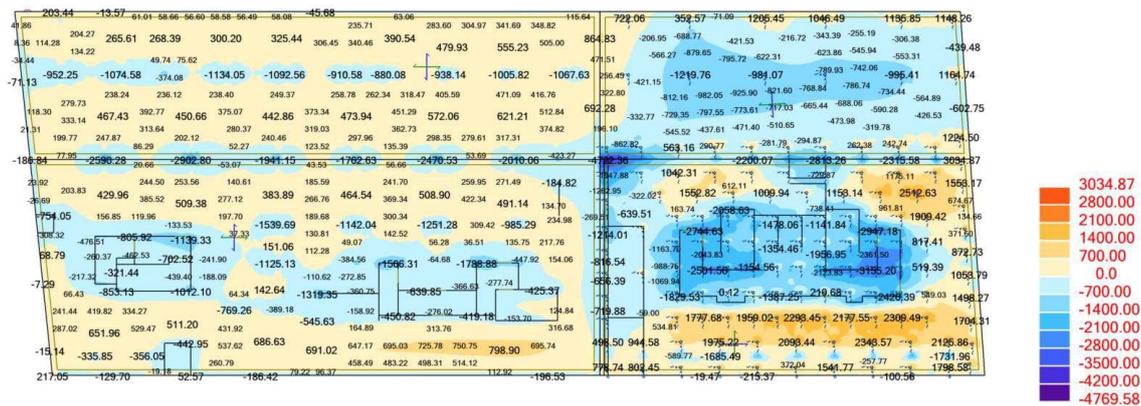


Fig. E2-3 Bending Moments M_{yy}

The cross-sectional bending moment capacity of the proposed foundation slab with the combined reinforcement can be determined as follows. In this example, the simplified stress distribution for ULS in SS 674, Fig. 6.1(b) [Fig. 2(b) in this Guide] is used. The maximum design tensile strength at the extreme tensile fibre is:

$$\sigma_{ft} = f_{ftd,R1} - \frac{\varepsilon_{ft}}{\varepsilon_{ftu}} \cdot (f_{ftd,R1} - f_{ftd,R3}) \quad (\text{SS 674, Eq. 6.1})$$

with

$$\varepsilon_{ct} = \frac{f_{ctd}}{E_c} = \frac{1.49}{34000} = 4.39 \cdot 10^{-5} \quad (\text{SS 674, Fig. 3.3})$$

$$\varepsilon_{ftu} = \varepsilon_{ct} + \frac{w_u}{l_{cs}} = 4.39 \cdot 10^{-5} + \frac{2.5}{0.8 \cdot 800} = 3.95 \cdot 10^{-3} \quad (\text{SS 674, Fig. 3.3})$$

Considering the idealized parabolic-rectangular stress-strain relation for concrete in compression, with $\varepsilon_{c2} = 0.002$ and $\varepsilon_{cu2} = 0.0035$, and taking the extreme tensile fibre strain, $\varepsilon_{ft} = \varepsilon_{ftu} = 0.00395$, and with $d' = 55$ mm, $d = 735$ mm, the following solution is obtained:

$x = 183.4$ mm; $\varepsilon_c = 0.00118 < \varepsilon_{c2} = 0.002$; $\varepsilon_{ft} = 0.00395$ (as assumed)

Force in concrete compression zone, $C = 1719.4$ kN/m

Force in compression steel, $C_s = 381.0$ kN/m

Forces in tension, $F_{f1} = 980.4$ kN/m; $F_{f2} = 27.7$ kN/m; $F_s = 1092.2$ kN/m;

$F_{f1} + F_{f2} + F_f = 2100.3$ kN/m $\approx C + C_s = 2100.4$ kN/m

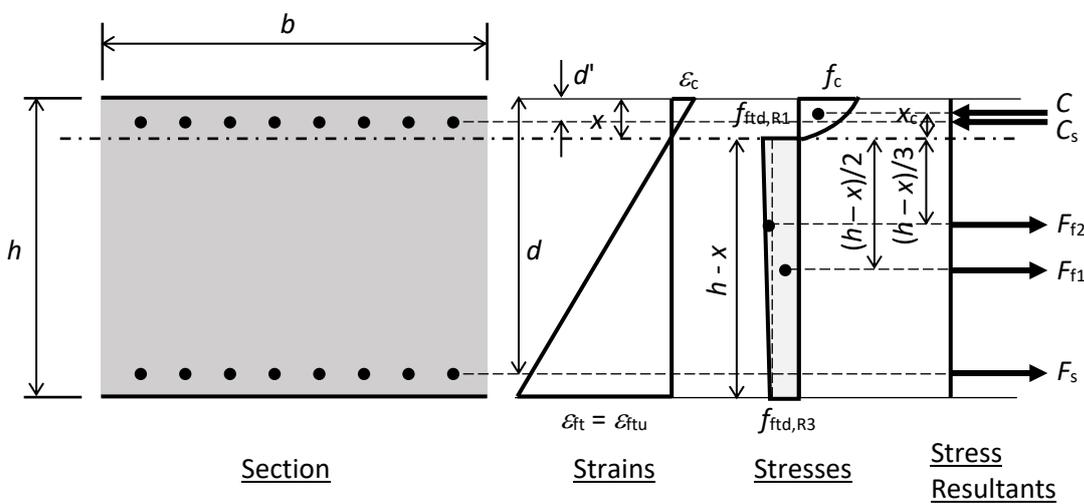


Fig. E2-4 Ultimate Moment Capacity

Moment capacity of section,

$$M_{Rd} = M'_{Rd} = [1719.3 \times 118.5 + 381.0 \times (183.4 - 65) + 980.4 \times (800 - 183.4) / 2 + 27.7 \times (800 - 183.4) / 3 + 1092.2 \times (735 - 183.4)] \times 10^{-3} = 1159 \text{ kNm/m}$$

NOTE The bending moment capacity of the proposed solution is not sufficient for all areas. In the areas where the design bending moment exceeds the moment capacity, additional reinforcement will be required. For example, to achieve a bending moment capacity of 2815 kNm/m, it would be necessary to provide H32 bars at 100 mm both ways, as both top and bottom reinforcement with the same fibre reinforced concrete.

Design for Shear and Punching

The design shear and punching forces are determined from an analysis using a FEM model. The shear and punching capacity of the foundation slab is increased due to the use of steel fibres, and determined as follows.

Resistant shear forces

The shear capacity of the steel fiber section can be determined from:

$$V_{Rd,cf} = \{(0.18/\gamma_c) \cdot k \cdot [100\rho(1+7.5f_{ft,R3}/f_{ctk}) \cdot f_{ck}]^{1/3}\} \cdot b_w d \geq \{0.035k^{3/2}f_{ck}^{1/2}\} \cdot b_w d$$

With $k = 1 + \sqrt{(200/d_{avg})} = 1 + \sqrt{(200/735)} = 1.52 < 2$; $\rho = 2512/(1000 \times 735) = 0.00342 < 0.02$; $f_{ctk} = 0.7f_{ctm} = 0.7 \times 3.2 = 2.24$ MPa, $f_{ft,R3} = 1.7$ MPa,

$$V_{Rd,cf} = \{(0.18/1.5) \cdot 1.52 \cdot [100 \cdot 0.00342(1+7.5 \cdot 1.7/2.24) \cdot 35]^{1/3}\} \cdot 1000 \cdot 735 / 1000$$
$$= 0.786 \cdot 735 = 578 \text{ kN/m}$$

$$\geq \{0.035k^{3/2}f_{ck}^{1/2}\} \cdot b_w d = 0.035 \cdot (1.52)^{3/2} (35)^{1/2} \cdot 1000 \cdot 735 / 1000 = 0.388 \cdot 735 = 285 \text{ kN/m}$$

Punching shear resistance

The punching shear resistance for ground-supported slab is determined from:

$$V_{Rd,cf} = \{(0.18/\gamma_c) \cdot k \cdot [100\rho(1+7.5f_{ft,R3}/f_{ctk}) \cdot f_{ck}]^{1/3}\} = 0.786 \text{ MPa}$$

The foundation is pile supported with a pile grid of approximately 4.5 m x 5 m; the internal piles have a diameter of 500 mm and a load bearing capacity of $V_{Ed} = 1500$ kN in ULS. The checks for control perimeters u_o (face of loaded area) and u_1 (basic control perimeter) are as follows.

Face shear:

$$u_o = \pi \cdot 500 = 1571 \text{ mm}; \quad \nu = 0.6 (1 - f_{cd}/250) = 0.516$$

$$v_{Ed} = \beta V_{Ed} / (u_o d) = 1.15 \times 1500 \times 10^3 / (1571 \times 735) = 1.49 \text{ MPa}$$

$$< v_{Rd,max} = 0.5 \nu f_{cd} = 0.5 \times 0.516 \times 35 / 1.5 = 6.02 \text{ MPa}$$

Shear at basic control perimeter (at 2d from face of pile):

$$u_1 = \pi \cdot (500 + 2 \times 2 \times 735) = 10,808 \text{ mm}; \quad \nu = 0.6 (1 - f_{cd}/250) = 0.516$$

$$v_{Ed} = \beta V_{Ed} / (u_o d) = 1.15 \times 1500 \times 10^3 / (10,808 \times 735) = 0.217 \text{ MPa}$$

$$< v_{Rd,cf} = 0.786 \text{ MPa}$$

2.3 Foundation Beams

Foundation beams can be used to support line loads and transmit the supported loads to piles through pile caps. The examples in this section illustrate the design of foundation beams using FRC. The design follows general procedure and is not detailed here.

Example 3 – Foundation Beams

Fig. E3-1 shows the foundation for an oil refinery structure. It consists of tie-beams supported on pile caps, each supported by two circular piles. This example considers the design of tie-beams, TB1 to TB6, located between grids N1/M2 and 1a/1c.

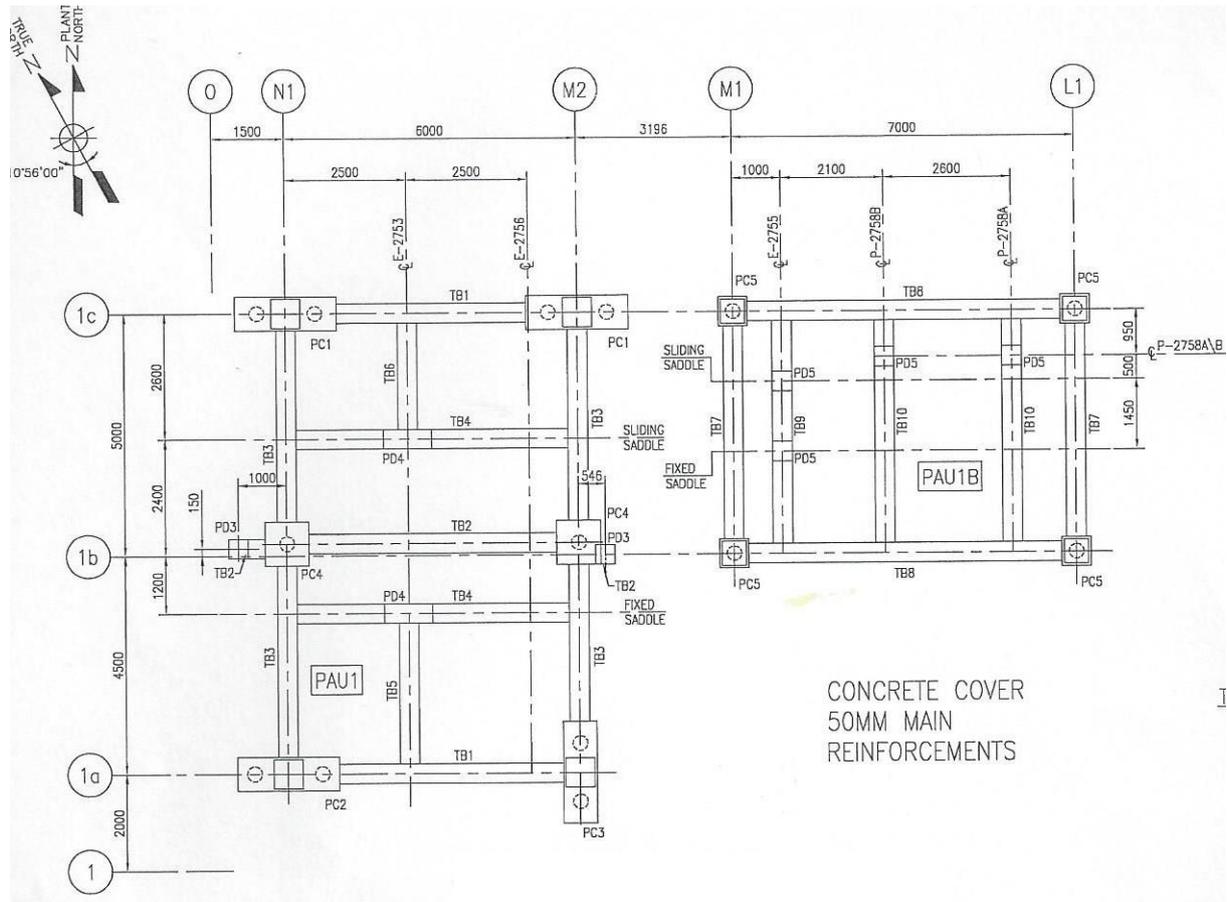


Fig. E3-1 Foundation Layout Plan

All tie-beams measure 400 mm in width and 400 mm in height. It is assumed for simplicity that the effective depth d is 340 mm for all beams, although in practice, consideration to avoid clashing of bars is needed. The concrete strength class is C40/50- $R_{14}R_{35}$ (Class 4d), which is achievable using 40 kg/m³ of hooked-end steel fibres. The mean concrete tensile strength f_{cm} is 3.5 MPa, and characteristic concrete tensile strength f_{ctk} is 2.45 MPa.

Design for Ultimate Limit State

The design values of action effects at Ultimate Limit States for the tie-beams are shown in Table E3-1. The maximum sagging moments occur at mid-spans and the maximum hogging moments occur at supports. Beams TB1, TB3 and TB4 are subjected to transverse moments in the horizontal plane; they can be evaluated from first principles based on strain compatibility and force equilibrium and will not be considered here.

Table E3-1 Action Effects at Ultimate Limit State

Beam	Span (m)	Sagging M_{Ed} (kNm)	Hogging M_{Ed} (kNm)	Transverse M_{Ed} (kNm)	Shear V_{Ed} (kN)	Axial Tension N_{Ed} (kN)
TB1	6	175.0	81.0	19.0	186.8	-
TB2	6	145.8	42.4	-	104	15
TB3	5/4.5	100.3	217.1	17.6	190	319
TB4	6	220.3	55.0	41.5	160	-
TB5/TB6	3.3/2.6	27.4	46.5	-	57	34

Design for bending. - Assuming fibre orientation factor, $\eta_f = 0.5$, and structural redundancy factor $\eta_{det} = 1.0$ leads to:

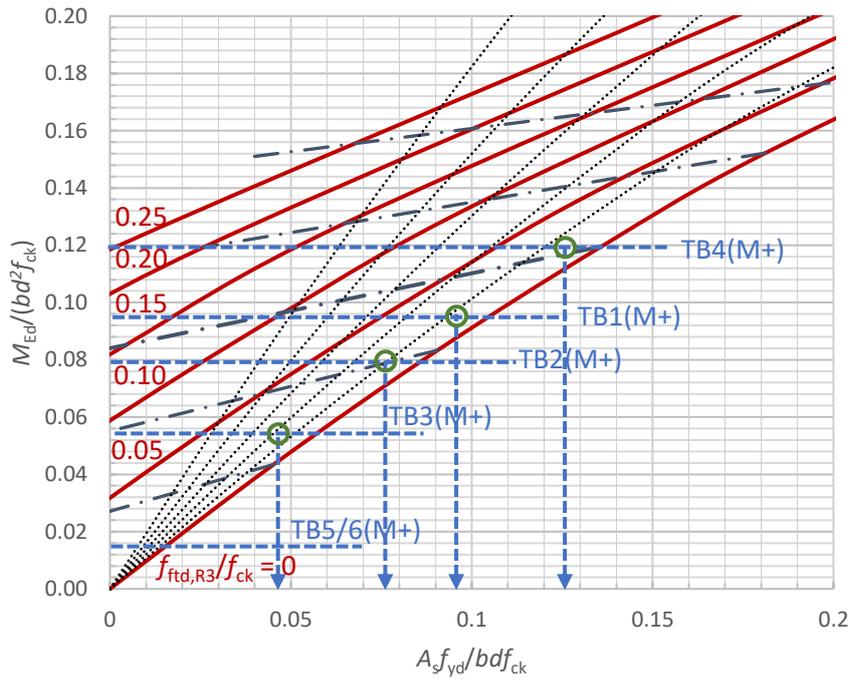
$$f_{ftd,R1} = \eta_f \cdot \eta_{det} \cdot \frac{f_{ft,R1}}{\gamma_f} = 0.5 \cdot 1.0 \cdot \frac{0.45(4)}{1.5} = 0.6 \text{ MPa} \quad (\text{SS 674, Eq. 3.3})$$

$$f_{ftd,R3} = \eta_f \cdot \eta_{det} \cdot \frac{f_{ft,R3}}{\gamma_f} = 0.5 \cdot 1.0 \cdot \frac{0.37(5)}{1.5} = 0.62 \text{ MPa} \quad (\text{SS 674, Eq. 3.4})$$

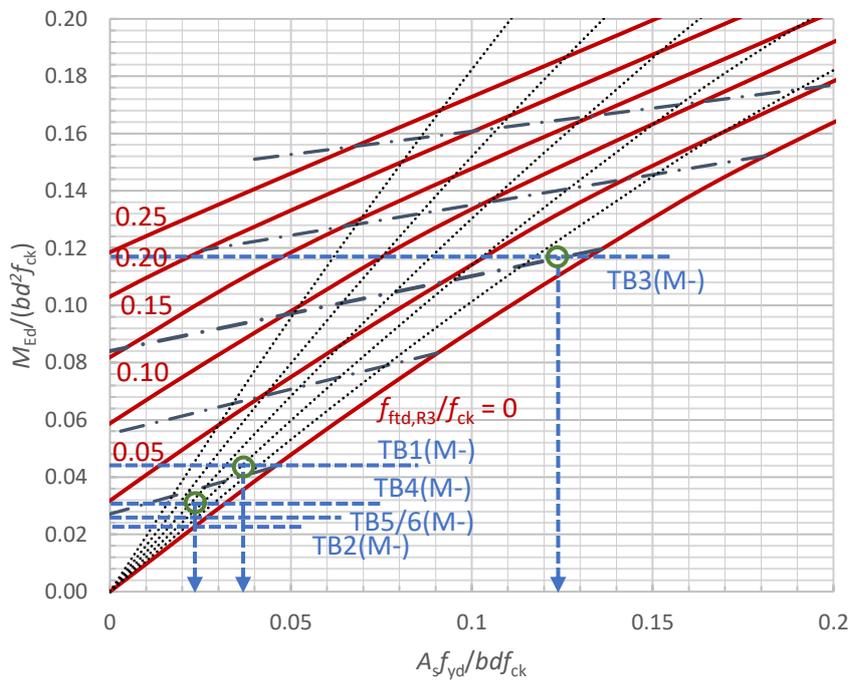
NOTE η_f may be taken as 1.0 if concrete is of consistency classes S2-S4, SS EN 206: 2014(2024)+A1:20242024.

Also, $f_{ftd,R3}/f_{ck} = 0.62/40 = 0.015$, and $d/h = 340/400 = 0.85$. Using Design Chart C-2, the required steel reinforcement are obtained as shown in Table E2-2. The minimum reinforcement area $A_{s,min}$ following Eq. (9.1N) of SS EN 1992-1-1 [refer to Design Guide (Tan

2022)] is given by $\frac{A_{s,min}}{bd} = 0.26 \cdot \frac{f_{ctm}}{f_{yk}} = 0.26 \cdot \frac{3.5}{500} = 0.00182$ or $\frac{A_{s,min} f_{yd}}{b d f_{ck}} = 0.00182 \cdot \frac{500}{40} = 0.02$.



(a) Design for Sagging Moments



(b) Design for Hogging Moments

Fig. E3-2 Design for Moments using Chart C-2 ($d/h = 0.85$)

Table E3-2 Tension Steel Reinforcement Requirements

Beam	M_{Ed} (kNm)	M_{Ed}/bd^2f_{ck}	$A_s f_{yd}/bd f_{ck}^1$	$A_{s,reqd}/bd$	$A_{s,reqd}^2$ (mm ²)	Bars	$A_{s,prov}^2$ (mm ²)	$A_{s,prov}/bd$
Maximum sagging moment at mid-span:								
TB1	175.0	0.095	0.096	0.00883	1201	4H20	1256	0.00924
TB2	145.8	0.079	0.077	0.00708	963	2H25	980	0.00721
TB3	100.3	0.054	0.048	0.00442	601	2H25	980	0.00721
TB4	220.3	0.119	0.127	0.01168	1589	4H25	1960	0.01441
TB5/TB6	27.4	0.015	0.020 ³	0.00074	250	2H16	402	0.00296
Maximum hogging moment at support:								
TB1	81.0	0.044	0.037	0.00340	463	3H16	603	0.00443
TB2	42.4	0.023	0.020 ³	0.00184	250	2H16	402	0.00296
TB3	217.1 ⁴	0.117	0.124	0.01141	1551	4H25	1960	0.01441
TB4	55.0	0.030	0.023	0.00212	288	2H16	402	0.00296
TB5/TB6	46.5	0.025	0.020 ³	0.00184	250	2H16	402	0.00296

Note: ¹ Values obtained from Chart C-2 (Fig. E3-2);

² $A_{s,reqd}$: required area of tension reinforcement; $A_{s,prov}$: provided area of tension reinforcement.

³ Minimum tension requirement applies (Eq. (9.1N), SS EN 1992-1-1);

⁴ At mid-support.

Design for Axial Tension. – Beams TB2, TB3 and TB5/6 are subjected to axial tension, and additional longitudinal bars must be provided. This means that for the tensile zone, the longitudinal reinforcement should be able to carry a tensile force of $0.5N_{Ed}$ in addition to the force due to the bending moment. For TB3, the additional area of tensile reinforcement is $\frac{1}{2} \times 319 \times 10^3 / (500/1.15) = 367 \text{ mm}^2$, which in addition to 1551 mm^2 due to bending, is catered for by 4H25 bars. For other beams TB2 and TB5/6, the additional area required is small (17 and 39 mm^2 respectively) and catered for by the reinforcement provided as shown in Table E3-2.

Design for Shear. – The shear capacity of the beams without shear reinforcement is given by SS 674:2021, Eq. (6.3) as:

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

Due to the axial tension, the shear capacity is reduced. Beam TB3 is the most critical with $V_{Ed} = 190 \text{ kN}$ and $N_{Ed} = -319 \text{ kN}$ (see Table E3-1). With $\gamma_c = 1.5$, $k = 1 + \sqrt{(200/340)} = 1.77$, $\rho = 0.0144$, $f_{ft,R3}/f_{ctk} = 0.37 \times 5 / 2.45 = 0.755$, $\sigma_{cp} = -319 \times 10^3 / (400 \times 400) = 1.99 \text{ MPa}$,

$$V_{Rd,cf} = \{(0.18/1.5) \cdot 1.77 \cdot [100 \times 0.0144(1 + 7.5 \times 0.755) \cdot 40]^{1/3} - 0.15 \times 1.99\} \times 400 \times 340 \times 10^{-3}$$

$$= \{1.54 - 0.30\} \times 400 \times 340 \times 10^{-3} = 1.25 \times 400 \times 340 \times 10^{-3} = 169 \text{ kN} < V_{Ed} = 190 \text{ kN}$$

Therefore, provide as recommended by Design Guide (Tan 2022) (assuming $\theta = 45^\circ$):

$$V_s \equiv \frac{A_{sw}}{s} z f_{wd} = \frac{1}{0.75} \cdot (V_{Ed} - V_{Rd,cf}) = \frac{1}{0.75} \cdot (190 - 169) = 28 \text{ kN}$$

$$\frac{A_{sw}}{s} = \frac{28 \times 10^3}{0.9 \times 340 \times \left(\frac{500}{1.15}\right)} = 0.212 \text{ mm} < 0.08 b_w \cdot \frac{\sqrt{f_{ck}}}{f_{yk}} = 0.08 \times 400 \cdot \frac{\sqrt{40}}{500} = 0.405 \text{ mm}$$

Note that hanger bars are also needed in TB3 to support Beam TB4 (see Fig. E2-1). The required area of hanger bars is $160 \times 10^3 / (500/1.15)$ over 400 mm or 0.92 mm. Hence, the required total $A_{sw}/s = 0.405 + 0.92 = 1.325 \text{ mm}$. Provide two-legged (2L) H10 links at 100 mm spacing ($A_{sw}/s = 1.570 \text{ mm}$) within 400 mm from the intersection with TB4 and H8 links at 200 mm spacing elsewhere ($A_{sw}/s = 0.53 \text{ mm}$).

For the other beams, the required shear reinforcement is shown in Table E3-3.

Table E3-3 Shear Capacity and Reinforcement Requirements

Beam	ρ^1	N_{Ed} (kN)	σ_{cp} (MPa)	$V_{Rd,cf}$ (kN)	V_{Ed} (kN)	A_{sw}/s	Links provided ²	$(A_{sw}/s)_{prov}$
TB1	0.00443	0	0	141	187	0.455	2L-H8-200	0.503
TB2	0.00296	15	-0.09	122	104	0.405 ³	2L-H8-200	0.503
TB3	0.01441	319	-1.99	169	190	0.405 ³	2L-H8-200/ 2L-H10-100 ⁴	0.503/ 1.570 ⁴
TB4	0.00296	0	0	124	160	0.405 ³	2L-H8-200	0.503
TB5/TB6	0.00296	34	-0.21	119	57	0.405 ³	2L-H8-200	0.503

Note: ¹ Based on support section, where shear force is maximum;

² 2L refers to two-legged links;

³ Minimum shear reinforcement governs;

⁴ Within 400 mm from intersection with TB4

Design for Serviceability Limit State

Crack Width Limitation. – Beam TB-3 is subjected to a bending moment $M_{Ed} = 217$ kNm and an axial force $N_{Ed} = 319$ kN at ULS. This is the most critical case with regard to crack width. Assuming a mixed safety factor of 1.35 for both dead and variable loads, the moment and axial tension acting at SLS are 161 kNm and 236 kN respectively.

Using the approach described in Appendix B of Design Guide (Tan 2022), the following information is obtained.

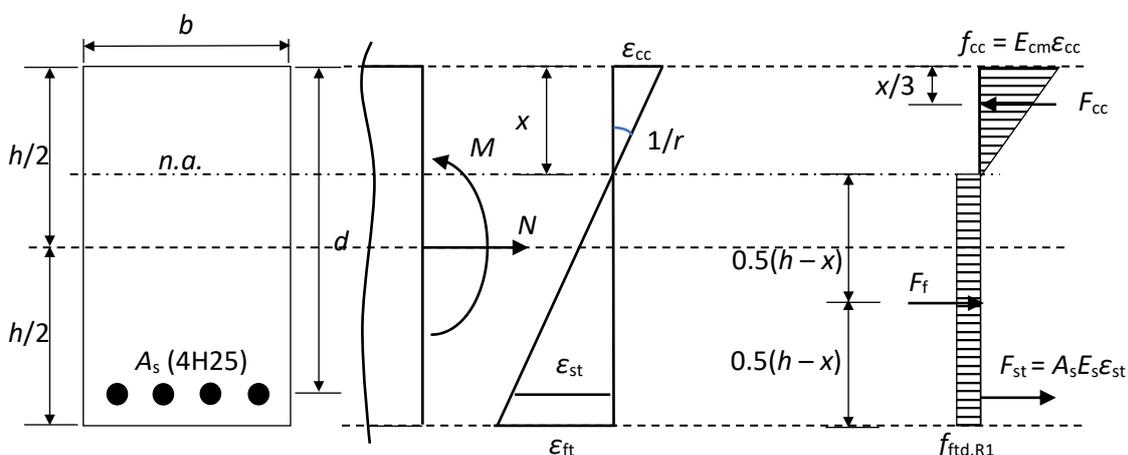


Fig. E3-3 Strains and Stresses at SLS

$$b = 400 \text{ mm}; h = 400 \text{ mm}; d = 340 \text{ mm}; f_{ck} = 40 \text{ MPa}; E_{cm} = 35 \text{ GPa}; f_{ftd,R1} = 0.6 \text{ MPa};$$

$$1/r = 0.00636 \text{ mm}^{-1}; x = 98.8 \text{ mm}; \varepsilon_{cc} = 0.628 \times 10^{-3} \text{ mm/mm}; f_{cc} = 22.1 \text{ MPa};$$

$$\varepsilon_{st} = 1.534 \times 10^{-3} \text{ mm/mm}; f_{st} = 306.8 \text{ MPa};$$

$$\varepsilon_{ft} = 1.916 \times 10^{-3} \text{ mm/mm} < \varepsilon_{ftu} = \varepsilon_{ct} + \frac{2.5}{0.8h} = \frac{1.64}{35000} + \frac{2.5}{0.8 \times 400} = 7.86 \times 10^{-3} \text{ mm/mm}$$

Check Equilibrium:

$$F_{cc} = \frac{1}{2} \times 400 \times 98.8 \times 22.1 \times 10^{-3} = 437 \text{ kN}$$

$$F_{st} = 1960 \times 306.8 \times 10^{-3} = 601 \text{ kN}$$

$$F_f = 400 \times (400 - 98.9) \times 0.6 \times 10^{-3} = 72 \text{ kN}$$

$$\therefore N = F_{cc} - F_{st} - F_{st} = 437 - 601 - 72 = -236 \text{ kN} = N_{Ed,SLS}$$

$$\begin{aligned} M &= F_{cc} (h/2 - x/3) + F_{st} (d - h/2) + F_{st} (x/2) \\ &= [437x(200 - 98.8/3) + 601x(340 - 200) + 72x(98.8/2)] \times 10^{-3} \\ &= 73 + 84 + 4 = 161 \text{ kNm} = M_{Ed,SLS} \end{aligned}$$

The maximum surface crack width w_k is given by:

$$w_k = s_{r,max} \cdot (\varepsilon_{sm} - \varepsilon_{cm}) \quad (\text{SS 674, Eq. 7.5})$$

where

$$s_{r,max} = k_3 \cdot c + k_1 \cdot k_2 \cdot k_4 \cdot (1 - k_f) \cdot \frac{\phi}{\rho_{p,eff}} \quad (\text{SS 674, Eq. 7.8})$$

in which $k_1 = 0.8$; $k_2 = 0.5$; $k_3 = 3.4$; $c = 47.5 \text{ mm}$; $k_f = f_{ftd,R1}/f_{ctm} = 1.68/3.2 = 0.53$; $\phi = 25 \text{ mm}$.

The value of $\rho_{p,eff} = A_s/h_{c,eff}$ where $A_s = 1960 \text{ mm}^2$; $h_{c,eff} = \min \{2.5(c + \phi/2), (h - x)/3, h/2\} = \min \{2.5 \times 60, (400 - 98.8)/3, 400/2\} = \min \{150, 100.4, 200\} = 100.4 \text{ mm}$. That is, $\rho_{p,eff} = A_s/bh_{c,eff} =$

$1960/(400 \times 100.4) = 0.0488$. Since $k_f = \frac{f_{ftd,R1}}{f_{ctm}} = \frac{0.6}{3.5} = 0.171$, therefore

$$s_{r,max} = 3.4 \cdot 47.5 + 0.8 \cdot 0.5 \cdot 0.425 \cdot (1 - 0.171) \cdot \frac{25}{0.0488} = 234 \text{ mm}$$

Also,

$$(\varepsilon_{sm} - \varepsilon_{cm}) = \frac{\sigma_s - (k_t + (1 - k_t) \cdot k_f) \cdot \frac{f_{ct,eff}}{\rho_{p,eff}} \cdot (1 + \alpha_e \cdot \rho_{p,eff})}{E_s} \geq 0.6 \cdot \frac{\sigma_s}{E_s}$$

(SS 674, Eq. (7.7))

where $k_t = 0.4$ (long-term loading); $f_{ct,eff} = f_{ctm} = 3.5 \text{ MPa}$; $\alpha_e = E_s/E_{cm} = 200/35 = 5.71$; and $\sigma_s = f_{st} = 306.8 \text{ MPa}$, gives:

$$\begin{aligned} (\varepsilon_{sm} - \varepsilon_{cm}) &= \frac{306.8 - (0.4 + (1 - 0.4) \cdot 0.171) \cdot \frac{3.5}{0.0488} \cdot (1 + 5.71 \cdot 0.0488)}{200000} = 1304 \cdot 10^{-6} \\ &\geq 0.6 \cdot \frac{\sigma_s}{E_s} = 0.6 \cdot \frac{306.8}{200000} = 920 \cdot 10^{-6} \end{aligned}$$

Therefore,

$$w_k = s_{r,max} \cdot (\varepsilon_{sm} - \varepsilon_{cm}) = 234 \cdot 1304 \cdot 10^{-6} = 0.305 \text{ mm}$$

Deflection Limits. – This is checked using the limit span-depth ratio neglecting the effect of fibres and assuming all beams are simply-supported on both ends, except TB3 which is a two-span continuous beam. It is assumed that all beams excluding TB4, have compression steel bars comprising 2H16 bars, with compression steel ratio of 0.00296. For Beam TB4, the compression steel reinforcement consists of 4H16 bars, with a steel ratio of 0.00592.

Table E3-4 shows that all beams satisfy the limiting span-depth ratio.

Table E3-4 Limiting Span-Depth Ratios

Beam	Span, L (mm)	Effective depth, d (mm)	L/d	K	Tension steel ratio, ρ	Compression steel ratio, ρ'	limit L/d
TB1	6000	340	17.6	1	0.00924	0.00296	20.9 ¹
TB2	6000	340	17.6	1	0.00721	0.00296	25.5 ¹
TB3	5000	340	14.7	1.3	0.00721	0.00296	33.1 ¹
TB4	6000	340	17.6	1	0.01441	0.00592	18.6 ¹
TB5	3300	340	9.7	1	0.00296	na	55.8 ²
TB6	2600	340	7.6	1	0.00296	na	55.8 ²

Note: ¹ $\frac{L}{d} = K \left[11 + 1.5\sqrt{f_{ck}} \frac{\rho_0}{\rho - \rho'} + \frac{1}{12}\sqrt{f_{ck}} \sqrt{\frac{\rho'}{\rho_0}} \right]$ applies as $\rho > \rho_0$;

² $\frac{L}{d} = K \left[11 + 1.5\sqrt{f_{ck}} \frac{\rho_0}{\rho} + 3.2\sqrt{f_{ck}} \left(\frac{\rho_0}{\rho} - 1 \right)^{3/2} \right]$ applies as $\rho < \rho_0$;

$$\rho_0 = 10^{-3} \sqrt{f_{ck}} = 10^{-3} \sqrt{40} = 0.00633$$

2.4 Crack Control in Thick Basement Slabs

The reinforcement requirement in thick basement slabs in underground structures is often governed by the serviceability limit state of cracking. The two examples in this section illustrate the calculation of crack widths in fibre-reinforced concrete sections, and how it can lead to lesser reinforcement bars by using fibres without compromising on the ultimate flexural strength.

References

PD 6687-1:2020. Background Paper to the National Annexes to BS EN 1992-1-1, BS EN 1992-3 and BS EN 1992-4. BSI Standards Limited 2020, 13-15.

Example 4.1 – Crack Width Calculation for Tunnel Base Slab

A 1500 mm thick section from the base slab of an underground tunnel (in Fig. E4.1-1) is subjected to a design sagging bending moment $M_{Ed} = 2650 \text{ kNm/m}$ at midspan. Given that the concrete class is C35/45-R₁₃/R₃₃, the characteristic yield strength of reinforcement, $f_{yk} = 500 \text{ MPa}$, the creep coefficient $\varphi = 1.34$, and the allowable crack width $w_{k,limit} = 0.25 \text{ mm}$. The Serviceability Limit State (SLS) for cracking for this section is to be verified.

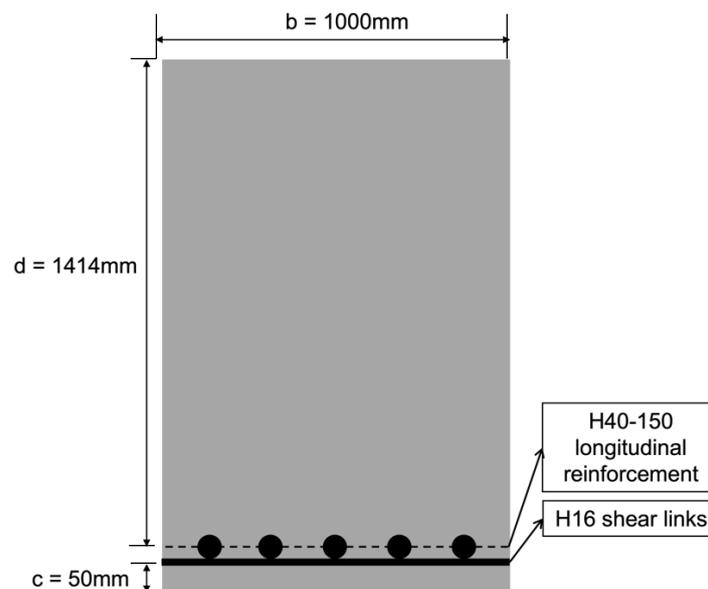


Fig. E4.1-1 Design section from the base slab of an underground tunnel

For concrete class C35/45-R₁₃/R₃₃, and assuming $\eta_f = 0.7^*$ and $\eta_{det} = 1$:

$$f_{ck} = 35 \text{ MPa}; E_{cm} = 22[(35 + 8)/10]^{0.3} = 34.1 \text{ GPa};$$

$$E_{c,eff} = E_{cm} / (1 + \varphi) = 34.1 / (1 + 1.34) = 14.5 \text{ GPa};$$

$$f_{ctm} = 3.2 \text{ MPa}; f_{ctd} = f_{ctk,0.05} / \gamma_c = 2.2 / 1.5 = 1.47 \text{ MPa};$$

$$f_{ftd,R1} = \eta_f f_{ft,R1} = 0.7 \times (0.45 \times 3) = 0.945 \text{ MPa};$$

$$k_f = f_{ftd,R1} / f_{ctm} = 0.945 / 3.2 = 0.295;$$

NOTE * η_f is intentionally taken to be 0.7 although SS 674:2021 allows a value of 1.0.

Also, $\alpha = E_s / E_{c,eff} = 200 / 14.5 = 13.8$; $\rho = A_s / bd = 1256 / (150 \times 1414) = 0.00592$.

Fig. E4.1-2 shows the strain and stress diagrams assumed for the section under elastic cracked condition. Following the procedure given in Appendix B, Design Guide (2022), the following values are obtained:

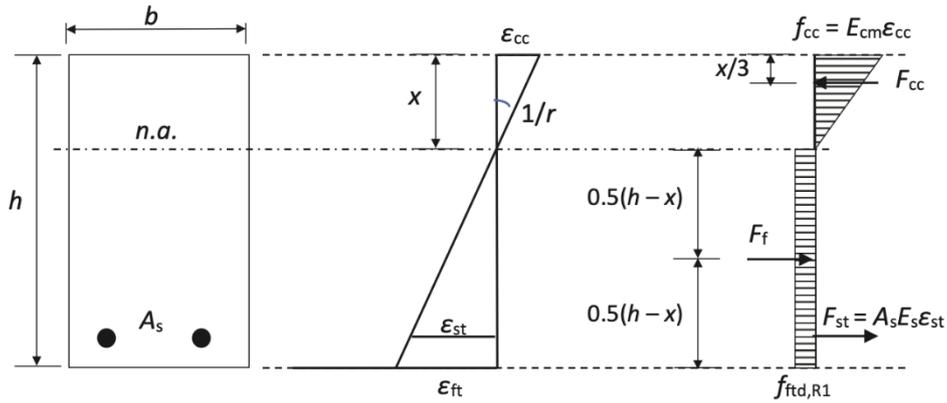


Fig. E4.1-2: Stress and strain diagrams assumed for the section under elastic cracked condition

Section curvature*, $1/r = 1.08 \times 10^{-6}/\text{mm}$; $x = 557.7 \text{ mm}$;

*NOTE Alternatively, this can be obtained from Charts B, Appendix B of this Guide as follows:

$$d/h = 1414/1500 \approx 0.95; \alpha\rho = 13.8 \times 0.00592 = 0.082 \times 10^{-3};$$

$$M_{ED}/(bd^2E_c) = 2650 \times 10^6 / (1000 \times 1414^2 \times 14.5 \times 10^3) = \mathbf{0.091 \times 10^{-3}};$$

$$f_{ftd,R1}/E_c = 0.945/(14.5 \times 10^3) = \mathbf{0.065 \times 10^{-3}}; \text{ from charts:}$$

Chart	d/h	$\alpha\rho$	$(1/r) d (\times 10^{-3})$	x/d
B-4(b)	0.95	0.05	2.1	0.33
B-4(c)	0.95	0.10	1.3	0.43
Interpolate between B-4(b) & B-4(c)	0.95	0.082	1.59	0.394

Hence, $(1/r) = 1.59 \times 10^{-3} / 1414 = 1.1 \times 10^{-6}$ and $x = 0.394 \times 1414 = 557.1 \text{ mm}$.

Therefore,

$$\epsilon_{cc} = (1/r) x = 1.08 \times 10^{-6} \times 557.7 \text{ mm/mm} = 602 \times 10^{-6} \text{ mm/mm};$$

$$f_{cc} = E_{c,eff} \epsilon_{cc} = 14.5 \times 10^3 \times 602 \times 10^{-6} = 8.74 \text{ MPa};$$

$$\epsilon_{ft} = \epsilon_{cc} (h/x - 1) = 602 \times (1500/557.7 - 1) \times 10^{-6} = 0.00102 \text{ mm/mm};$$

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

$$\epsilon_{ftu} = f_{ctd} / E_{cm} + 2.5/(0.8h) = 1.47/34100 + 2.5/(0.8 \times 1500) = 0.00213 \text{ mm/mm}$$

$$> \epsilon_{ft} = 0.00102 \text{ mm/mm} \quad \dots \text{ ok};$$

$$\epsilon_{st} = \epsilon_{cc} (d/x - 1) = 602 \times (1414/557.7 - 1) \times 10^{-6} = 0.000924 \text{ mm/mm}$$

$$< \epsilon_{st} = (500/1.15)/(200 \times 10^3) = 0.002173 \text{ mm/mm} \quad \dots \text{ ok};$$

$$\sigma_s = E_s \epsilon_{st} = 200 \times 10^3 \times 0.000924 = 185 \text{ MPa}$$

Check for equilibrium:

$$\begin{aligned}
 F_{cc} &= \frac{1}{2} b x f_{cc} = \frac{1}{2} \times 1000 \times 557.7 \times 8.74 \times 10^{-3} = 2437 \text{ kN}; \\
 F_f &= b(h-x) f_{ftd,R1} = 1000 \times (1500 - 557.7) \times 0.945 \times 10^{-3} = 890 \text{ kN}; \\
 F_{st} &= A_s \sigma_s = 1256 \times (1000/150) \times 185 \times 10^{-3} = 1549 \text{ kN};
 \end{aligned}$$

$$\text{Therefore, } F_{cc} - F_f - F_{st} = 2437 - 890 - 1549 \approx 0 \text{ kN}$$

Check value of moment:

$$\begin{aligned}
 M_{Rd} &= F_{cc} (2x/3) + F_f (1/2)(h-x) + F_{st} (d-x) \\
 &= [2437 \times 2 \times 557.7/3 + 890 \times 1/2 \times (1500 - 557.7) + 1549 \times (1414 - 557.7)] \times 10^{-3} \\
 &= 906 + 419 + 1326 = 2651 \text{ kNm/m} \approx M_{Ed} = 2650 \text{ kNm/m} \quad \dots \text{ ok}
 \end{aligned}$$

Crack Width Calculation. – SS 674:2021 Eq. (7.6) gives:

$$\begin{aligned}
 f_{ct,eff} &= f_{ctm} = 3.2 \text{ MPa}; \\
 h_{eff} &= \min\{2.5(h-d), (h-x)/3, h/2\} \\
 &= \min\{2.5(1500 - 1414), (1500 - 557.7)/3, 1500/2\} \\
 &= \min\{215, 314, 750\} = 215 \text{ mm}; \\
 \rho_{p,eff} &= A_s / b h_{eff} = 1256 / (150 \times 215) = 0.039; \\
 \alpha_e &= E_s / E_{cm} = 200 / 34.1 = 5.87.
 \end{aligned}$$

Therefore, considering that $k_t = 0.4$ for long-term crack width,

$$\begin{aligned}
 (\epsilon_{sm} - \epsilon_{cm}) &= [\sigma_s - (k_t + (1 - k_t) k_f) (f_{ct,eff} / \rho_{p,eff}) (1 + \alpha_e \rho_{p,eff})] / E_s \\
 &= [185 - (0.4 + 0.6 \times 0.295) \times (3.2 / 0.039) (1 + 5.87 \times 0.039)] / (200 \times 10^3) \\
 &= (185 - 58) / (200 \times 10^3) = 635 \times 10^{-6} \\
 &> 0.6 \sigma_s / E_s = 0.6 \times 185 / (200 \times 10^3) = 555 \times 10^{-6} \quad \dots \text{ok}
 \end{aligned}$$

Since bar spacing $s = 150 \text{ mm} < 5(c + \phi/2) = 5 \times (50 + 16 + 40/2) = 430 \text{ mm}$, and with $k_3 = 3.4$, $k_1 = 0.8$ (high bond bars), $k_2 = 0.5$ (bending), $k_4 = 0.425$ [as per NA to SS EN1992-1-1], SS 674:2021 Eq. (7.8) gives:

$$\begin{aligned}
 S_{r,max} &= k_3 \cdot c + k_1 \cdot k_2 \cdot k_4 \cdot (1 - k_f) \cdot \phi / \rho_{p,eff} \\
 &= 3.4c + 0.8 \times 0.5 \times 0.425 \times (1 - 0.295) \times \phi / \rho_{p,eff} \\
 &= 3.4 \times (50 + 16) + 0.8 \times 0.5 \times 0.425 \times (1 - 0.295) \times 40 / 0.039 = 347 \text{ mm}
 \end{aligned}$$

Therefore, from SS 674:2021 Eq. (7.5):

$$\begin{aligned}
 w_k &= S_{r,max} (\epsilon_{sm} - \epsilon_{cm}) \\
 &= 347 \times 635 \times 10^{-6} \\
 &= 0.22 \text{ mm} < w_{k,limit} = 0.25 \text{ mm}
 \end{aligned}$$

Alternatively, the maximum crack width w_k can be obtained using Chart F-1 (Appendix F) as follows (refer to Fig. E4.1-3). Using $\phi = 40$ mm, $f_{ct,eff} = 3.2$ MPa, $k_f = 0.295$, $\alpha_e = 5.87$, $\rho_{p,eff} = 0.039$ gives:

$$K_{f1} = 0.17 \cdot (1 - k_f) = 0.120; \quad K_{f2} = 0.4 + 0.6 \cdot k_f = 0.577;$$

$$K_{f1} \cdot \frac{\phi}{\rho_{p,eff}} = 0.120 \cdot \frac{40}{0.039} = 123 \text{ mm}$$

$$K_{f2} \cdot \frac{f_{ct,eff}}{\rho_{p,eff}} \cdot (1 + \alpha_e \cdot \rho_{p,eff}) = 0.577 \cdot \frac{3.2}{0.039} \cdot (1 + 5.87 \times 0.039) = 58 \text{ MPa}$$

$$s^* = \frac{K_{f1}/K_{f2}}{f_{ct,eff} \cdot (1 + \alpha_e \cdot \rho_{p,eff})} \cdot \phi = \frac{123}{58} = 2.1 \text{ mm/MPa}$$

From Point A in Q3, project horizontal and vertical lines to Points B in Q4 and C in Q2, corresponding to $k_{3C} = 3.4 \times 40 = 224$ mm and $\sigma_s = 185$ MPa, respectively. Finally, project vertical line from Point B and horizontal line from point C to meet at Point D in Q1, which gives $w_k = 0.22$ mm.

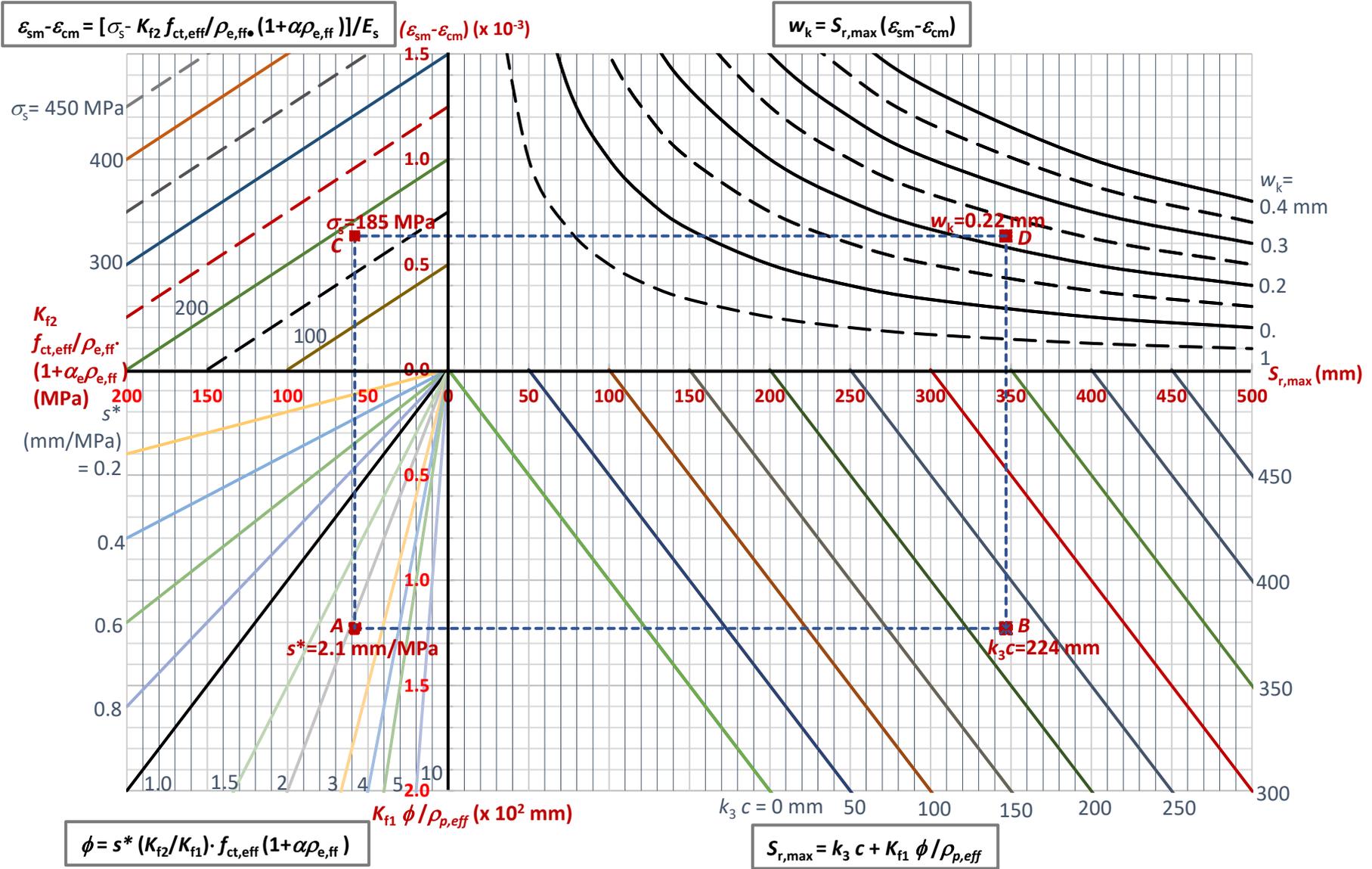


Fig. E4.1-3 Determination of Maximum Crack Width using Chart F-1

Example 4.2 – Crack Control in Basement Slab

The basement slab shown in Fig. E4.2-1 is designed with three layers of H40-150 bars at a vertical spacing between layers of 100 mm. The slab has an overall thickness, $h = 2000$ mm, and the effective depth to the centroid of the tension bars is $d = 1800$ mm. The mid-span moments at ULS and SLS are respectively, $M_{ULS} = 11363$ kNm/m and $M_{SLS} = 7899$ kNm/m.

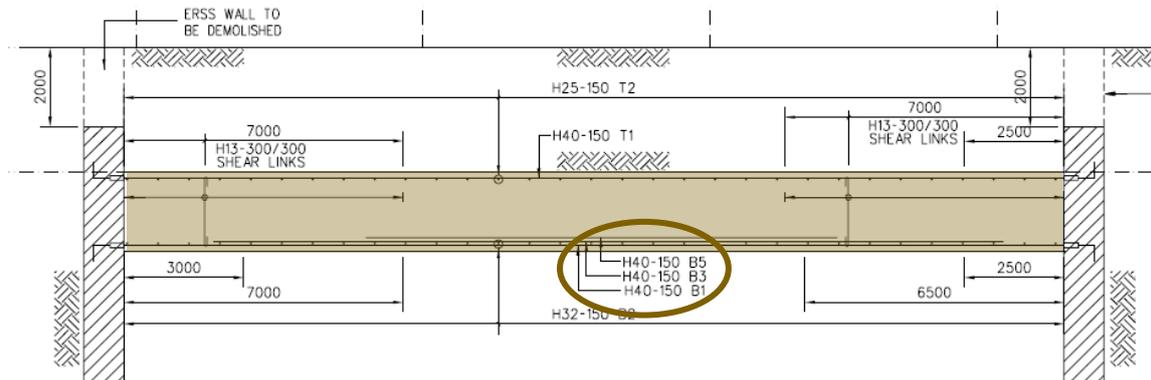


Fig. E4.2-1 Basement Slab Built In-Between Two Walls

The concrete strength class is C32/40. The tension steel ratio is $\rho = 1.4\%$. It is governed by crack width limitation, being $w_k = 0.35$ mm at the bottom face of the slab and 0.3 mm at 40 mm from the face. It is desired to reduce the steel ratio ρ to less than 1.0%, i.e., 2 layers of H40-150 without compromising on flexural strength using steel fibre concrete (Fig. E4.2-2).

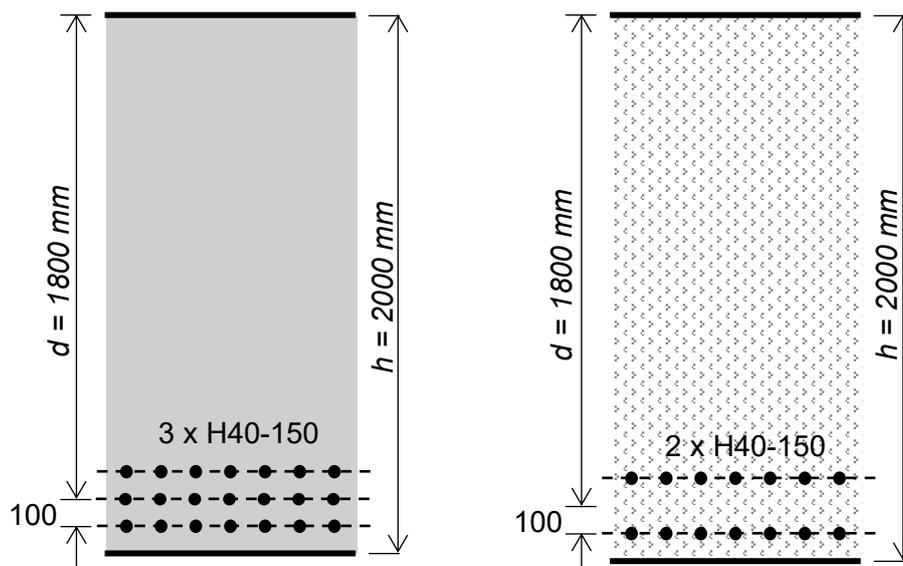


Fig. E4.2-2 Original Section (Left) versus SFRC Section (Right)

At flexural ULS, $M_{Ed}/bd^2f_{ck} = 11363 \times 10^6 / (1000 \times 1800^2 \times 32) = 0.110$. With $d/h = 1800/2000 = 0.9$, Design Chart C-3, Appendix C (see Fig. E4.2-3) gives the required tension steel area for a section without fibres as $A_s = 0.124 bdf_{ck}/f_{yd} = 0.124 \times 1000 \times 1800 \times 32 / (500/1.15) = 16428 \text{ mm}^2/\text{m}$, which can be provided by two layers of H40-150 bars ($A_s = 16747 \text{ mm}^2/\text{m}$; $A_s f_{yd}/bdf_{ck} = 0.126$).

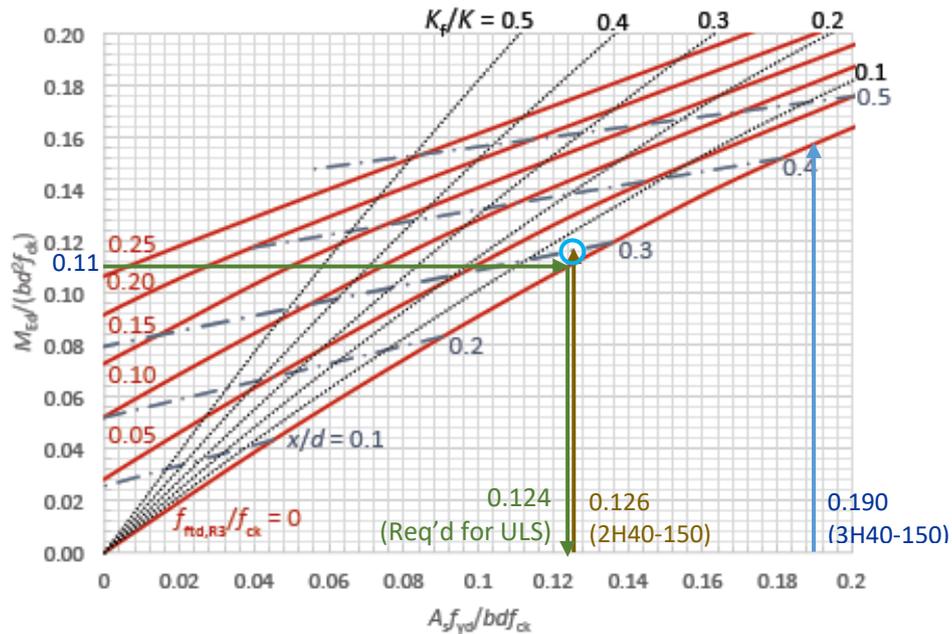


Fig. E4.2-3 Design Chart C-3 ($d/h = 0.9$)

Consider fibre concrete section with strength class C32/40-R₁₃/R₃₃, and assuming the creep coefficient is $\varphi = 1.7$. Then,

$$f_{ck} = 32 \text{ MPa}; E_{cm} = 33.3 \text{ GPa}; f_{ct,eff} = f_{ctm} = 3.0 \text{ MPa};$$

$$f_{ftd,R1} = 0.45 \times 3 / 1 = 1.35 \text{ MPa}; k_f = f_{ftd,R1} / f_{ctm} = 0.45;$$

$$E_{c,eff} = E_{cm} / (1 + \varphi) = 12.4 \text{ GPa};$$

$$\alpha = E_s / E_{c,eff} = 16.2$$

Crack width calculation using fictitious steel stress (**Conservative Approach**):

$$\rho = A_s/bd = 16747 / (1000 \times 1800) = 0.00930; \alpha\rho = 16.2 \times 0.00930 = 0.151$$

Neutral axis depth & moment of inertia:

$$x/d = 0.419; x = 754 \text{ mm};$$

$$I = 440 \times 10^9 \text{ mm}^4/\text{m}$$

Under $M_s = 7899$ kNm/m, fictitious steel stress for the outermost steel layer ($d_{out} = 1900$ mm) is:

$$\sigma_{s,fict} = \alpha M_s (d_{out} - x) / I = 16.2 \times 7899 \times 10^6 \times (1900 - 754) / (440 \times 10^9) = 333 \text{ MPa}$$

Alternatively,

$$\begin{aligned} \sigma_{s,fict} &= (M_s / M_u) f_{yd} (d_{out} - x) / (d - x) = (7899 / 11363) \times (500 / 1.15) \times (1900 - 754) / (1800 - 754) \\ &= 331 \text{ MPa} \end{aligned}$$

With $f_{ct,eff} = 3.0$ MPa;

$$h_{c,ef} = \min. (2.5(h - d), (h - x)/3, h/2) = \min. (500, 415, 1000) = 415 \text{ mm},$$

but this is limited to $(c + \phi/2 + s_b/2) = 80 + 40/2 + 200/2 = 200$ mm (PD 6678-1:2020), where s_b is the vertical spacing between layers of bars. Hence,

$$\rho_{p,eff} = \frac{1}{2} \times 16747 / (1000 \times 200) = 0.0419, \text{ and } k_t = 0.4,$$

$$\begin{aligned} (\varepsilon_{sm} - \varepsilon_{cm}) &= (1 - k_f) [\sigma_{s,fict} - k_t (f_{ct,eff} / \rho_{p,eff}) (1 + \alpha_e \rho_{p,eff})] / E_s \\ &= (1 - 0.45) [333 - 0.4(3.0/0.0419) (1 + (200/33.3) \times 0.0419)] / 200,000 \\ &= 817 \times 10^{-6} > 0.6(1 - k_f) \sigma_s / E_s = 0.6(1 - 0.45) \times 333 / 200,000 = 549 \times 10^{-6} \end{aligned}$$

Since bar spacing, $s = 150$ mm $< 5(c + \phi/2) = 5 \times (80 + 40/2) = 500$ mm, therefore, with $k_3 = 3.4$, $k_4 = 0.425$, $k_1 = 0.8$ (high bond bars), $k_2 = 0.5$ (bending),

$$\begin{aligned} S_{r,max} &= 3.4c + 0.425 \times 0.8 \times 0.5 \times (1 - k_f) \phi / \rho_{p,eff} \\ &= 3.4 \times 80 + 0.425 \times 0.8 \times 0.5 \times (1 - 0.45) \times 40 / 0.0419 = 361 \text{ mm}. \end{aligned}$$

Therefore,

$$w_k = S_{r,max} (\varepsilon_{sm} - \varepsilon_{cm}) = 361 \times 817 \times 10^{-6} = 0.29 \text{ mm} < 0.35 \text{ mm}$$

2.5 Precast Tunnel Segmental Linings

Designing fibre-reinforced concrete (FRC) tunnel linings involves specialized engineering to ensure structural safety, durability, and performance under various load conditions.

1. *Key Parameters for Tunnel Lining Design*

- Geological conditions: Type of soil or rock surrounding the tunnel, groundwater conditions, and potential for movement or settlement.
- Loads on the lining: Includes earth pressure, hydrostatic pressure from groundwater, and potential seismic loads.
- Tunnel dimensions: Diameter, length, and shape of the tunnel.
- Service life and durability: The desired lifespan of the tunnel and environmental exposure conditions (e.g., chemical exposure, temperature variations).

2. *Loads and Stresses*

- Static loads: These include earth pressure due to the overburden and groundwater pressure. The self-weight of the tunnel lining is also considered.
- Dynamic loads: Earthquakes or other ground movements, dynamic loads from trains or other vehicles.
- Long-term creep and shrinkage.

3. *Structural Design Approaches*

- Empirical methods: These rely on experience and past performance of similar tunnels and can be used for simpler designs where geotechnical conditions are well understood.
- Finite element analysis (FEA): For more complex or critical tunnels, FEA software is used to simulate the interaction between the tunnel lining and the surrounding ground. This method allows for precise modeling of the effects of loads, fibre reinforcement, and material properties.

4. *Design Criteria*

- Ultimate load-bearing capacity: includes consideration of both the short-term and long-term loads.

- Serviceability limit states: limiting crack widths, preventing excessive deformations, and ensuring durability.

5. Lining Thickness and Fibre Dosage

- *Governing factors*: Geotechnical conditions (weak soils or high groundwater pressure may require thicker linings and higher fibre content); Load conditions; Crack control requirements.
- Typical steel fibre dosages for tunnel linings range from 25 to 60 kg/m³. A **minimal residual strength and ductility class of 4c**, corresponding to R_{14}/R_{34} should be used.

6. Reinforcement

- Rebar + fibres: Steel fibers are used in combination with rebar to provide additional reinforcement. The rebar provides flexural strength, while the fibers enhance crack control and toughness.
- Fully fibre-reinforced: Steel fibres alone are used to reinforce the lining.

7. Durability Considerations

- Durability is a key concern for tunnel linings, especially in environments with high moisture, chemicals, or temperature variations:
- Corrosion resistance: Fibres improve the durability of the tunnel lining by reducing crack widths, thus limiting the ingress of water and harmful chemicals.
- Chemical resistance: In some cases, synthetic fibres are used to improve resistance to chemicals in harsh environments.

References

- AITES/ITA: Guidelines for the Design of Segmental Tunnel Linings. ITA Working Group 2 – Research, ITA Report No. 22, April 2019.
- ACI 544.4R-18: Guide for Design with Fiber-Reinforced Concrete. American Concrete Institute, USA.
- ACI 544.7R-16: Report on Design and Construction of Fiber-Reinforced Precast Concrete Tunnel Segments. American Concrete Institute, USA.
- fib* Bulletin 83: Precast tunnel segments in fibre-reinforced concrete. State-of-the-art report, October 2017, 162 pp.

Segment Lining Analysis:

Analysis carried out for the tunnel under load combinations specified in LTA Civil Design Criteria (LTA 2019) gives the maximum and minimum axial loads (N_a and N_c respectively), as well as maximum bending moments (M_{max}) as shown in Table E5.1-1.

Table E5.1-1 Design Moments and Axial Forces

Load Cases*	N_a (kN/m)	N_c (kN/m)	M_{max} (kNm/m)	N_a/bhf_{ck}	N_c/bhf_{ck}	M_{max}/bh^2f_{ck}
LC1	3531	2655	225	0.235	0.177	0.050
LC1a	3166	2447	186	0.211	0.163	0.041
LC2	3545	2763	202	0.236	0.184	0.045
LC2a	3181	2555	164	0.212	0.170	0.036
LC3	3186	2599	155	0.212	0.173	0.034
LC4	3502	2520	169	0.233	0.168	0.038
LC4a	3127	2219	156	0.208	0.148	0.035

* based on LTA Civil Design Criteria (LTA 2019).

N_a = axial force at mid-axis of tunnel; N_c = axial force at crown of tunnel;

M_{max} = maximum bending moment; b = 1000 mm; h = 300 mm; f_{ck} = 50 MPa.

ULS Verifications

The N-M combinations are checked against the design N-M curves given in Appendix D, Chart D-5(c), with $\alpha_{cc,f} = 0.75$ and $f_{ftd,R3}/f_{ck} = 0.0232$, derived following SS 674:2021, Annex O, Section 3. The results are shown in Fig. E5.1-2, which indicates that the segments are adequate in resistance against bending with axial load.

The maximum shear force, $V_{Ed} = 129$ kN/m. For SFRC elements without traditional bar reinforcement, the shear capacity of steel fibre concrete section may be taken, assuming $d = 0.75h$, and ignoring the beneficial effect of axial force, as:

$$V_{Rd,cf} = V_{Rd,cf(min)} \cdot bd = (0.035k^{3/2}f_{ck}^{1/2}) \cdot b(0.75h)$$

where

$$k = 1 + \sqrt{(200/d)} = 1 + \sqrt{(200/(0.75 \times 300))} = 1.94 > 2 \text{ Hence,}$$

$$V_{Rd,cf} = (0.035 \times 1.94^{3/2} \times 50^{1/2}) \cdot (0.75 \times 300)$$

$$= 0.67 \times 225 = 150 \text{ kN/m} > V_{Ed} = 129 \text{ kN/m} \dots \text{ok}$$

Alternatively, the shear capacity of SFRC may be taken conservatively, in view of the provisions of the 2nd Generation EC2, as:

$$V_{Rd,cf} = \tau_{Rd,cf} \cdot b_w d = \frac{0.37 f_{R,3}}{\gamma_f} \cdot b_w d = \frac{0.37 \cdot 4.7}{1.5} \cdot 1000 \cdot 0.75 \cdot 300 \cdot 10^{-3}$$

$$= 261 \text{ kN/m} > V_{Ed}$$

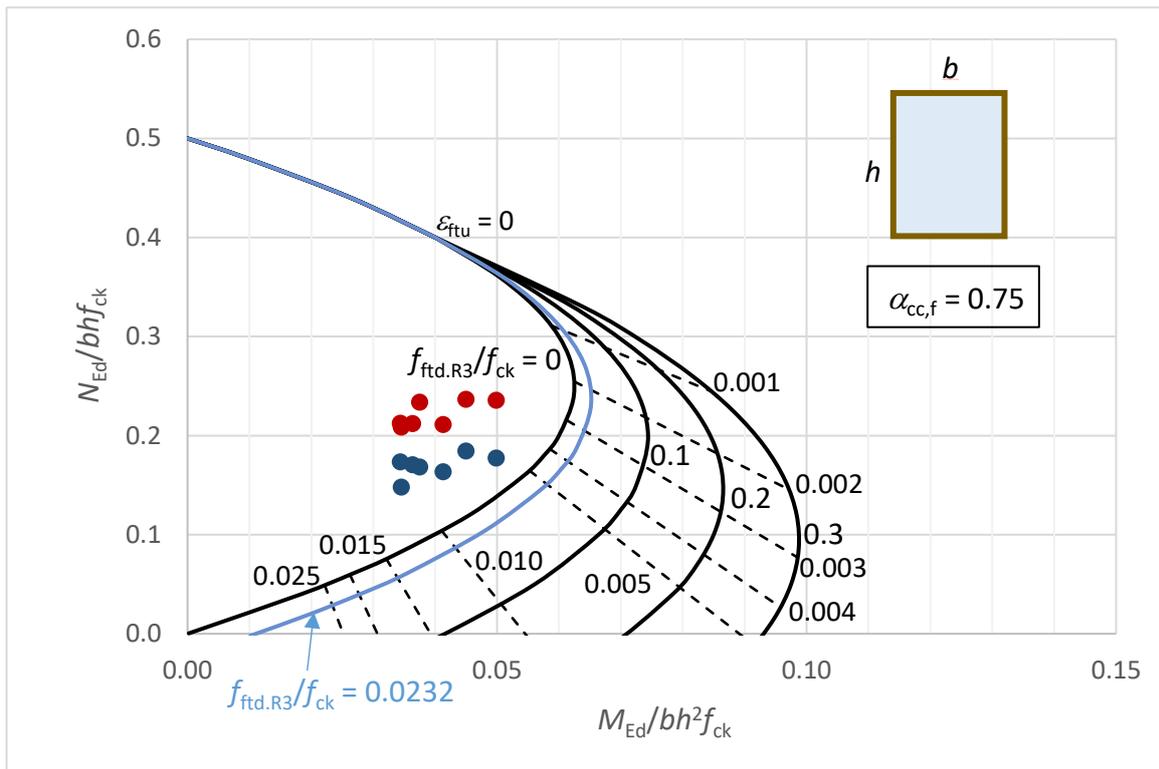


Fig. E5.1-2 Verification for ULS in bending with axial force

Example 5.2 – Precast Macro-Synthetic FRC Tunnel Segments

This example deals with the design of precast macro-synthetic FRC tunnel segments for a utility tunnel, including the design for production, storage and transportation of segments. The tunnel ring has an internal diameter D_i of 3700 mm with 6 segments per ring.

Segment Dimensions

- Length, $L_i = 2.068$ m; width, $b = 1200$ mm; thickness, $h = 250$ mm; aspect ratio, $L_i/h = 8.4$.

Material Properties

- Concrete strength class C50/60-R₁₂/R₃₂;
- Fibre Class II (EN14889-2); fibre dosage = 6 kg/m³;
- Characteristic residual flexural strengths, $f_{R,1} = 2.21$ MPa; $f_{R,3} = 2.94$ MPa (Class 2e).
- Characteristic residual tensile strengths, $f_{ft,R1} = 0.45 f_{R,1} = 0.99$ MPa; $f_{ft,R3} = 0.37 f_{R,3} = 1.09$ MPa [SS 674, Eqs. (3.1) & (3.2)]

Design Residual Tensile Strengths and Minimum Ductility Requirement

- Width, $b = 1200$ mm $\approx 5 \times$ thickness, $h = 5 \times 250 = 1250$ mm; therefore, fibre orientation factor, η_f may be taken as 1.0.
- Magnification/Redundancy factor, $\eta_{det} = 1.0$.
- Design residual tensile strengths,

$$f_{ftd,R1} = \eta_f \eta_{det} (f_{ft,R1}/1.5) = 0.66 \text{ MPa} \quad [\text{SS 674, Eq. (3.3)}]$$

$$f_{ftd,R3} = \eta_f \eta_{det} (f_{ft,R3}/1.5) = 0.73 \text{ MPa} \quad [\text{SS 674, Eq. (3.3)}]$$

- Check for minimum ductility [SS 674, Cl. 5.6.2]

$$f_{fctk,0.05} = 0.21 f_{ck}^{2/3} = 0.21 \times (50)^{2/3} = 2.85 \text{ MPa}$$

$$f_{R,1} / f_{fctk,0.05} = 2.21/2.85 = 0.78 > 0.75$$

$$f_{R,3} / f_{fctk,0.05} = 2.94/2.85 = 1.03 > 0.65$$

Sectional Strain Analysis

- Characteristic length, $l_{cs} = h = 250$ mm [SS 674, Eq. (3.5)]
- Ultimate crack opening, $w_u = 2.5$ mm
- $\varepsilon_{ct} = f_{fctd} / E_{cm} = (2.85/1.5)/37,000 = 0.000051$
- Ultimate tensile strain in MS FRC, $\varepsilon_{ftu} = \varepsilon_{ct} + w_u / l_{cs} = 0.01005$

Considering the idealized parabolic-rectangular stress-strain relation for concrete in compression, with $\varepsilon_{c2} = 0.002$ and $\varepsilon_{cu2} = 0.0035$, and taking the extreme tensile fibre strain, $\varepsilon_{ft} = \varepsilon_{ftu} = 0.01005$ (Fig. E5.2-1), the following solution is obtained:

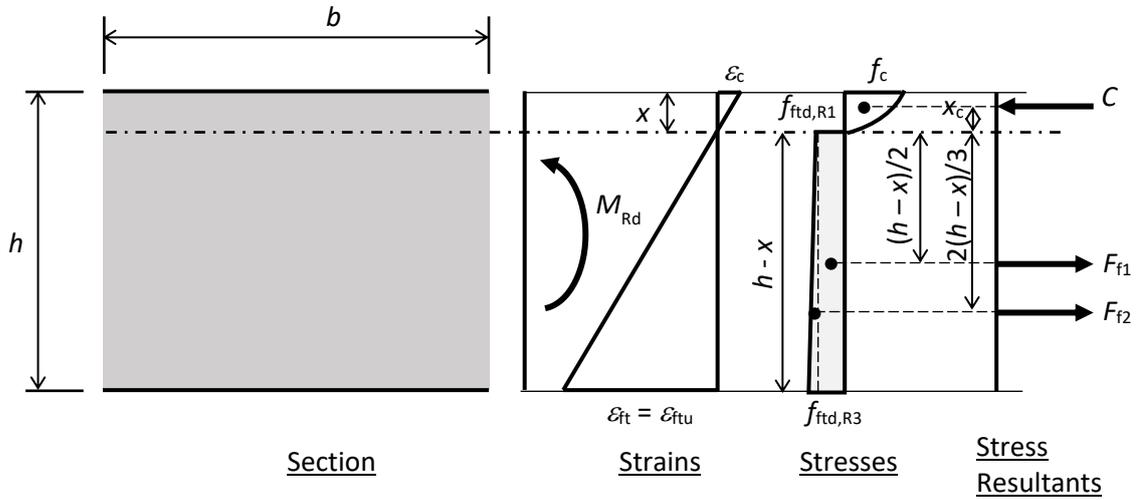


Fig. E5.2-1 Section Analysis for Flexure at ULS

- $x = 17.4 \text{ mm}$; $\varepsilon_c = 0.00075 < \varepsilon_{c2} = 0.002$; $\varepsilon_{ft} = 0.01005$ (as assumed)
- Force in concrete compression zone, $C = 194.0 \text{ kN}$; acting at $x_c = 11.4 \text{ mm}$ from neutral axis.
- Forces in tension, $F_{f1} = 184.2 \text{ kN}$; $F_{f2} = 9.8 \text{ kN}$; $F_f = F_{f1} + F_{f2} = 194 \text{ kN} = C$

Moment capacity of section,

- $M_{Rd} = [194.0 \times 11.4 + 184.2 \times (250 - 17.4) / 2 + 9.8 \times (250 - 17.4) \times 2 / 3] \times 10^{-3} = 25.2 \text{ kNm}$

Alternatively, M_{Rd} may be conservatively obtained from Chart C-1 using $f_{ftd,R3} / f_{ck} = 0.66 / 50 = 0.0132$ as $M_{Rd} = 0.01b(0.8h)^2 f_{ck} = 0.0095 \times 1200 \times (0.8 \times 250)^2 \times 50 \times 10^{-6} = 24.0 \text{ kNm}$.

Design for Production and Transient Stages

The following verifications ensure that the segment maintains structural integrity during production, storage and transportation. The self-weight of each segment is: $b \times h \times L_i \times$ unit weight of structural concrete $= 1.2 \times 0.25 \times 2.068 \times 25 = 15.5 \text{ kN}$. Since the chord length of segment is 1.975 m, therefore, self-weight of segment is $g_k = 15.5 / 1.975 = 7.85 \text{ kN/m}$.

For the load cases of segment production, handling and storage, the post-cracking properties of MSFRC is assumed to be 50% of the values observed at 28 days, that is:

$$\text{Moment of resistance, } M_{Rd} = 0.5 \times 25.2 = 12.6 \text{ kNm}$$

The design moments for segment demoulding, storage, transportation and handling are summarized in Table E5.2-1 and Figs. E5.2-2 and E5.2-3.

Table E5.2-1 Design Moments for Production and Transient Stages

Load Case		Design Formula for M_{Ed}	Design Moment (kNm)	Remarks
1 – Segment Demoulding	Lifting at $\frac{1}{4}$ - points	$1.35g_k a^2/2$	1.3	Assume $a = 1.975/4 = 0.494$ m.
2 – Segment Handling	(a) Extrados up (Fig. E5.2-2a)	$1.35g_k l^2/8$	4.5	$l = 1.85$ m
	(b) Intrados up (Fig. E5.2-2b)	$1.35g_k a^2/2$	5.8	$a = 1.05$ m
	(c) Two supports at $\frac{1}{4}$ - points (Fig. E5.2-2c)	$1.35g_k [l^2/8 - a^2/2]$ $1.35g_k a^2/2$	0.2 1.2	$l = 1.02$ m; $a = 0.478$ m
3 – Segment Storage	(a) Stacking with 2 supports at $\frac{1}{4}$ -points (Fig. E5.2-3a)	Bottom segment: $1.35g_k [l^2/8 - a^2/2] + 1.35F_1 e$ $1.35g_k a^2/2 + 1.35F_1 e$	10.6 11.7	$l = 1.02$ m; $a = 0.478$ m; $e = 0.1$ m; $F_1 = 77.5$ kN
	(b) Two supports at 0.65 m from centreline (Fig. E5.2-3a)	Bottom segment: $1.35g_k [l^2/8 - a^2/2] + 1.35F_1 e$ $1.35g_k a^2/2 + 1.35F_1 e$	12.1 11.1	$l = 1.3$ m; $a = 0.338$ m; $e = 0.1$ m; $F_1 = 77.5$ kN
4 – Segment Transportation	Stack of three segments with two supports at 0.8 m from centreline (Fig. E5.2-3b)	Bottom segment: $2(1.35g_k [l^2/8 - a^2/2] + 1.35F_2 e)$ $2(1.35g_k a^2/2 + 1.35F_2 e)$	19.0 12.9	$l = 1.6$ m; $a = 0.188$ m; $e = 0.1$ m; $F_2 = 46.5$ kN & dynamic shock factor = 2.

NOTE g_k = uniform load due to self-weight of segment, in kN/m;
 a = distance of lifting points from nearer end of segment, in m;
 l = distance between supports in m;
 F_1 = self-weight of all segments completing a ring, excluding bottom segment, in kN;
 F_2 = self-weight of all segments placed in one truck for transportation phase, in kN;
 e = eccentricity due to imperfect alignment of stack supports, in m.

For load cases 1, 2 and 3, the maximum design moment is 12.1 kNm (Case 3b), which is less than the early-age moment of resistance of 12.6 kNm. For load case 4, the maximum design moment is 19.0 kNm, which is less than the design moment of resistance of 25.2 kNm. Hence, the ULS is verified for the production and transient stages.

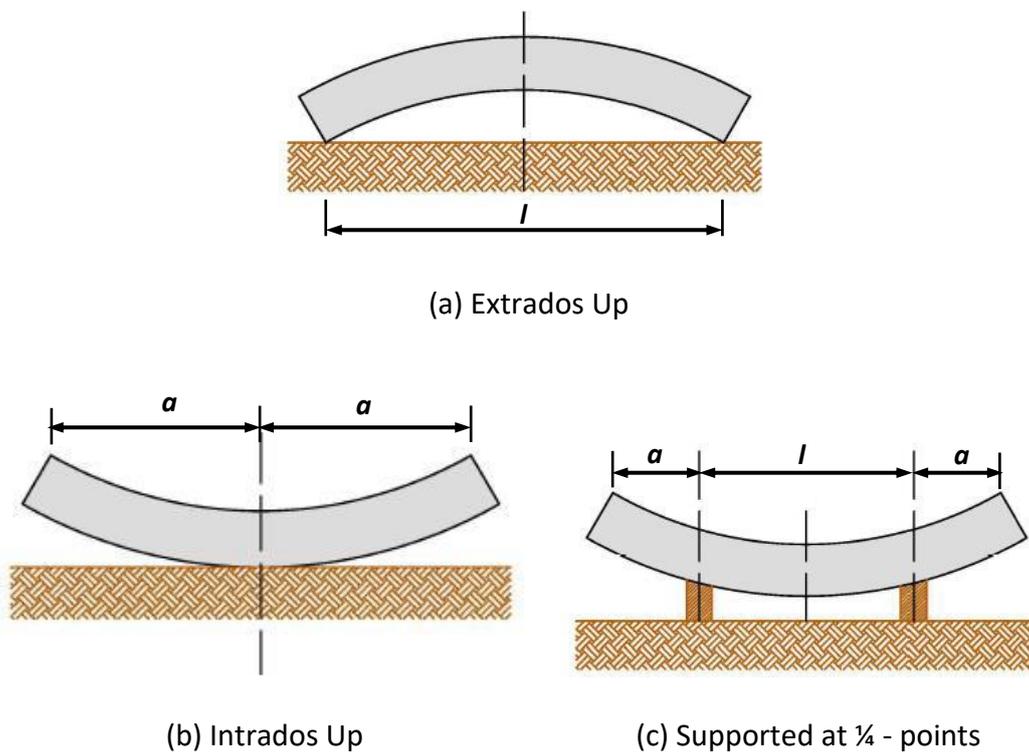


Fig. E5.2-2 Segment Handling

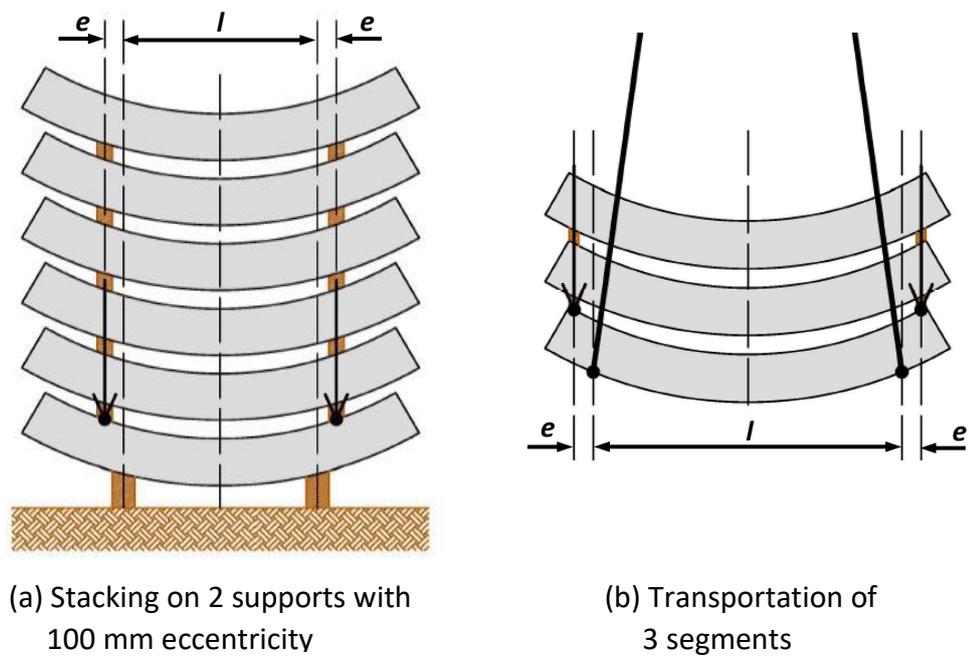


Fig. E5.2-3 Segment Stacking and Transportation

Design for Construction Stages

This is not dealt with herein. Readers may refer to specialist literature including ACI 544.7R-15 and *fib* Bulletin 83 for more information.

Design for Service Stages (Persistent Design Situation)

Ultimate Limit State. - Design moments M_{Ed} and axial forces N_{Ed} are obtained by analysing the tunnel structure for various load cases, including those involving ground embedding, and potential deformation due to construction quality. For this example, a single N-M combination with $N_{Ed} = 2153$ kN and $M_{Ed} = 163$ kNm is considered to demonstrate the calculation method.

Use the method of strain compatibility (Fig. E5.2-4), the neutral axis depth corresponding to $N_{Rd} = N_{Ed} = 2153$ kN, can be obtained by trial and error as $x = 84.16$ mm, giving $\varepsilon_{ft} = \varepsilon_{cu} (h - x)/x = 0.0035 \times (250 - 84.16)/84.16 = 0.00690 (< \varepsilon_{ftu} = 0.01005)$, and $f_{ft} = f_{ftd,R1} + (f_{ftd,R3} - f_{ftd,R1}) \varepsilon_{ft}/\varepsilon_{ftu} = 0.66 + (0.73 - 0.66) \times 0.00690/0.01005 = 0.708$ MPa.

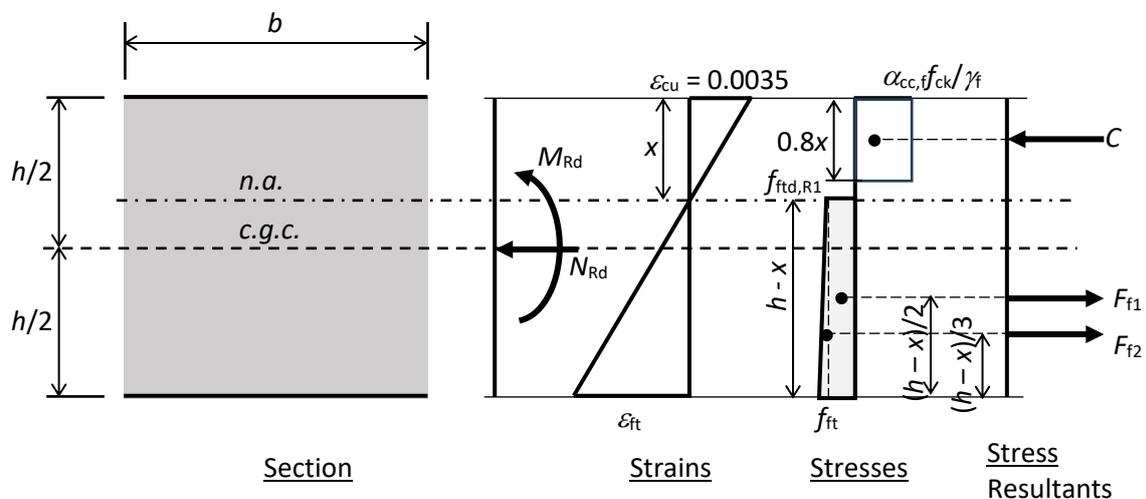


Fig. E5.2-4 Section Analysis for Flexure with Axial Force at ULS

The forces acting on the section are:

$$C = b (0.8x) \alpha_{cc} f_{ck} / \gamma_f = 1200 \times (0.8 \times 84.16) \times 0.85 \times 50 / 1.5 \times 10^{-3} = 2289.1 \text{ kN}$$

$$F_{f1} = b (h - x) f_{ftd,R1} = 1200 \times (250 - 84.16) \times 0.66 \times 10^{-3} = 131.3 \text{ kN}$$

$$F_{f2} = \frac{1}{2} b (h - x) (f_{ft} - f_{ftd,R1}) = \frac{1}{2} \times 1200 \times (250 - 84.16) \times (0.708 - 0.66) \times 10^{-3} = 4.8 \text{ kN}$$

$$\therefore N_{Rd} = C - F_{f2} - F_{f1} = 2289.1 - 131.3 - 4.8 = 2153 \text{ kN} = N_{Ed} \dots \text{ok}$$

The corresponding moment of resistance,

$$\begin{aligned} M_{Rd} &= C (h/2 - 0.4x) + F_{f2} [h/2 - (h-x)/2] + F_{f1} [h/2 - (h-x)/3] \\ &= 209.1 + 5.5 + 0.3 = 214.9 \text{ kNm} > M_{Ed} = 163 \text{ kNm}. \end{aligned}$$

Alternatively, it can be shown using Design Chart D-5(d), Appendix D, that the combination $(M_{Ed}/bh^2f_{ck}, N_{Ed}/bhf_{ck}) = (0.0432, 0.144)$ lies within the interaction curve with $f_{ftd.R3}/f_{ck} = f_{ftd.R1}/f_{ck} = 0.66/50 = 0.0132$, indicating the adequacy of the section. Also, the value of M_{Rd} can be obtained as $M_{Rd} = 0.057bh^2f_{ck} = 0.057 \times 1200 \times 250^2 \times 50 \times 10^{-6} = 214 \text{ kNm}$.

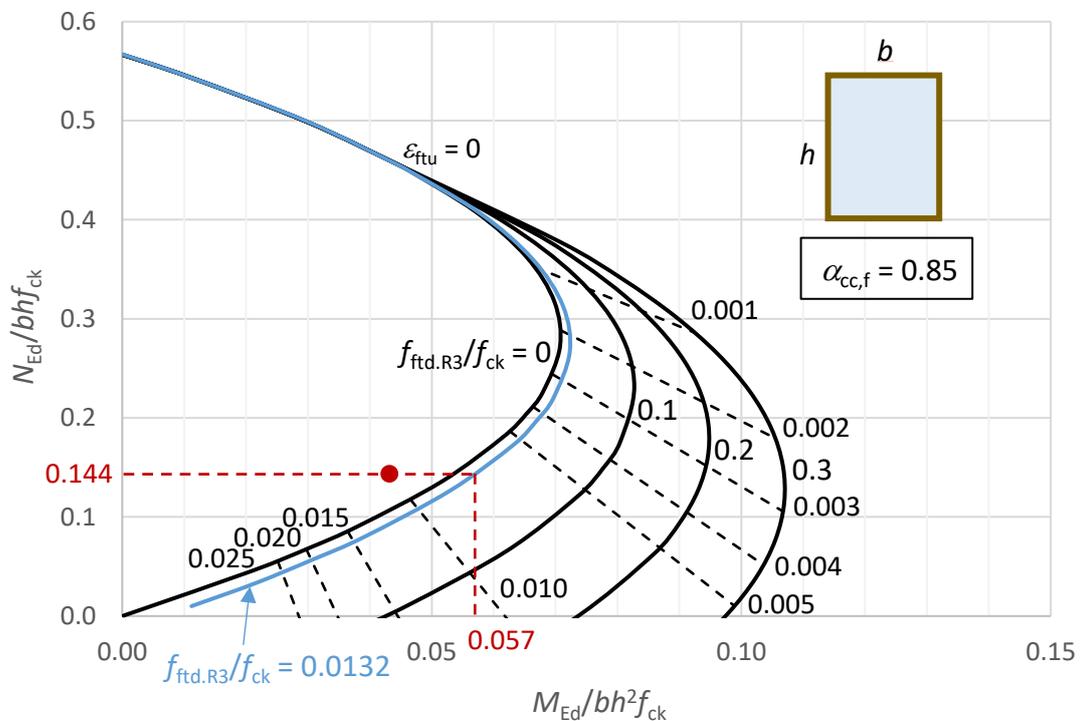


Fig. E5.2-5 Verification using Design Chart D-5(d)

Serviceability Limit State. – Consider $N_{Ed} = 1530 \text{ kN}$ and $M_{Ed} = 108 \text{ kNm}$ under quasi-permanent loads. Assuming a cracked elastic section with $f_{ftd,R1} = \eta_f f_{ft,R1} = 0.99 \text{ MPa}$, and following the general procedure given in Appendix B, the following results are obtained: neutral axis depth, $x = 100.5 \text{ mm}$; extreme concrete compressive fibre stress, $f_{cc} = 28.3 \text{ MPa} < 0.6f_{ck} = 30 \text{ MPa}$; and extreme tensile strain, $\varepsilon_{ft} = 0.00113$. Hence, the maximum surface crack width, $w_{max} = \varepsilon_{ft} \cdot 2 \cdot (h - x) = 0.00113 \times 2 \times (250 - 100.5) = 0.34 \text{ mm}$.

2.6 Pile Caps

Pile caps may be designed using strut-and-tie models or they may be designed for bending and shear by considering them as slabs (SS EN 1992-1-1:2008, 9.8(2); EN 1992-1-1:2023, 12.8(2)). In strut-and-tie models, fibres may be used to substitute up to 30% of the total required main longitudinal reinforcement in ties. Also, they may replace transverse reinforcement, hanger reinforcement where the pile cap supports the foundation beams at the sides, and skin reinforcement where required.

In using the bending theory for design, the contribution of the applied load on the pile cap to the design shear force may be multiplied by a factor $\beta = a_v/2d$, where a_v is the clear distance between the applied load and the support pile reaction, and d is the effective depth to the longitudinal reinforcement, and $0.5d \leq a_v \leq 2d$ (SS EN 1992-1-1:2008, 6.2.2(6)). This applies only if the longitudinal reinforcement is fully anchored. For $a_v \leq 0.5d$, the value $a_v = 0.5d$ is assumed. Also, the design shear force calculated without reduction should always be at most equal to $0.5bdvf_{cd}$, where b is the width of the pile cap, $\nu = 0.6(1 - f_{ck}/250)$ and $f_{cd} = 0.85f_{ck}/1.5$.

This example illustrates the design of a pile cap with 2 piles in SFRC using both the strut-and-tie model and the bending theory.

Example 6 – Pile Cap

The pile caps, designated PC1, in Example 3 (refer Fig. E3-1) are designed using the strut-and-tie method in this example. These pile caps are located at the intersection between Grid 1c with Grid N1 and M1. Loads are transferred on its top through a plinth with dimensions 600 mm x 600 mm. At the bottom, the cap is supported by 2 piles with a diameter of 300 mm. The chosen dimensions of the pile cap are length, $L = 2100\text{mm}$, width $b = 700\text{mm}$ and depth $h = 510\text{mm}$ (refer to Fig. E6-1).

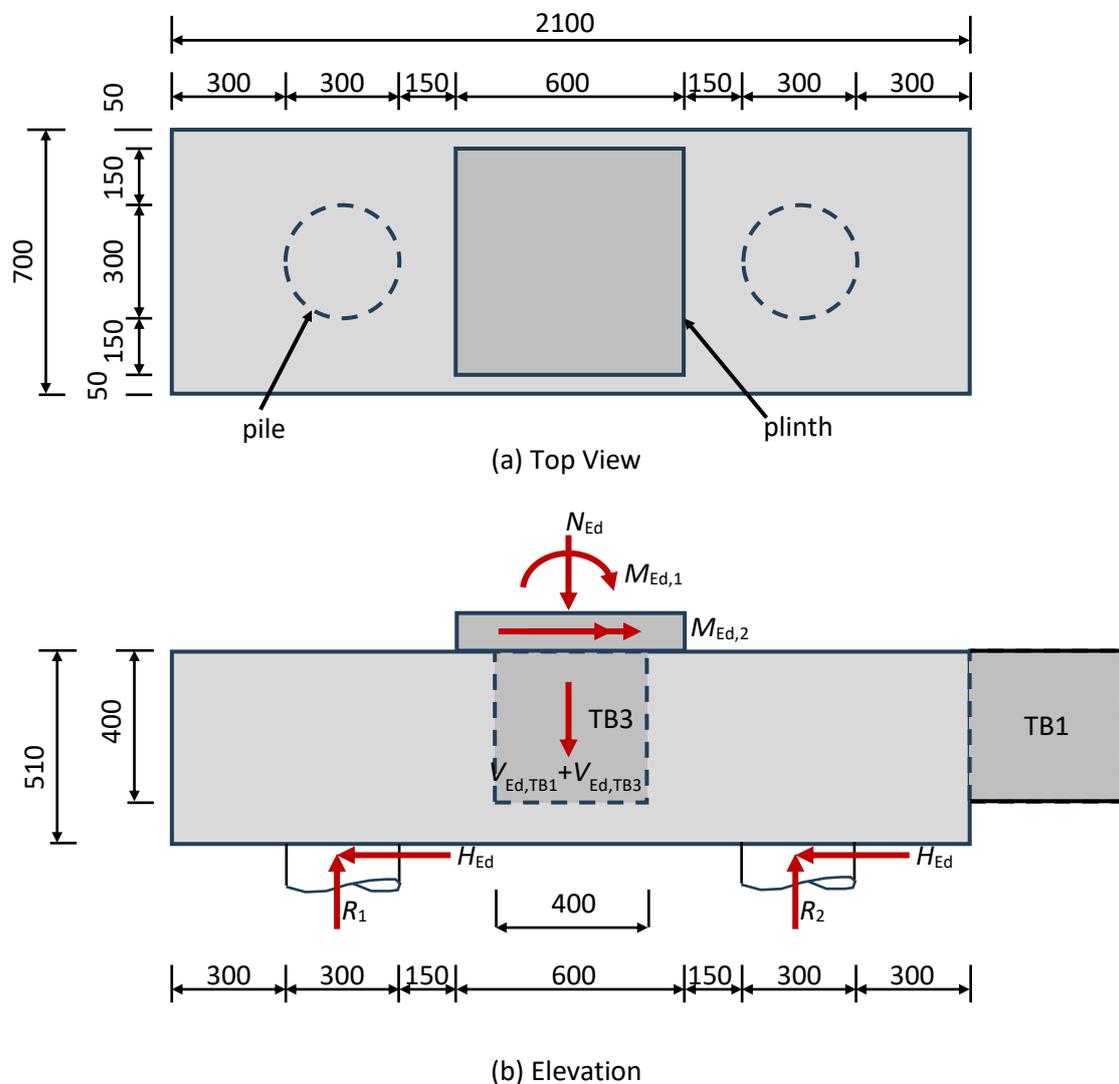


Fig. E6-1 Geometry and Loads on Pile Cap PC-1

The fibre concrete strength class is C40/50-R₁4R₃5 (Class 4d). The material properties are as follows:

$$\begin{aligned}
 f_{ck} &= 40 \text{ MPa}; & E_{cm} &= 33.3 \text{ GPa}; & f_{ctm} &= 3.02 \text{ MPa}; & f_{ctk} &= 2.12 \text{ MPa}; \\
 f_{R,1} &= 4 \text{ MPa}; & f_{R,3} &= 5 \text{ MPa}; & f_{ft,R1} &= 1.80 \text{ MPa}; & f_{ft,R3} &= 1.75 \text{ MPa}; \\
 \eta_f &= 0.5; & \eta_{det} &= 1.0; & f_{ftd,R1} &= 0.60 \text{ MPa}; & f_{ftd,R3} &= 0.62 \text{ MPa}; \\
 f_{yk} &= 500 \text{ MPa}; & E_s &= 200 \text{ GPa}
 \end{aligned}$$

The pile cap is subjected to the following action effects (maximum values) due to applied loads:

Normal compressive force, $N_{Ed} = 640 \text{ kN}$ (applied through the plinth);

Bending moments, $M_{Ed,1} = 38 \text{ kNm}$, $M_{Ed,1} = 210 \text{ kNm}$ (applied through the plinth);

Shear forces, $V_{Ed,TB1} = 143 \text{ kN}$ (applied through beam TB-1),

$V_{Ed,TB3} = 163 \text{ kN}$ (applied through beam TB-3);

Horizontal forces, $H_{Ed} = 32 \text{ kN}$ (due to wind and inclination of the structure on top of the foundation, applied through each of the supporting piles).

In the following analysis, the 2-pile cap is idealized by a strut-and-tie system (Fig. E6-2), with the following action effects based on the maximum plinth reactions:

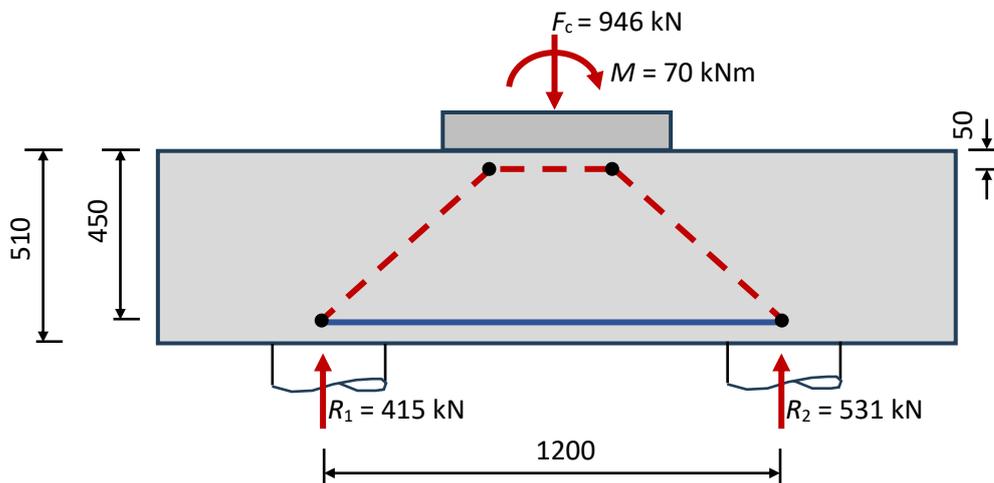


Fig. E6-2 Design Action Effects for 2-Pile Cap PC1

$$F_c = N_{Ed} + V_{Ed,TB1} + V_{Ed,TB3} = 640 + 143 + 163 = 946 \text{ kN};$$

$$M = M_{Ed,1} + 2 H_{Ed} \times 0.5 = 38 + 2 \times 32 \times 0.5 = 70 \text{ kNm};$$

which leads to pile reactions $R_1 = 946/2 - 70/1.2 = 415 \text{ kN}$ and $R_2 = 946 - 415 = 531 \text{ kN}$.

This indicates that the pile is not subjected to tension and no top reinforcement is required in the pile cap. In reality, additional loads such as self-weight of the pile cap, soil and paving materials above the pile and live load on the paving apply, and these are transmitted directly to the piles. Accounting for such loads, the maximum pile reaction is found to be 662 kN. Accordingly, as a conservative approach, the strut-and-tie model shown in Fig. E6-3 with two reaction forces each equal to 662 kN may be considered.

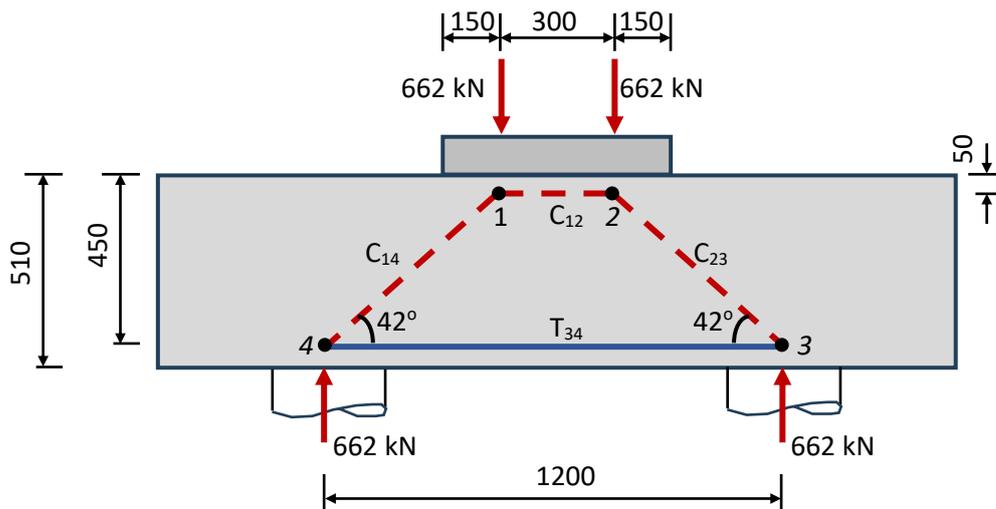


Fig. E6-3 Strut-and-Tie Model for Design of Pile Cap PC1

Design of Ties

From equilibrium of forces at nodes 3 or 4, the tensile force in Tie T_{34} is:

$$N_t = 662 \cot 42^\circ = 735 \text{ kN}$$

which requires a cross-sectional area $A_{s,req} = 735,000 / (500 / 1.15) = 1691 \text{ mm}^2$ (see NOTE). Assuming 5H20 bars with a design yield force of $5 \times 314 \times (500 / 1.15) \times 10^{-3} = 683 \text{ kN}$ are provided. This represents 92% of N_t , which is more than 70% (EN 1992-2023, L.8.5.2(2)). The remainder force of $(735 - 683) = 52 \text{ kN}$ may be carried by the fibres, that is:

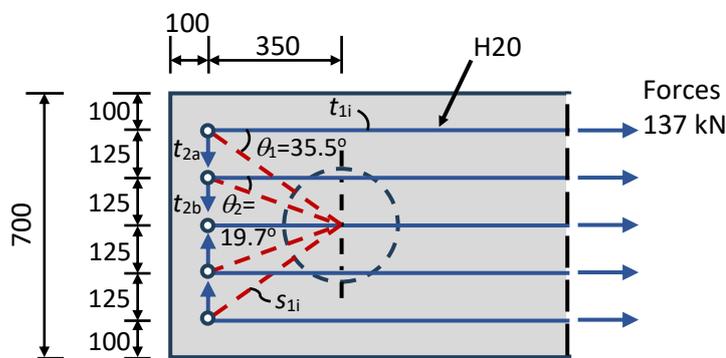
$$2b(h - d)f_{ftd,R3} = 2 \times 700 \times (510 - 450) \times 0.62 \times 10^{-3} = 52 \text{ kN}$$

NOTE: Based on beam theory, the required area would be $M/f_{yd}z = 662,000 \times 450 / [(500 / 1.15) \times (0.9 \times 450)] = 1655 \text{ mm}^2$ without steel fibres. With steel fibres, $M/bd^2f_{ck} = 662,000 \times 450 / (700 \times 450^2 \times 40) = 0.0525$, $f_{ftd,R3}/f_{ck} = 0.62/40 = 0.155$ and $d/h = 450/510 = 0.88$, the required steel bar area can be conservatively obtained from Chart C-2 ($d/h = 0.85$) as $A_s = 0.05bd^2f_{ck}/f_{yd} = 1449 \text{ mm}^2$, similar to the requirements using strut-and-tie method.

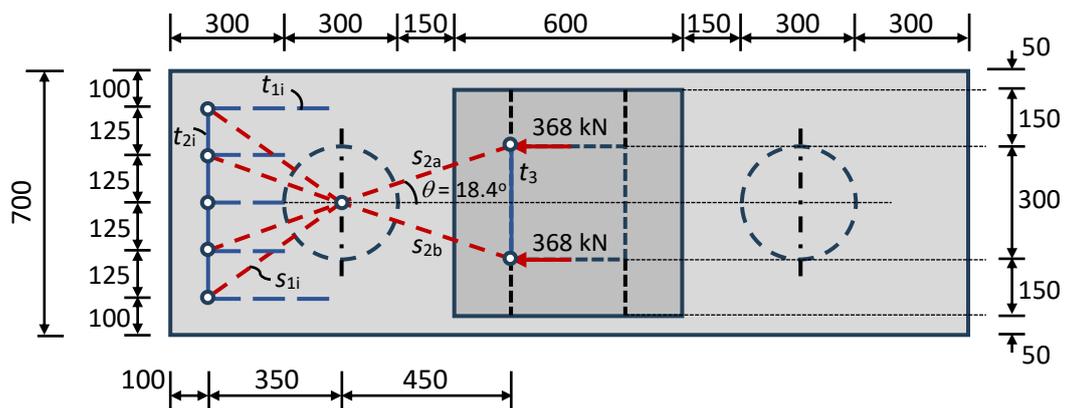
The longitudinal bars are spread over the width of the pile cap as shown in Fig. E6-4(a). Each of the five H20 bars carries a tensile force of $683/5 = 137$ kN. For equilibrium, transverse reinforcement is required at the end of the pile cap. The maximum tensile force in the transverse reinforcement is in tie t_{2b} :

$$F_{t2b} = 137 (\tan \theta_1 + \tan \theta_2) = 137 (\tan 35.5^\circ + \tan 19.7^\circ) = 147 \text{ kN}$$

The required steel area is $147,000/(500/1.15) = 338 \text{ mm}^2$, which can be catered by 2H16 (402 mm^2) bars.



(a) Transverse Forces at End of Pile Cap



(b) Flow of Forces in Pile Cap (Top View)
(Symmetrical about mid-span)

Fig. E6-4 Transfer of Forces in Pile Cap

The flow of forces, observed from the top of the pile cap, is shown in Fig. E6-4(b). The forces in the piles are transmitted to the plinth area through inclined concrete struts s_{2i} ($i = a, b$) which diverge from the longitudinal axis by an angle of $\tan^{-1} (150/450) = 18.4^\circ$. The compression strut below the plinth (C12 in Fig. E6-3) carries a force equal to the force of 735 kN carried by the tensile tie T34. This force may be considered as two compressive forces each equal to 368 kN acting at quarter-points along the width of the plinth. This leads to lateral tensile forces in the tie t_3 with $F_{t3} = 368 \tan 18.4^\circ = 123$ kN. The required area for the ties is $123,000 / (500 / 1.15) = 283 \text{ mm}^2$, which can be provided by adding two horizontal H10 links ($A_{s,prov} = 4 \times 78.5 = 314 \text{ mm}^2$) around the bars extending from the plinth into the pile cap.

Special consideration should be given to the introduction of the shear force from beam TB3 into the pile cap (EN 1992-1-1:2023, 12.3.4(1)). Suppose that the full shear force $V_{Ed,TB3}$ of 163 kN is to be introduced through its compression zone into the pile cap (see Fig. E6-5). If there is insufficient suspension capacity, the zone would be ripped off and separated by the crack. However, the crack is intersected by fibres. Considering the width of the pile cap of 700 mm and a length between the piles of 900 mm, and a post-cracking tensile strength $f_{td,R3}$ of 0.62 MPa, the tensile resistance is $700 \times 900 \times 0.62 / 1000 = 390$ kN. This is more than twice the design shear force of 163 kN, and there is no need for suspension bars. The shear force is hence transmitted by fibres to the top of the pile cap, which is assumed in the strut-and-tie model.

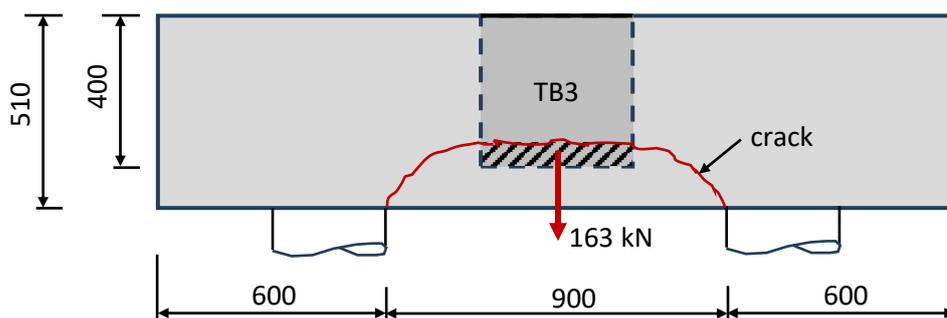
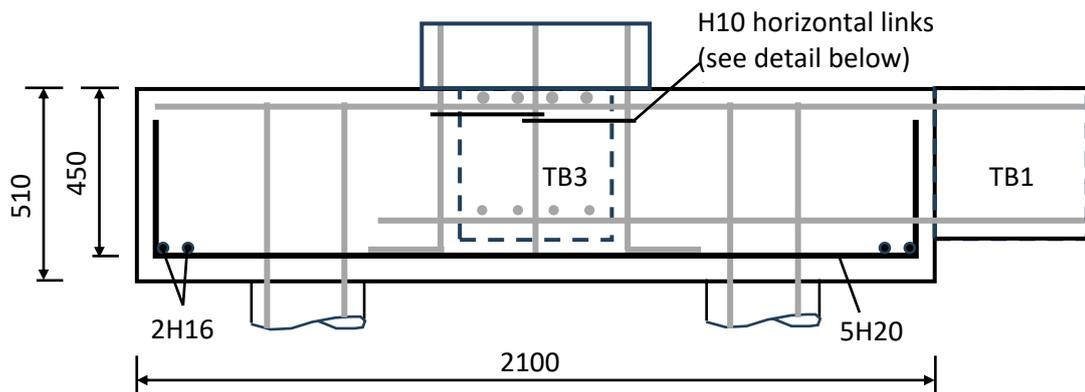
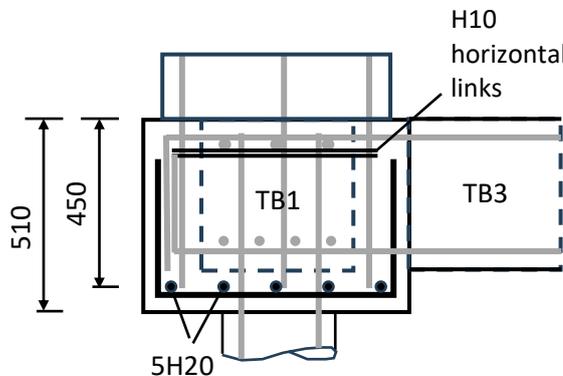


Fig. E6-5 Transfer of Shear Force from Beam TB3

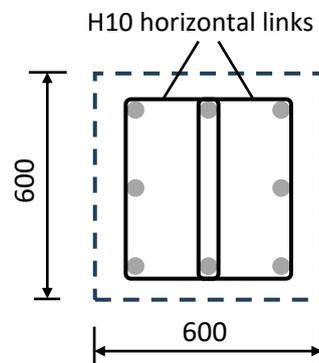
Fig. E6-6 shows the reinforcement details in the pile cap and the supported members. No side and top reinforcement is required in the pile cap as there is no tension developing in these areas. All the required longitudinal reinforcement is to be fully anchored.



Cross-Section of Pile Cap in Longitudinal Direction



Cross-Section in Transverse Direction



Top View of Horizontal Links under Plinth

Fig. E6-6 Reinforcement Details

Design of Struts

The strut C_{12} is subjected to a compressive force of 735 kN (refer Fig. E6-3). As the strut is subjected to transverse compression, the effective strength is $f_{cd} = 0.85 \times 40/1.5 = 22.7$ MPa (SS EN 1992-1-1:2008, 6.5.2). Considering that the strut has a width of 600 mm (same as the plinth) and a depth of $2 \times 50 = 100$ mm, this gives a resistant force of $700 \times 100 \times 22.7 / 1000 = 1587$ kN. Hence, the strut has sufficient strength.

Struts C_{14} and C_{23} are each subjected to a force of $662/\sin 42^\circ = 989$ kN. The strength of the struts is $0.6(1 - f_{ck}/250)f_{cd} = 0.6 \times (1 - 40/250) \times 0.85 \times 40/1.5 = 11.4$ MPa. The required width of the struts is therefore $989,000/(700 \times 11.4) = 123$ mm, which can be accommodated.

Design of Nodes

Nodes 1 and 2 are CCC nodes, and the strength is $(1 - f_{ck}/250) f_{cd} = 19.0$ MPa (SS EN 1992-1-1:2008, 6.5.4), while the stress under the plinth is $2 \times 662,000 / (600 \times 600) = 3.7$ MPa, which is lower. Also, Nodes 3 and 4 are CCT nodes with a compressive strength of $0.85(1 - f_{ck}/250) f_{cd} = 16.2$ MPa. The stress in the pile is $662,000 / (\pi \times 300^2/4) = 9.4$ MPa, which is lower and also less than the compressive strength of the struts C₁₃ and C₂₄. Hence all nodes have adequate strength capacity.

NOTE: *Based on bending theory, the shear capacity of the pile cap is checked as follows (SS EN 1992-1-1:2008, 6.2.2(6)). The design shear force is $V_{Ed} = 662$ kN. Due to the proximity of the applied load to the pile support, this may be reduced to $V_{Ed} (a_v/2d)$, where a_v is the clear distance between the applied load and the pile, equal to 150 mm, and d is the effective depth, equal to 450 mm. However, since $a_v/d = 150/450 = 0.33 < 0.5$, the value of a_v/d is taken as 0.5, and the reduced design shear force is $0.5/2 \times 662 = 166$ kN.*

Assuming 5H20 as longitudinal steel bars, the shear resistance is given by (SS 674, Eq. 6.3):

$$V_{Rd,cf} = \{(0.18/\gamma_c) k [100\rho_l(1+7.5f_{ft,R3}/f_{ctk}) f_{ck}]^{1/3}\} b_w d \geq \{0.035k^{3/2}f_{ck}^{1/2}\} b_w d$$

With $k = 1 + \sqrt{(200/d)} = 1 + \sqrt{(200/450)} = 1.67 < 2$; $\rho_l = 1570/(700 \times 450) = 0.00556 < 0.02$; $f_{ctk} = 0.7f_{ctm} = 0.7 \times 3.02 = 2.11$ MPa, $f_{ft,R3} = 1.75$ MPa,

$$\begin{aligned} V_{Rd,cf} &= \{(0.18/1.5) \times 1.67 \times [100 \times 0.00556 \times (1 + 7.5 \times 1.75/2.11) \times 40]^{1/3}\} \times 700 \times 450/1000 \\ &= 0.689 \times 700 \times 450/1000 = 217 \text{ kN} \\ &\geq \{0.035k^{3/2}f_{ck}^{1/2}\} b_w d = 0.035 \times (1.67)^{3/2} (40)^{1/2} \times 700 \times 450/1000 \\ &= 0.478 \times 700 \times 450/1000 = 150 \text{ kN} \end{aligned}$$

Therefore, $V_{Rd,cf} = 217$ kN $> V_{Ed} (a_v/2d) = 166$ kN.

Also, the design shear force calculated without reduction should be less than $0.5bdvf_{cd}$, where b is the width of the pile cap, $v = 0.6(1 - f_{ck}/250)$ and $f_{cd} = 0.85f_{ck}/1.5$. That is:

$$V_{Ed} = 662 \text{ kN} < 0.5bdvf_{cd} = 0.5 \times 700 \times 450 \times 0.6(1 - 40/250) \times 0.85 \times 40/1.5 \times 10^{-3} = 1799 \text{ kN}.$$

2.7 Box Culverts

Reinforced concrete box culverts are commonly used as tunnels to facilitate the passage of vehicles, passengers or utility services. This example indicates savings in reinforcing steels by incorporating steel fibres in a twin-celled box culvert.

Example 7 – Twin-Cell Box Culvert

Fig. E7-1 shows the cross-section of a twin-cell reinforced concrete (RC) box culvert for a road tunnel. The box culvert consisted of roof slabs S1, base slabs S2, side walls W1 and interior wall W2. All slab and wall elements measure 600 mm in thickness. The concrete cover to the nearest reinforcement is to be 50 mm for all faces except the external faces of the roof and base slabs, for which the cover is to be 55 mm. Under SLS, the crack width is to be limited to 0.25 mm at link diameter plus 40 mm from the outermost main bar for external faces, and 0.30 mm for internal faces. The structure has been designed with concrete strength class C32/40 to withstand load combinations with loads as indicated in Table E7-1.

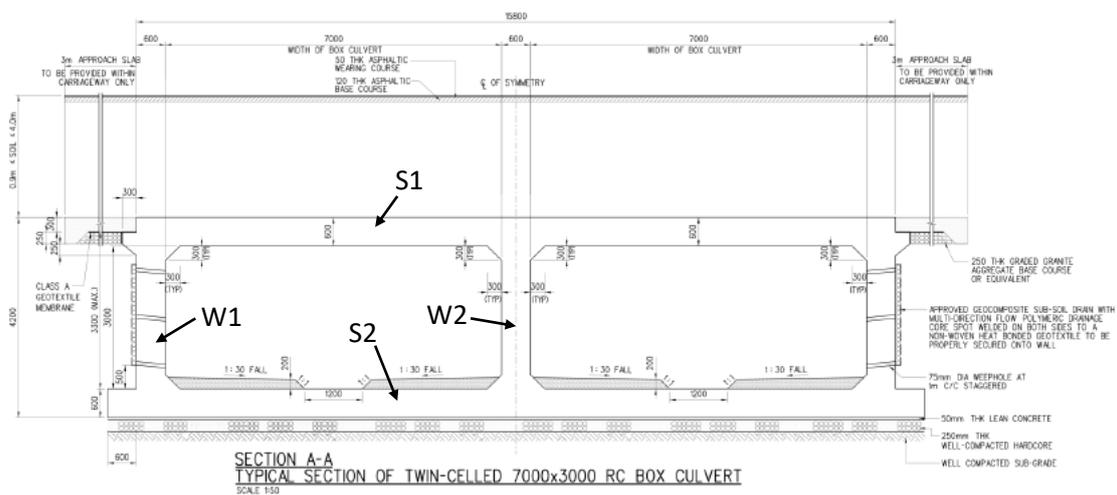


Fig. E7-1 Typical section of twin-celled 7000 mm x 3000 mm RC box culvert

Table E7-1 Load combinations

Load Combination	Vertical Loads	Horizontal Loads	Braking & Acceleration Force	Base Uplift
LC1	Maximum	Maximum	No	Yes
LC2	Maximum	Minimum	No	Yes
LC3	Minimum	Maximum	No	Yes
LC4	Maximum	Maximum	Yes	No
LC5	Minimum	Maximum	Yes	No
LC6	Minimum	Maximum	Yes	Yes

The maximum values of design action effects at ULS and SLS are summarized in Table E7-2. The slabs are predominately subjected to flexure, while the walls to axial loading.

Table E7-2 Design moments, axial loads, and shear forces

Limit State	Action Effects	Location	S1	S2	W1	W2
ULS	Bending moment, M_{Ed} (kNm/m)	At Mid-Span	620	627	- ^a	- ^a
		At Supports	997	945	581	100
	Axial load, N_{Ed} (kN/m)	-	258	343	753	1790
	Shear force, V_{Ed} (kN/m)	Outer/Top Support	626	617	221	53
		Interior/Bottom Support	804	781	284	53
SLS	Bending moment, M_{Ed} (kNm/m)	At Mid-Span	449	451	346 ^b	- ^a
		At Supports	709	670	416	38

Note: ^a Bending moments are negligible;

^b Mid-span moment is in same direction as support moments (i.e., wall is bent in single curvature).

Design using SFRC

The box culvert is herein designed using fibre concrete class C32/40- $R_{13}R_{33}$. The material properties are:

$$f_{ck} = 32 \text{ MPa}; \quad E_{cm} = 33.3 \text{ GPa}; \quad f_{ctm} = 3.02 \text{ MPa}; \quad f_{ctk} = 2.12 \text{ MPa};$$

$$f_{R,1} = 3 \text{ MPa}; \quad f_{R,3} = 3 \text{ MPa}; \quad f_{ft,R1} = 1.35 \text{ MPa}; \quad f_{ft,R3} = 1.11 \text{ MPa};$$

$$\eta_f = 1.0 \text{ (for Slabs S1 and S2), } 0.5 \text{ (for Walls W1 and W2); } \eta_{det} = 1.0;$$

$$f_{yk} = 500 \text{ MPa}; \quad E_s = 200 \text{ GPa}$$

Design for Bending. – For each slab or wall element, $b = 1000 \text{ mm}$, $h = 600 \text{ mm}$. Conservatively, assume $d = 510 \text{ mm}$ and $d/h = 0.85$. The value of $f_{ftd,R3} = \eta_f \eta_{det} f_{ft,R3} / 1.5$ is 0.74 MPa for slabs and 0.37 MPa for walls, with corresponding values of $f_{ftd,R3}/f_{ck} = 0.023$ for slabs and 0.0115 for walls.

The requirement for flexural tensile reinforcement in slabs and walls is evaluated using Design Chart C-2, Appendix C, as shown in Fig. E7-2. The required minimum tensile steel ratio is $A_{s,min} = 0.26(f_{ctm}/f_{yk})bd = 0.157\%bd = 801 \text{ mm}^2$. The results are tabulated in Table E7-3. For Wall W1 and W2, only minimum reinforcement is required except for the exterior face of W1.

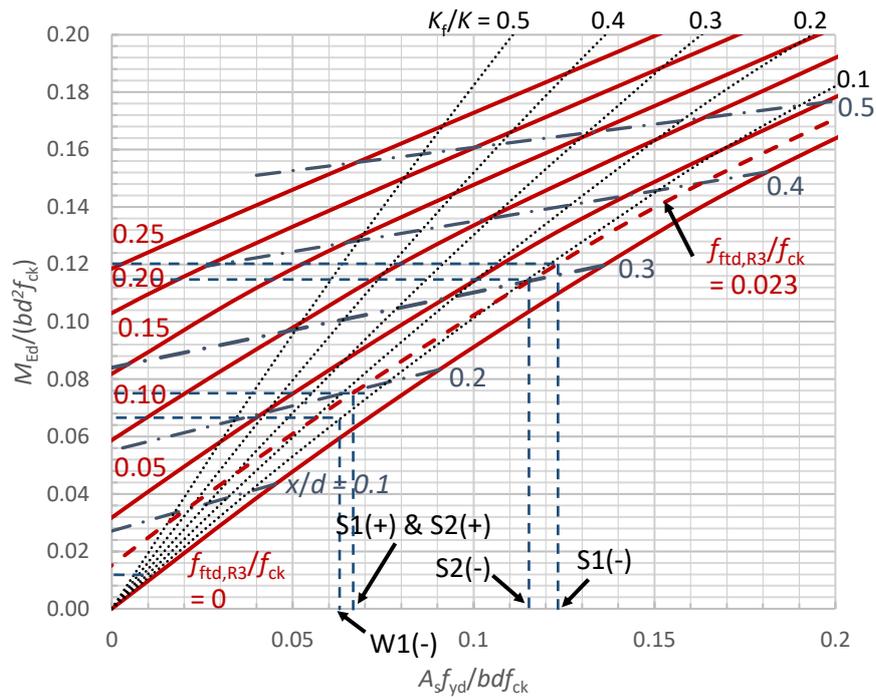


Fig. E7-2 Determination of Flexural Reinforcement (Chart C-2, $d/h = 0.85$)
 (Note: (+) refers to mid-span moment; (-) refers to support moment)

Table E7-3 Longitudinal Tensile Steel Requirements due to Flexure

Member	Location	M_{Ed} (kNm/m)	M_{Ed}/bd^2f_{ck}	$A_s f_{yd}/bdf_{ck}$	A_s/bd	$A_{s,reqd}$ (mm ² /m)
S1	At mid-span	620	0.074	0.068	0.00435	2552
	At support	997	0.120	0.123	0.00787	4617
S2	At mid-span	627	0.075	0.068	0.00435	2552
	At support	945	0.114	0.116	0.00742	4354
W1 (At midspan & support) ^a		581	0.067	0.063	0.00403	2365
W2 (At midspan & support)		Only minimum tensile reinforcement is required to resist flexure.				801

Note: ^a Midspan moment is in the same direction as support moments.

Effect of Axial Load. – Fig. E7-3 shows the design load-moment (N_{Ed} , M_{Ed}) combinations for the slab and wall elements plotted against the N - M interaction curves for symmetrical reinforced sections with $d/h = 0.85$ and $f_{ftd,R3}/f_{ck} = 0.0116$ (extract from Chart D-2(b), Appendix D). The required longitudinal steel area is shown in Table E7-4.

For both S1 and S2, the required longitudinal steel can be provided by H32 bars at 150 mm spacing at each face of the slab. For Wall W1, the required steel is catered for by H25 bars at 150 mm at each face. For the interior wall W2, only a minimum steel area, that is, $0.2\%bh$ or 1200 mm^2 , is required and this can be catered for by H13 bars at 150 mm spacing at both faces.

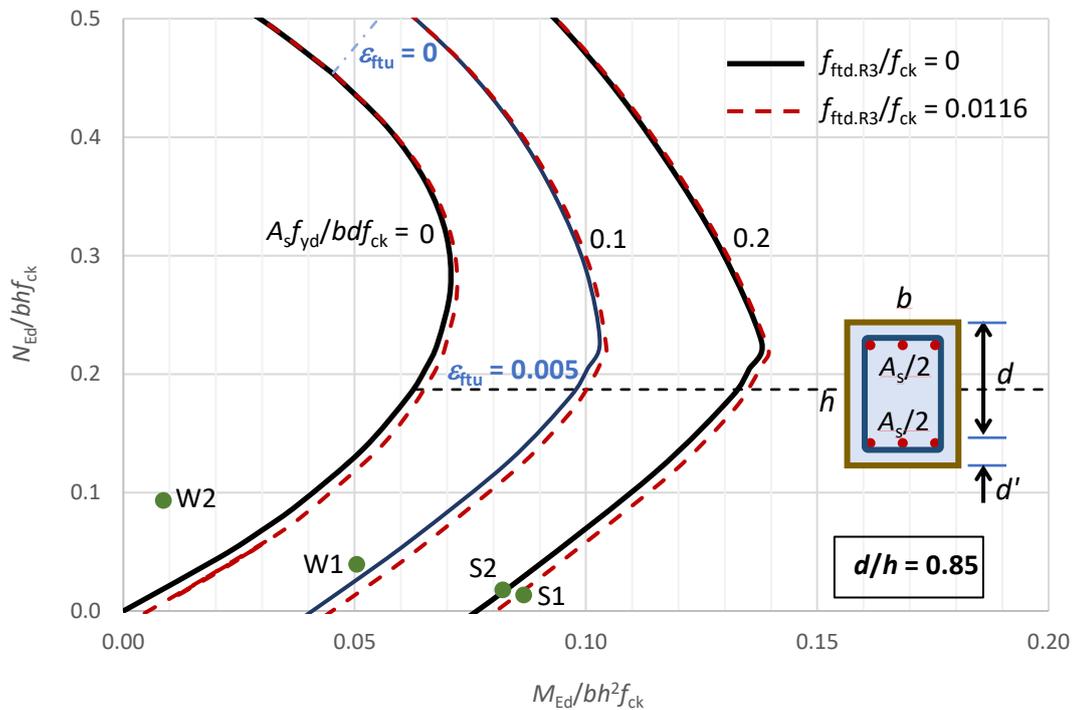


Fig. E7-3 Load-Moment Combinations

Table E7-4 Longitudinal Steel Area Requirement due to Axial Compression

Member	M_{Ed} (kNm/m)	N_{Ed} (kN/m)	M_{Ed}/bh^2f_{ck}	N_{Ed}/bhf_{ck}	$A_s f_{yd}/bdf_{ck}$	A_s (mm^2/m)	Required Longitudinal Reinforcement
S1	997	258	0.0865	0.0134	0.20	7507	H32-150 EF (10720 mm^2/m)
S2	945	343	0.0820	0.0179	0.19	7132	H32-150 EF (10720 mm^2/m)
W1	581	753	0.0504	0.0392	0.08	3003	H20-150 EF (4187 mm^2/m)
W2	100	1790	0.0087	0.0932	0.03	1200	H13-150 EF (1760 mm^2/m)

Design for Shear. – The design shear forces V_{Ed} , taken at the face of support, are compared with the shear resistance $V_{Rd,cf}$ in Table E7-5, neglecting the effect of axial load. The value of θ is conservatively assumed as 45° . In all locations except in Slab S1 and S2 at the connection with the interior wall W2, only nominal shear resistance is required, and this can be catered for using H13 vertical links at 300 mm spacing in both the transverse and longitudinal directions ($A_{sw}/s = 1467 \text{ mm}^2/\text{m}$). In Slab S1 and S2 near the junction with the interior wall, the required shear reinforcement of about $2100 \text{ mm}^2/\text{m}$ can be catered for using H13 links at a transverse spacing of 150 mm and longitudinal spacing of 300 mm ($A_{sw}/s = 2934 \text{ mm}^2/\text{m}$).

The additional tensile longitudinal reinforcement due to shear is also shown in Table E7-5, together with those required to resist bending moment. The final longitudinal steel bar requirement should account for that required by axial load, shown in Table E7-4 for axial compression.

Table E7-5 Shear and Tensile Longitudinal Reinforcement Requirements

Member	Support	η_f	$100\rho^a$	V_{Ed}^b (kN/m)	$V_{Rd,cf}^c$ (kN/m)	$A_{sw,reqd}/s^d$ (mm ² /m)	$A_{sl,add}^e$ (mm ² /m)	$A_{sl,M}^a$ (mm ² /m)	$A_{sl,M+V}^e$ (mm ² /m)	$A_{sl,prov}^e$ (mm ² /m)
S1	Outer	1.0	0.762	626	491	905	720	4617	5337	H32- 150 (5360)
	Interior	1.0	0.762	804	491	2089	925	4617	5542 ^f	
S2	Outer	1.0	0.710	617	480	916	710	4354	5064	
	Interior	1.0	0.710	781	480	2012	898	4354	5252	
W1	Top	0.5	0.358	221	382	905	254	2365	2619	H25- 150 (3267)
	Bottom	0.5	0.358	284	382	905	327	2365	2691	
W2	Top	0.5	0.157	53	290	905	61	802	863	H13- 150 (880)
	Bottom	0.5	0.157	53	290	905	61	802	863	

Note: ^a $\rho = A_{s,M}/bd$ where $A_{s,M}$ is the area of tensile reinforcement required to resist flexure;

^b V_{Ed} is conservatively taken as face shear;

$$^c V_{Rd,cf} = (0.18/\gamma_c) \cdot k \cdot \left[100\rho_l \cdot \left(1 + \frac{7.5f_{ft,R3}}{f_{ctk}} \right) \cdot f_{ck} \right]^{1/3} \cdot b_w d \quad (\text{SS 674, Eq. 6.3})$$

^d Derived from $V_{Ed} = V_{Rd} = 0.75V_{Rd,s} + V_{Rd,cf}$ and $\frac{A_{sw,reqd}}{s} = \frac{V_{Rd,s}}{z f_{yw} \cot \theta} \geq \frac{0.08\sqrt{f_{ck}}}{f_{yk}} b_w$; in all cases, the minimum shear reinforcement applies.

^e $A_{sl,add}$: additional longitudinal reinforcement due to shear; $A_{sl,M+V}$: longitudinal tensile reinforcement due to moment and shear; $A_{sl,prov}$: longitudinal tensile reinf. provided;

^f This amount could be reduced and catered for by H32-150 considering the enhancement of shear capacity near the support (see Note b above).

Crack Width Calculations. – The maximum crack width under the moments at SLS are evaluated for the sections at the external faces at the supports and internal faces at the mid-

span for Slabs S1 and S2, and for the external face at the bottom of Wall W1. The effect of axial load is ignored. A cover of 55 mm to the link is assumed for external faces, and 50 mm for internal faces. Also, main bars consisting of H32 bars are considered for Slabs S1 and S2, and H25 bars for Wall W1. The area of main bars is taken as that provided to meet ULS requirements (see Table E7-5). In all cases, H13 links are assumed. The input parameters are summarized in Table E7-6(a).

Table E7-6(b) shows the calculated maximum crack widths using the simplified approach considering a fictitious stress $\sigma_{s,fict}$ in the reinforcement (SS 674, Eq. 7.7 to 7.9) and an effective modular ratio of 12. The crack width at “link diameter + 40 mm” from the main bar is calculated by assuming that the crack width varies linearly with zero width at the face of the bar. The calculated crack widths are within the allowable crack widths of 0.25 mm for external faces and 0.3 mm for internal faces.

Table E7-6(a) Input Parameters for Crack Width Calculations

Member/Face	M_{SLS} (kNm/m)	c^a (mm)	d (mm)	A_s (mm ² /m)	ρ (= A_s/bd)	$f_{ftd,R1}$ (MPa)
S1/ext	709	68	516	5360	0.010388	1.35
S1/int	449	63	521	4692	0.010288	1.35
S2/ext	670	68	516	5360	0.010388	1.35
S2/int	451	63	521	5360	0.010288	1.35
W1/ext	412	68	532	3267	0.006283	0.675

Note: ^a c : cover to main bar.

Table E7-6(b) Maximum Crack Widths

Member/Face	x_c^a (mm)	$h_{c,eff}^b$ (mm)	$\rho_{s,eff}^c$	k_f^d	$\sigma_{s,fict}$ (MPa)	$\epsilon_{sm} - \epsilon_{cm}^e$ ($\times 10^{-6}$)	$S_{r,max}^f$ (mm)	w_k^g (mm)	$w_{k,40}^h$ (mm)
S1/ext	201.2	132.9	0.0403	0.446	295	713	305.9	0.218	0.170
S1/int	202.4	132.5	0.0404	0.446	185	476	288.7	0.118	0.099
S2/ext	201.2	132.9	0.0403	0.446	278	666	305.9	0.204	0.159
S2/int	202.4	132.5	0.0404	0.446	186	412	288.7	0.119	0.100
W1/ext	166.5	144.5	0.0226	0.223	271	817	377.2	0.308	0.240

Note: ^a x_c : neutral axis depth assuming cracked section and ignoring presence of fibres;

^b $h_{c,eff} = \min [2.5(h - d), (h - x_c)/3, h/2]$;

^c $\rho_{s,eff} = A_s/bh_{c,eff}$ where $b = 1000$ mm;

^d $k_f = f_{ftd,R1}/f_{ctm} \leq 1$ with $f_{ftd,R1}$ in SLS;

^e $(\epsilon_{sm} - \epsilon_{cm}) = (1 - k_f)[\sigma_{s,fict} - k_t(f_{ct,eff}/\rho_{p,eff})(1 + \alpha_e\rho_{p,eff})]/E_s$ (SS 674, Eq. 7.7);

^f $S_{r,max} = 3.4c + 0.425 \times 0.8 \times 0.5 \times (1 - k_f) \phi/\rho_{p,eff}$ (SS 674, Eq. 7.8);

^g $w_k = S_{r,max}(\epsilon_{sm} - \epsilon_{cm})$ (SS 674, Eq. 7.9);

^h $w_{k,40}$: crack width at “link diameter (13 mm) + 40 mm” from main bar (= $w_k \times 53/c$, refer NA to SS EN 1992-1-1:2008, Table NA.4).

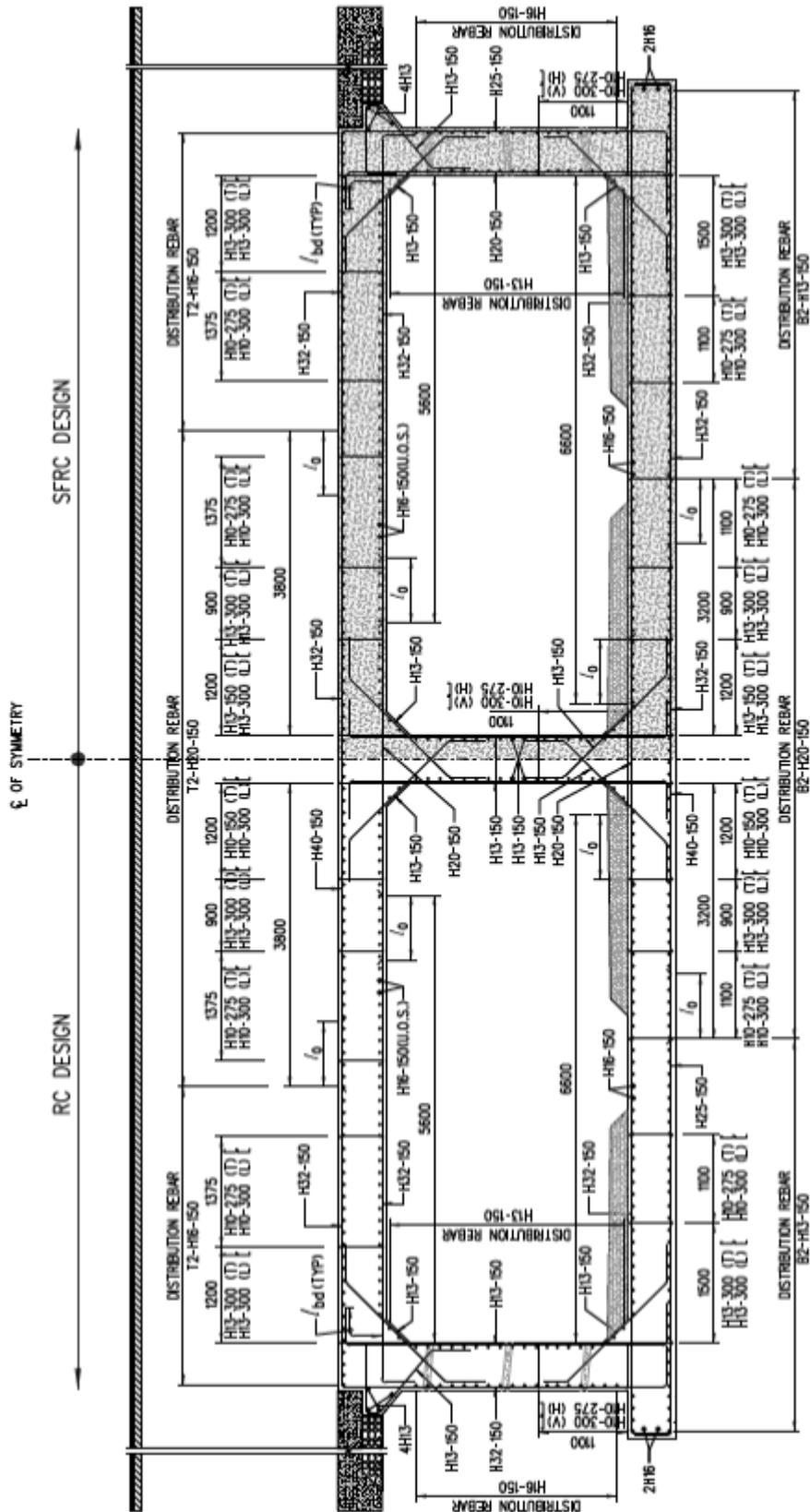
Comparison with RC Design

The reinforcement requirements for the RC box culvert are compared to those for SFRC box culvert for the critical sections in Table E7-6 and shown in Fig. E7-4. The transverse links and reinforcement for thermal shrinkage are assumed to be the same. Some savings in longitudinal steel reinforcement is noted, while the crack widths are generally smaller using SFRC design.

Table E7-7 Comparison of Reinforcement Requirements

Member	Location	Longitudinal Reinforcement				Max. Crack Width (mm)		
		Face	RC Design	SFRC Design	ΔA_s^a (mm ² /m)	RC Design	SFRC Design	Allowable
S1	Mid-span & Ext. Support	Tension	H32-150	H32-150	0	0.226	0.099	0.30
		Comp.	H32-150	H32-150	0	-	-	-
	Int. Support	Tension	H40-150	H32-150	-3013	0.222	0.170	0.25
		Comp.	H32-150	H32-150	0	-	-	-
S2	Mid-span & Ext. Support	Tension	H32-150	H32-150	0	0.242	0.100	0.30
		Comp.	H25-150	H32-150	+2040	-	-	-
	Int. Support	Tension	H40-150	H32-150	-3013	0.205	0.159	0.25
		Comp.	H32-150	H32-150	0	-	-	-
W1	-	Ext. Face	H32-150	H25-150	-2040	0.167	0.240	0.30
		Int. Face	H13-150	H20-150	+1213	-	-	-
W2	-	Both Faces	H13-150	H13-150	0	-	-	-

Note: ^a ΔA_s : change in steel area due to SFRC design (+ indicates increase; - indicates reduction)



RC AND SFRS DESIGN
 REINFORCEMENT DETAILS OF TWIN-CELLED 7000x3000 RC BOX CULVERT
 SCALE 1/50

Fig. E7-4 Reinforcement Details for RC Box Culvert

2.8 Secant Bored Pile Wall

Secant Bored Pile (SBP) Walls, consisting of reinforced concrete (RC) circular piles with soft piles cast in-between, are commonly seen in underground structures such as underpasses. The example illustrates the design of such circular bored piles using steel fibre reinforced concrete (SFRC), which indicates a 20 percent reduction in steel reinforcement requirement.

References

- BS 8110 : Part 2 : 1985. *Structural use of concrete. Part 2. Code of practice for special circumstances.* British Standards Institution.
- PD 6687-1:2020 (2021). Background Paper to the National Annexes to BS EN 1992-1, BS EN 1992-3 and BS EN 1992-4. *British Standards Institution.*

Example 8 – Bored Piles

Fig. E8-1(a) shows the cross-section of a bored RC circular pile in a SBP wall for a road tunnel. It has a diameter D of 1000 mm and is reinforced with 20H40 and 20H20 bars on two circumferences. It is provided with circular links, being H20 links at 125 mm spacing for the shear critical zone. The concrete strength class is C32/40 and cover to the links is 75 mm. The characteristic yield strength, f_{yk} , of reinforcement is 500 MPa. The RC section has been designed to meet the requirements of ULS and SLS as shown in Fig. E8-2.

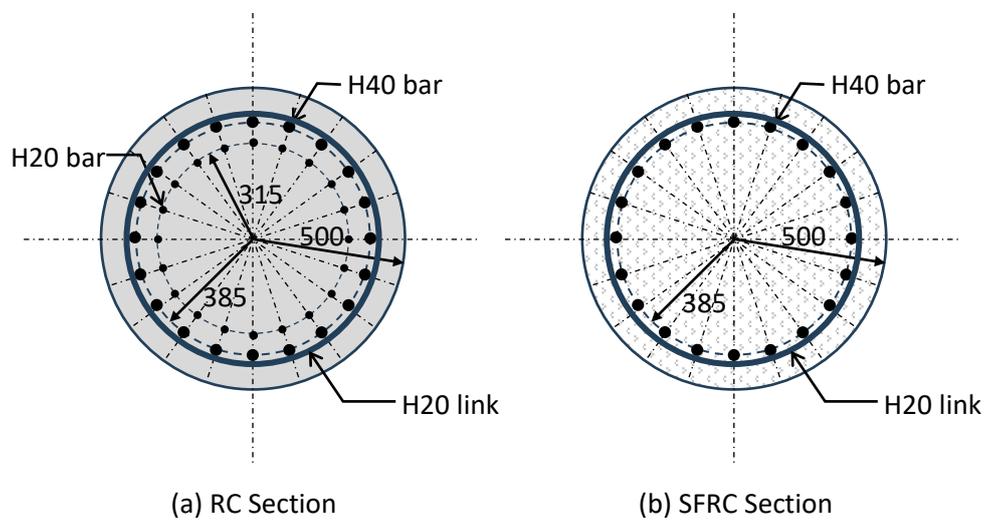


Fig. E8-1 Cross-Section of Bored Pile (All dimensions are in mm.)

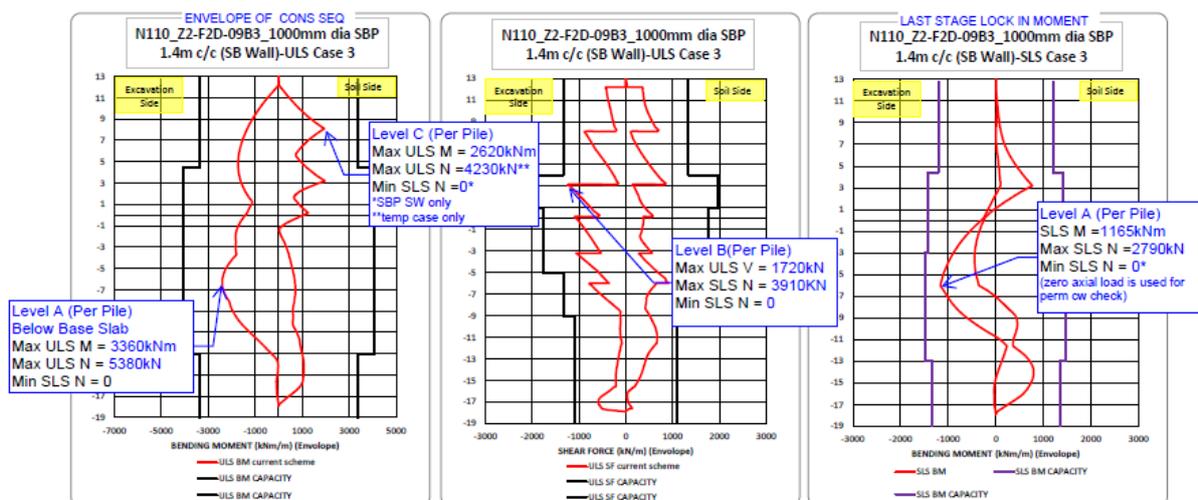


Fig. E8-2 Bending Moment and Shear Force Envelopes

(Note: Levels A, B and C are at bottom, middle and top of pile, respectively.)

At ULS, the pile is to carry load-moment combinations, (N_{Ed}, M_{Ed}) , equal to (5380, 3360) at Level A below the base slab, and (4230, 2620) at Level C below the soffit of the capping beam, where N_{Ed} and M_{Ed} , are in kN and kNm, respectively. It is also required to carry an ultimate shear force, V_{Ed} , equal to 1720 kN, at Level B below the tunnel roof slab. The SLS requires crack width to be limited to 0.25 mm at 40 mm from the links.

Alternative Design using SFRC

Consider the SFRC section shown in Fig. E8-1(b) with only twenty H40 bars at the same locations as the RC section in Fig. E8-1(a). A fibre concrete strength and ductility class of C32/40-R₁3R₃3 is adopted. Accordingly, the following material properties are derived.

$$f_{ck} = 32 \text{ MPa}; f_{ctk} = 2.12 \text{ MPa}; E_{cm} = 3.3 \text{ GPa};$$

$$f_{ft,R1} = 0.45 f_{R1} = 1.35 \text{ MPa}; f_{ft,R3} = 0.37 f_{R3} = 1.11 \text{ MPa};$$

Assuming $\eta_f = 0.5$ (see NOTE below) and $\eta_{det} = 1.0$ results in

$$f_{ftd,R1} = \eta_f \eta_{det} f_{ft,R1}/1.5 = 0.45 \text{ MPa}; f_{ftd,R3} = \eta_f \eta_{det} f_{ft,R3}/1.5 = 0.37 \text{ MPa}.$$

NOTE: *It would be necessary to verify the value of η_f in heavily reinforced vertical elements cast under tremie conditions.*

Design for ULS

Bending and Axial Loading. – The piece-wise linear N-M interaction diagrams for the sections shown in Fig. E8-1 are derived based on the method of strain compatibility, considering four key points: (a) pure compression; (b) decompression; (c) balanced condition; and (d) pure bending. The strain and stress diagrams across the cross-section for the SFRC sections are shown in Fig. E8-3. It is noted that the RC section shown in Fig. E8-1(a) has a longitudinal steel ratio, A_s/A_c of 4%, and the SFRC section in Fig. E8-1(b) a ratio of 3.2%.

The N-M diagrams are shown in Fig. E8-4 with corresponding values of (M_{Rd}, N_{Rd}) shown in Table E8-1. The fibres contributed little to the capacity of the section and the SFRC section, even without fibres would be able to resist the design (M_{Ed}, N_{Ed}) combinations mentioned earlier.

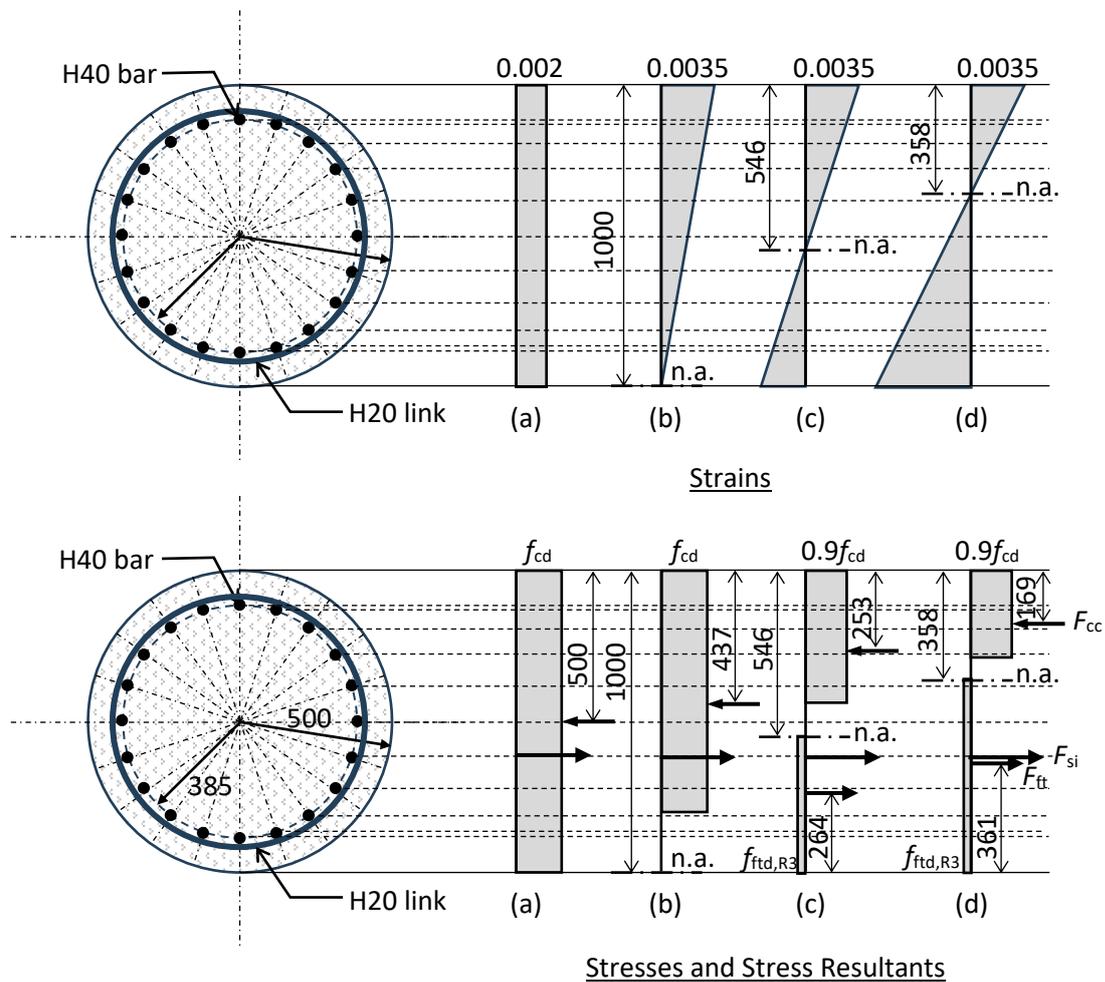


Fig. E8-3 Strains and Stresses Corresponding to: (a) Pure Compression; (b) De-compression; (c) Balanced Condition; and (d) Pure Bending

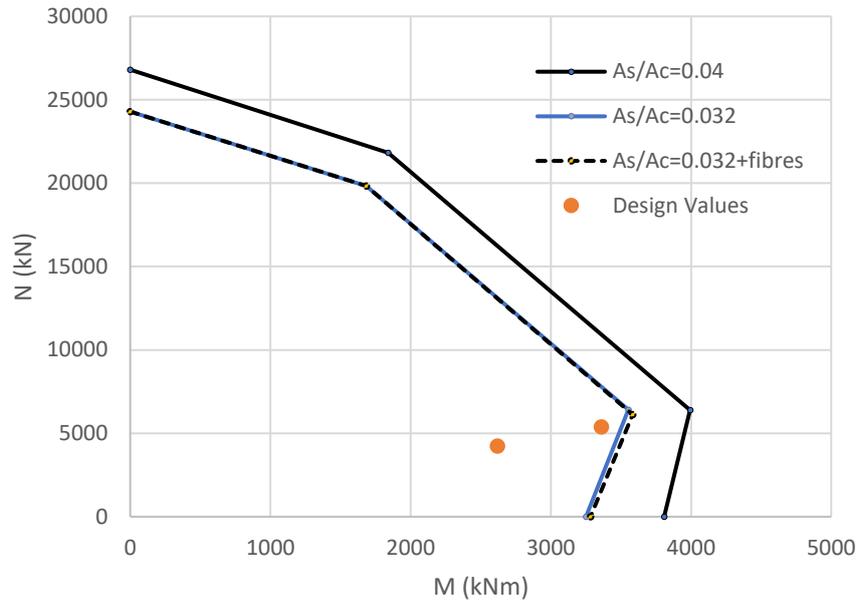


Fig. E8-4 *N-M* Interaction Diagrams

Table E8-1 Values of *N* and *M* at Key Points

Reinforcement	(a) Pure Compression	(b) De-Compression		(c) Balanced Condition		(d) Pure Bending
	N_o (kN)	N_{dec} (kN)	M_{dec} (kNm)	N_b (kN)	M_b (kNm)	M_o (kNm)
$A_s/A_c = 4\%$	26804	21815	1840	6394	3993	3809
$A_s/A_c = 3.2\%$	24292	19825	1684	6416	3553	3251
$A_s/A_c = 3.2\% +$ steel fibres	24292	19825	1684	6126	3584	3283

Shear. – Following the approach of the 2nd Generation EC2 (EN 1992:2003, Cl. 8.2.3(9)) for circular section, a web width b_w less than D_h where D_h is the hook (link) diameter, may be assumed. In this example, assuming H20 links, the value of $D_h = 1000(D) - 2 \times 75(\text{cover}) - 2 \times 20(\text{diameter}) / 2 = 830$ mm. Referring to Fig. E8-5, assume $b_w = 2x (D/2) \sin 36^\circ = 588$ mm (< 830 mm). The number of H40 tension steel bars within b_w is five, giving an area A_s of 6280mm² with a centroid at $y_t = 348$ mm below the centroidal axis of the section. The corresponding tensile force is $T = A_s f_{yd}$ or 2730 kN.

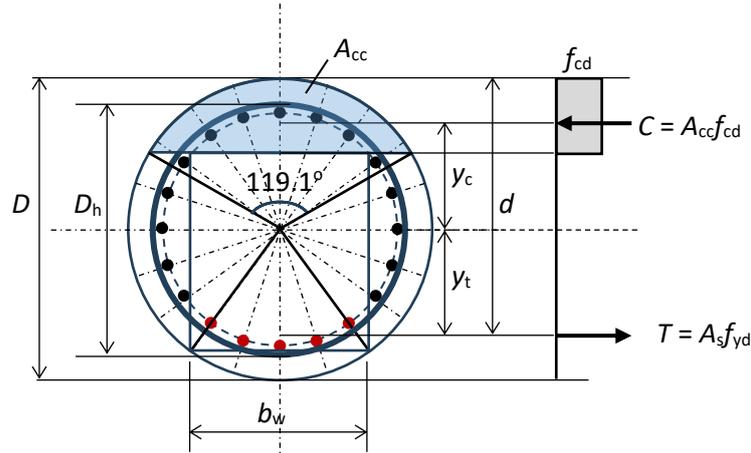


Fig. E8-5 Determination of b_w

The force in the compression chord is $C = T = 2730$ kN. The area of the segment forming the compression chord, A_{cc} , is $C/f_{cd} = 2730 \times 10^3 / (0.85 \times 32 / 1.5) = 150,575$ mm². The distance of the centroid of the segment from the centroidal axis of the circle is $y_c = 355$ mm. The level arm is therefore $z = y_c + y_t = 355 + 348 = 703$ mm, and the effective depth $d = D/2 + y_t = 1000/2 + 348 = 848$ mm.

a. Check for maximum shear capacity

Assume $\cot \theta = 2$, i.e., $\theta = 26.6^\circ$. The maximum shear capacity is (SS EN 1992-1-1, Eq. (6.9))

$$V_{Rd,max} = b_w \cdot z \cdot \frac{v f_{cd}}{\cot \theta + \tan \theta} = 588 \cdot 703 \cdot \frac{0.6 \left(1 - \frac{32}{250}\right) \cdot \frac{32}{1.5}}{2 + \frac{1}{2}} \times 10^{-3} = 1845 \text{ kN} \quad (\text{i})$$

$$> V_{Ed} = 1720 \text{ kN} \quad \dots \text{ok}$$

b. Determine shear link requirement

The shear resistance of fibre reinforced section may be taken as [Design Guide (Tan 2022)], Eq. (6.3):

$$V_{Rd} = 0.75 V_{Rd,s} + V_{Rd,cf}$$

where neglecting the beneficial effect of axial compression (SS 674, Eq. (6.2),

$$V_{Rd,cf} = (0.18/\gamma_c) \cdot k \cdot \left[100 \rho_l \cdot \left(1 + \frac{7.5 f_{t,R3}}{f_{ctk}} \right) \cdot f_{ck} \right]^{1/3} \cdot b_w d \quad (\text{ii})$$

in which $\gamma_c = 1.5$, $k = 1 + \sqrt{\frac{200}{d}} = 1 + \sqrt{\frac{200}{848}} = 1.486$, $\rho_l = \frac{A_s}{b_w d} = \frac{6280}{588 \cdot 848} = 0.0126$, $f_{ft,R3} = 1.1 \text{ MPa}$, $f_{ctk} = 2.12 \text{ MPa}$. Hence,

$$V_{Rd,cf} = (0.18/1.5) \cdot 1.486 \cdot \left[1.26 \cdot \left(1 + \frac{7.5 \cdot 1.1}{2.12} \right) \cdot 32 \right]^{\frac{1}{3}} \cdot 588 \cdot 848 \cdot 10^{-3} = 518 \text{ kN}$$

Equating $V_{Rd} = V_{Ed}$ in Eq. (i) above and using SS EN 1992-1-1, Eq. (6.8) for $V_{Rd,s}$ as well as replacing $\frac{A_{sw}}{s}$ by $\left(\frac{b_w}{D_h}\right) \frac{A_{sw}}{s}$ leads to:

$$V_{Rd,s} \equiv \left(\frac{b_w}{D_h}\right) \frac{A_{sw}}{s} z f_{ywd} \cot \theta = \frac{(V_{Ed} - V_{Rd,cf})}{0.75} = \frac{(1720 - 518)}{0.75} = 1602 \text{ kN}$$

from which

$$\frac{A_{sw}}{s} = \left(\frac{D_h}{b_w}\right) \frac{V_{Rd,s}}{z f_{ywd} \cot \theta} = \left(\frac{830}{588}\right) \frac{1602 \cdot 10^3}{703 \cdot \left(\frac{500}{1.15}\right) \cdot 2} = 3.70 \text{ mm}$$

Therefore, provide H20 circular links at 160 mm, $\left(\frac{A_{sw}}{s}\right)_{prov} = 3.93 \text{ mm}$.

c. Determine additional longitudinal reinforcement

The additional tensile force due to shear is $N_t = V_{Ed} \cot \theta = 1720 \times 2 = 3440 \text{ kN}$. This force can be accounted for in the section analysis involving M_{Ed} and N_{Ed} based on equilibrium requirements.

Design for SLS

Crack width calculation. – The maximum moment M_{SLS} at SLS is 1165 kNm (Fig. E8-2) at Level A. Using a modular ratio of $E_s/(0.5E_{cm}) = 200 / (0.5 \times 33.3) = 16.7$, and following the procedure given in Appendix B, the section curvature $1/r$ and neutral axis depth x are found to be $1.817 \times 10^{-3} / \text{mm}$ and 380 mm respectively (see Fig. E8-6). The forces acting on the section are:

Force in concrete compression zone, $F_{cc} = 1326 \text{ kN}$;

Force in compression steel bars, $F_{sc} = 617 \text{ kN}$;

Force in tension steel bars, $F_{st} = 1713 \text{ kN}$; and

Force due to fibres in tension zone, $F_{ft} = 230 \text{ kN}$.

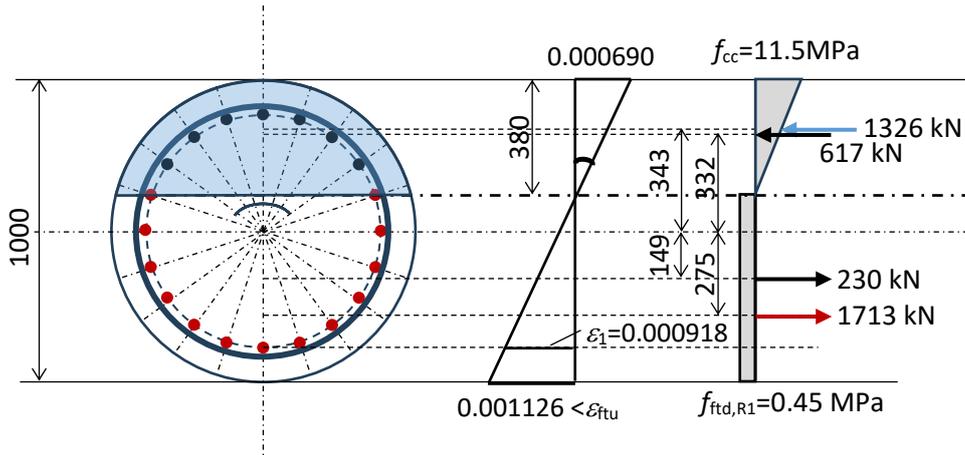


Fig. E8-5 Analysis for section curvature and neutral axis depth under M_{SLS}

Check for equilibrium:

$$F_{cc} + F_{sc} - F_{st} - F_{ft} = 1326 + 617 - 1713 - 230 = 0 \text{ kN} \quad \dots \text{ok}$$

$$\begin{aligned} F_{cc} (0.343) + F_{sc} (0.332) + F_{st} (0.275) + F_{ft} (0.149) \\ = 1326 (0.343) + 617 (0.332) + 1713 (0.275) + 230 (0.149) \\ = 454.8 + 204.8 + 471.1 + 34.3 = 1165.0 \text{ kNm} = M_{SLS} \quad \dots \text{ok} \end{aligned}$$

The maximum crack width is calculated following the recommendations of Published Document PD 6687-1:2020 as follows.

a. Steel strain ϵ_1

From Fig. E8-5, the strain in the outermost steel bar $\epsilon_1 = 0.000918 = 918 \times 10^{-6}$.

b. Strain due to concrete tension stiffening ϵ'

Referring to Fig. E8-6, the concrete stress varies linearly from zero at the neutral axis to 0.7 MPa at the extreme fibre in tension. The tensile force generated by this stress distribution is $T = 156.8 \text{ kN}$. Dividing T by the area of steel bars in tension, equal to $13 \times 1256 = 16328 \text{ mm}^2$, and the elastic modulus of steel gives:

$$\epsilon' = \frac{T}{A_s E_s} = \frac{156.8 \cdot 10^3}{16328 \cdot 200 \cdot 10^3} = 48 \times 10^{-6}$$

c. Average strain at the level at which cracking is considered ϵ_m

The crack width is checked at the surface between the two lowest layers of steel bars. The height of the level from the extreme compressive fibre is $a' = D/2(1 + \cos 9^\circ) = 994 \text{ mm}$. The average strain is thus

$$\epsilon_m = \epsilon_1 - \frac{a' - x}{d - x} \epsilon' = 918 \times 10^{-6} - \frac{994 - 380}{885 - 380} \times 48 \times 10^{-6} = 860 \times 10^{-6}$$

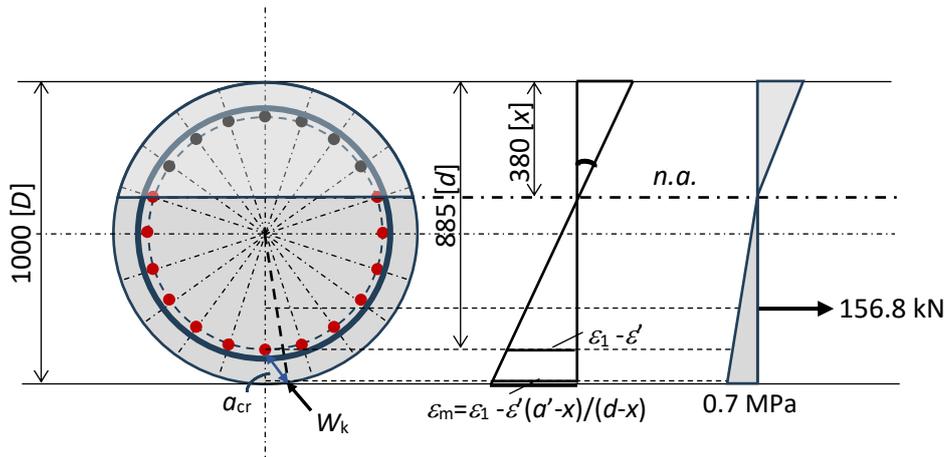


Fig. E8-6 Calculation of average strain ε_m

d. Calculation of crack width W_k

The distance from the point calculated to the surface of the nearest longitudinal bar is $a_{cr} = 114$ mm. With $c = 95$ mm, the crack width is given by:

$$W_k = \frac{3a_{cr}\varepsilon_m}{1 + \frac{2(a_{cr}-c)}{h-x}} = \frac{3 \times 114 \times 860 \times 10^{-6}}{1 + \frac{2(114-95)}{1000-380}} = 0.277 \text{ mm}$$

At 40 mm from the link, $a_{cr} = 84$ mm, $a' = 924$ mm, $\varepsilon_m = 866 \times 10^{-6}$, and considering $c = 60$ mm, the crack width is estimated as:

$$W_k = \frac{3a_{cr}\varepsilon_m}{1 + \frac{2(a_{cr}-c)}{D-x}} = \frac{3 \times 84 \times 866 \times 10^{-6}}{1 + \frac{2(84-60)}{1000-380}} = 0.203 \text{ mm}$$

Alternatively, it may be assumed that the crack width increases linearly with distance a_{cr} from the surface of the nearest bar, for which the crack width is calculated as:

$$W_k = \frac{84}{114} \times 0.277 = 0.204 \text{ mm}$$

Appendices

Design Aids

A. Relation between Fibre Content (kg/m^3) and Fibre Volume Fraction, V_f (%)

Fibre dosage is generally specified in terms of weight per unit volume (kg/m^3) of concrete. Table A-1 shows the equivalency of fibre content (kg/m^3) and fibre volume fraction (that is, ratio of total volume of fibres per unit volume of concrete, in percentage), for steel fibres and polymeric fibres (typically macro-synthetic fibres).

Table A-1 Equivalency of fibre content (kg/m^3) and volume fraction (%)

Type	Steel Fibres (EN 14889-1)		Synthetic Fibres (EN 14889-2)	
Density (kg/m^3)	7850		910	
Volume Fraction, V_f (%)	Fibre Content (kg/m^3)	Approx. Fibre Content* (kg/m^3)	Fibre Content (kg/m^3)	Approx. Fibre Content** (kg/m^3)
0.25	19.6	20	2.3	-
0.33	25.9	25	3.0	3
0.5	39.3	40	4.6	4.5
0.66	51.8	50	6.0	6
0.75	58.9	60	6.8	-
1	78.5	80	9.1	9
1.25	98.1	100	-	-
1.5	117.8	120	-	-

*to nearest 5 kg/m^3 ; **to nearest 0.5 kg/m^3

NOTE 1 Fibre Content (kg/m^3) = V_f (%) x Density (kg/m^3) / 100

NOTE 2 EN 14889-2 recommends dosage of 2.5 to 7.5 kg/m^3 for polymeric fibres.

B. Cracked Section Properties of Fibre Reinforced Concrete Section

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

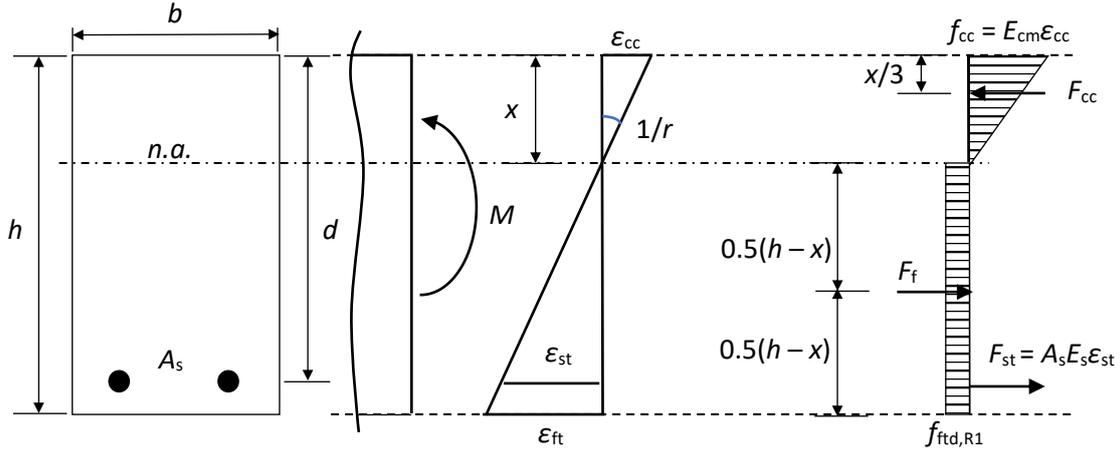


Fig. B-1 Stress and strain diagrams for cracked FRC section

The internal forces acting on the section are:

$$F_{cc} = \frac{1}{2} b x E_c \varepsilon_{cc} ; F_f = b(h-x) f_{ftd,R1} ; F_{st} = A_s E_s \varepsilon_{st} \quad (B.1)$$

where

$$\varepsilon_{st} = \frac{(d-x)}{x} \varepsilon_{cc} ; \varepsilon_{cc} = \frac{1}{r} x = \left(\frac{1}{r} \cdot d \right) \frac{x}{d} \quad (B.2)$$

in which $\frac{1}{r}$ is the section curvature.

Moment equilibrium gives:

$$M = F_{cc} \left(\frac{2}{3} x \right) + F_f \left[\frac{1}{2} (h-x) \right] + F_{st} (d-x) \quad (B.3)$$

Substituting (B.2) into (B.1) and then (B.1) into (B.3) gives, after normalizing:

$$\frac{M}{b d^2 E_c} = \frac{1}{3} \left(\frac{x}{d} \right)^3 \left(\frac{1}{r} \cdot d \right) + \frac{1}{2} \left(\frac{h}{d} - \frac{x}{d} \right)^2 \frac{f_{ftd,R1}}{E_c} + \alpha \rho \left(1 - \frac{x}{d} \right)^2 \left(\frac{1}{r} \cdot d \right) \quad (B.4)$$

where $\alpha = \frac{E_s}{E_c}$ and $\rho = \frac{A_s}{b d}$.

Using Eq. (B.4), for a specified set of values of d/h and $\alpha\rho$, the relation between $\frac{M}{bd^2E_c}$ and $\left(\frac{1}{r} \cdot d\right)$ can be obtained for different values of $\frac{f_{ftd,R1}}{E_c}$ by varying the value of $\frac{x}{d}$. The derived relations are shown in the following figures:

Chart B-1 : $d/h = 0.80$

Chart B-2 : $d/h = 0.85$

Chart B-3 : $d/h = 0.90$

Chart B-4 : $d/h = 0.95$

with $\alpha\rho = 0.025, 0.05, 0.10$ and 0.15 , in sub-charts (a), (b), (c) and (d), respectively.

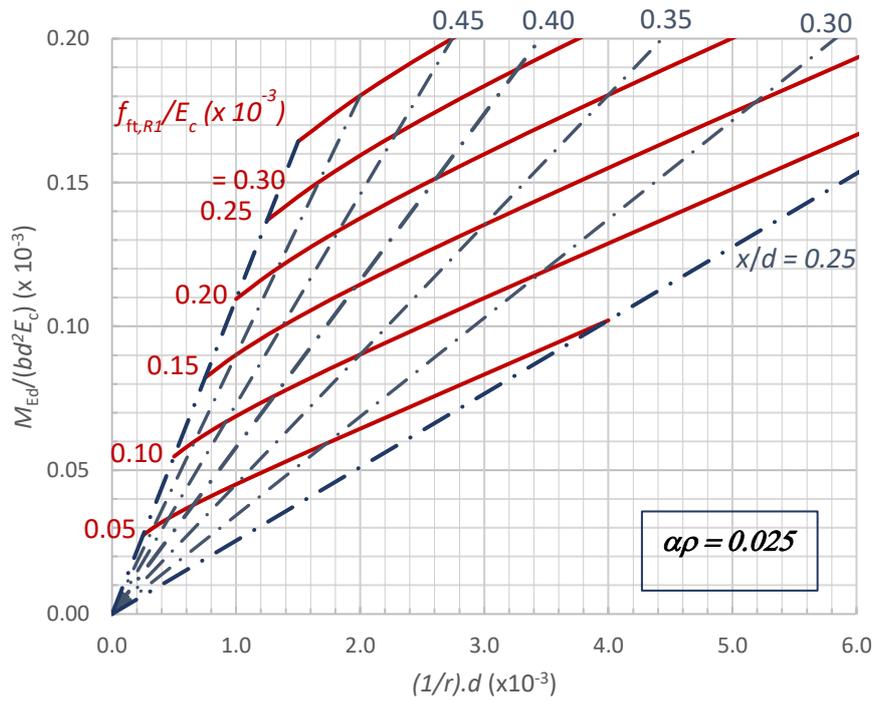


Chart B-1(a) Moment-Curvature Relation ($d/h = 0.8$; $\alpha\rho = 0.025$)

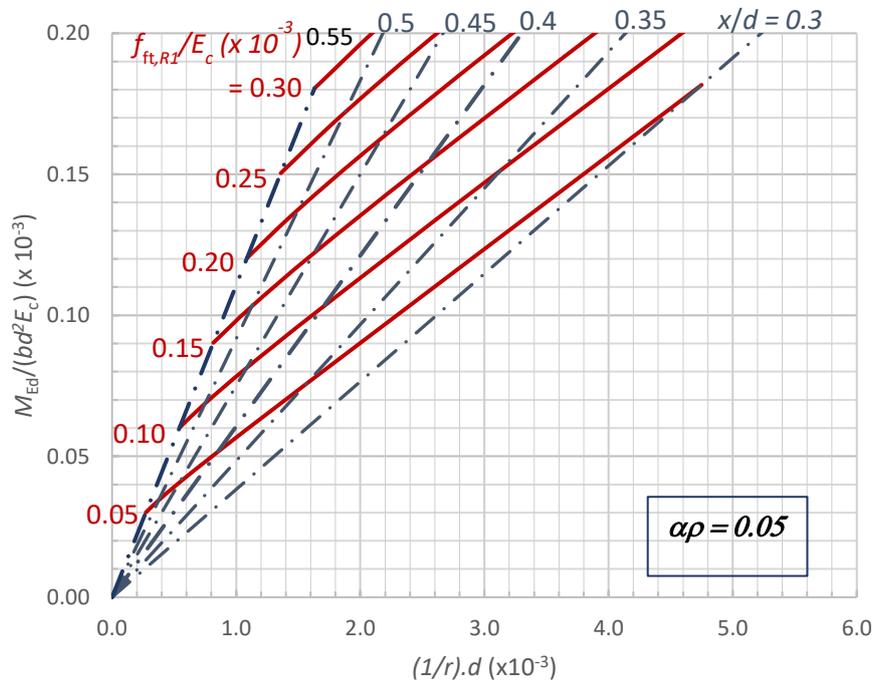


Chart B-1(b) Moment-Curvature Relation ($d/h = 0.8$; $\alpha\rho = 0.05$)

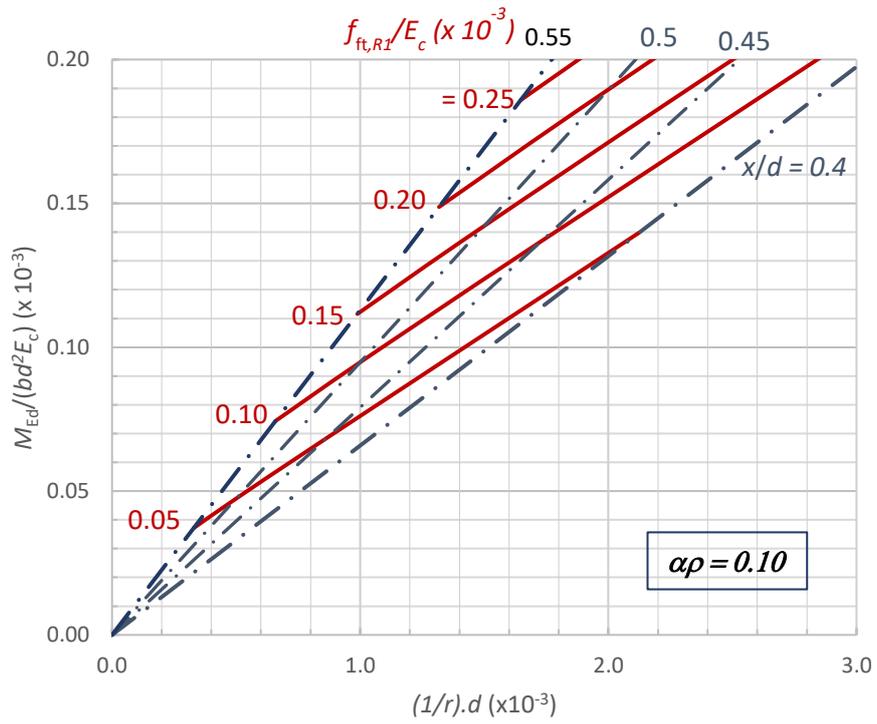


Chart B-1(c) Moment-Curvature Relation ($d/h = 0.8$; $\alpha\rho = 0.10$)

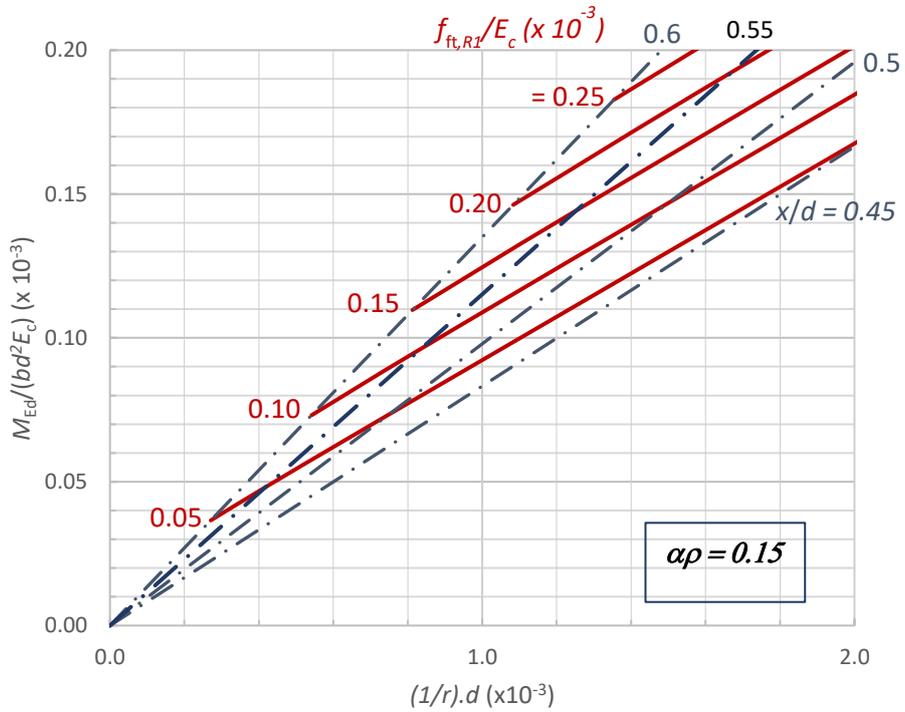


Chart B-1(d) Moment-Curvature Relation ($d/h = 0.8$; $\alpha\rho = 0.15$)

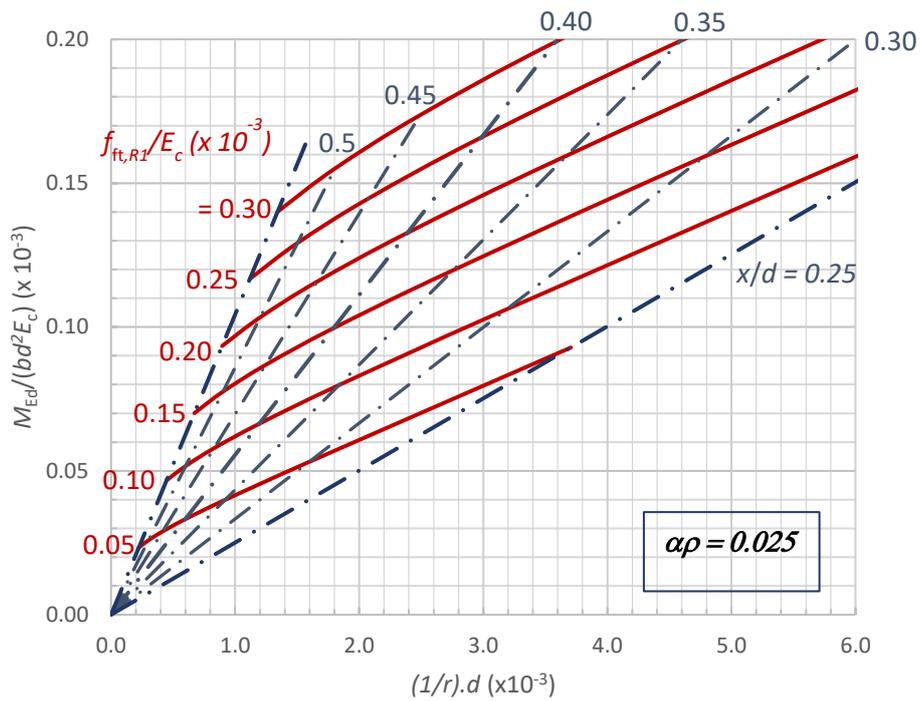


Chart B-2(a) Moment-Curvature Relation ($d/h = 0.85$; $\alpha\rho = 0.025$)

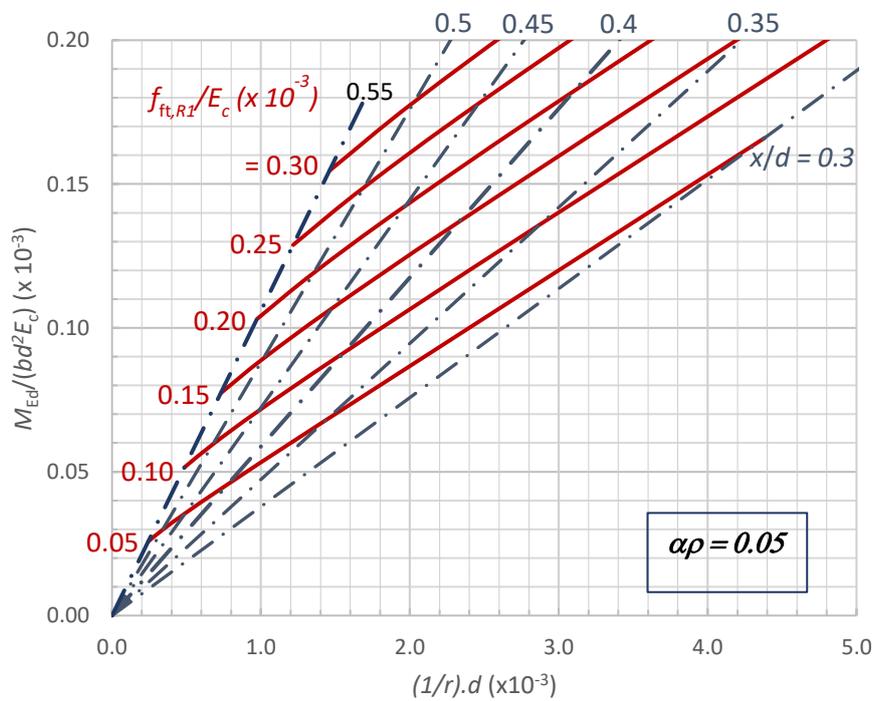


Chart B-2(b) Moment-Curvature Relation ($d/h = 0.85$; $\alpha\rho = 0.050$)

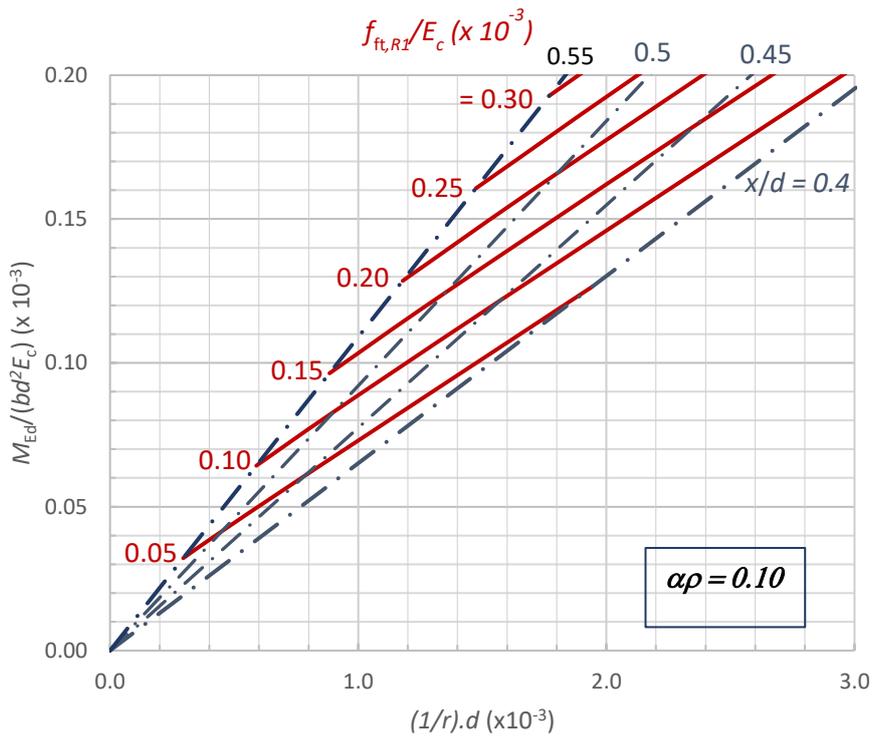


Chart B-2(c) Moment-Curvature Relation ($d/h = 0.85$; $\alpha\rho = 0.10$)

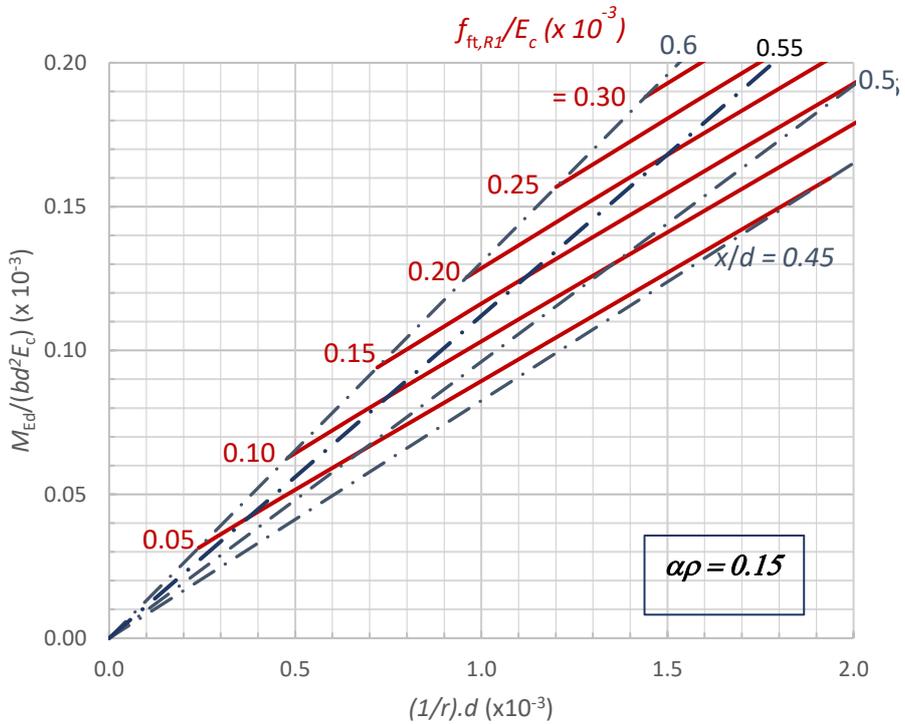


Chart B-2(d) Moment-Curvature Relation ($d/h = 0.85$; $\alpha\rho = 0.15$)

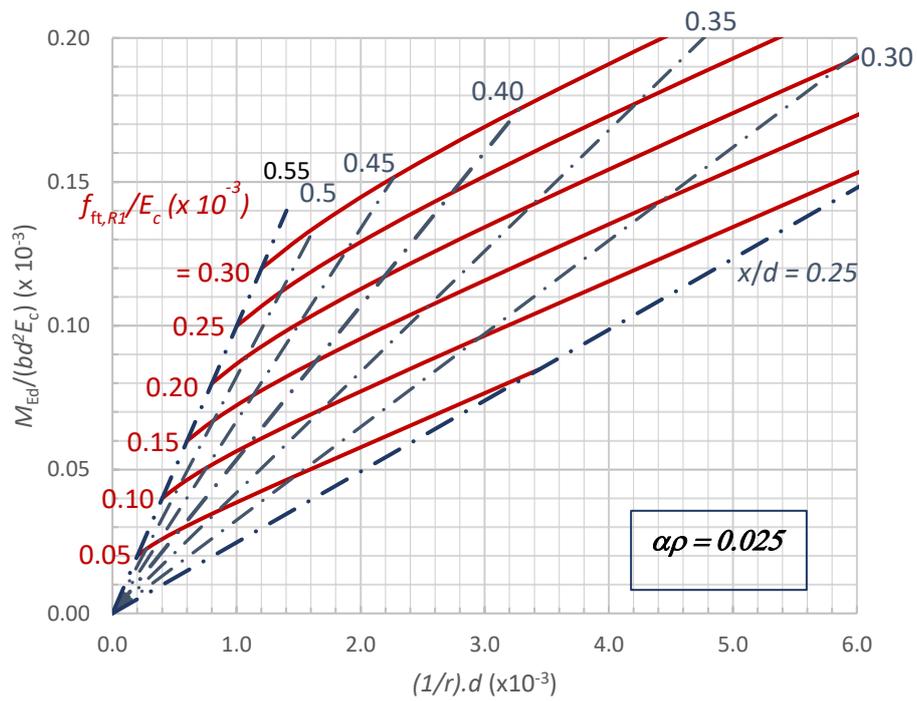


Chart B-3(a) Moment-Curvature Relation ($d/h = 0.90$; $\alpha\rho = 0.025$)

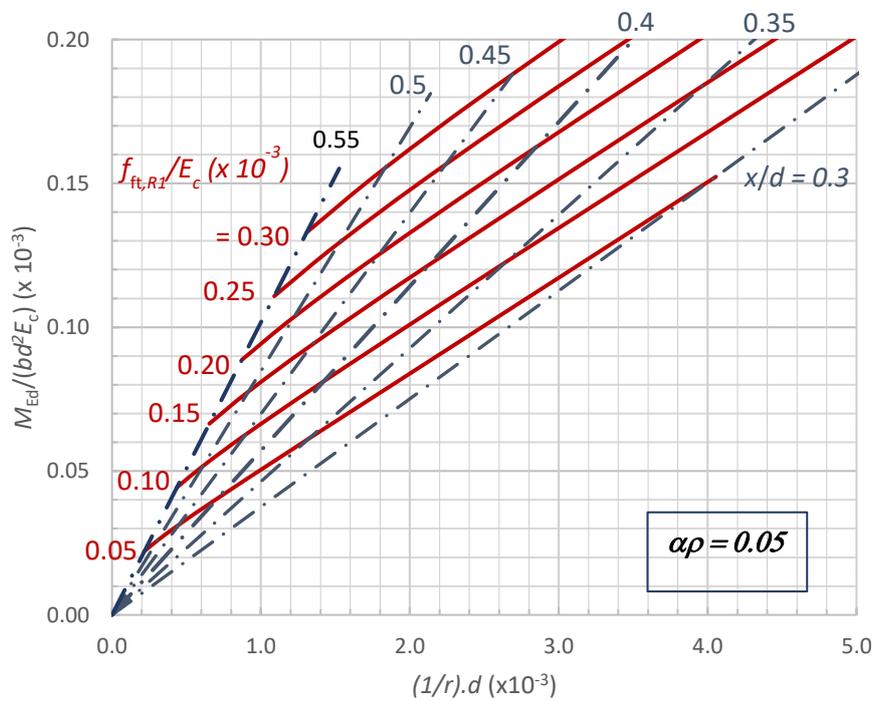


Chart B-3(b) Moment-Curvature Relation ($d/h = 0.90$; $\alpha\rho = 0.050$)

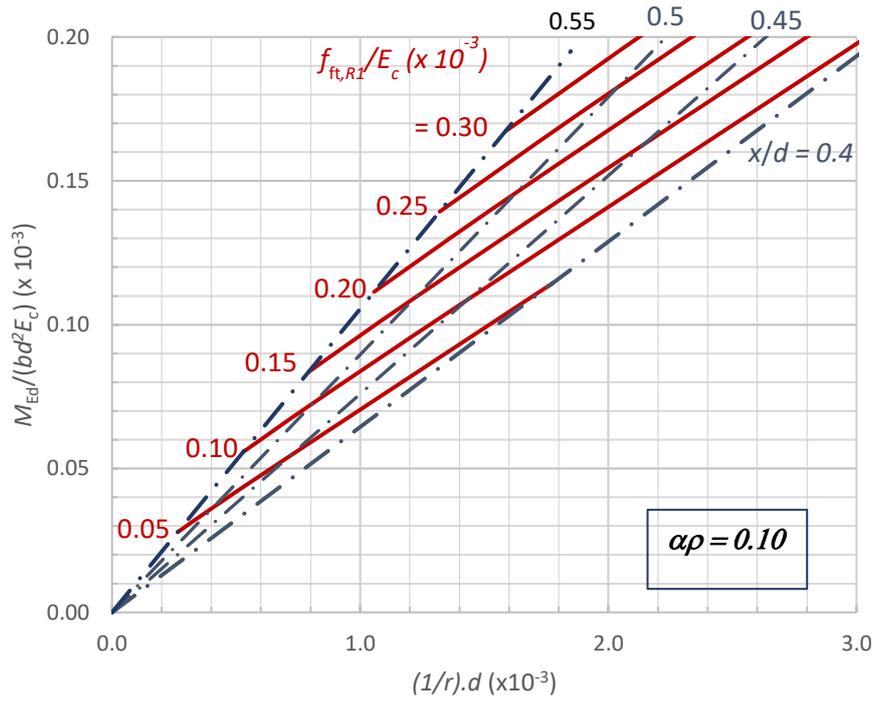


Chart B-3(c) Moment-Curvature Relation ($d/h = 0.90$; $\alpha\rho = 0.10$)

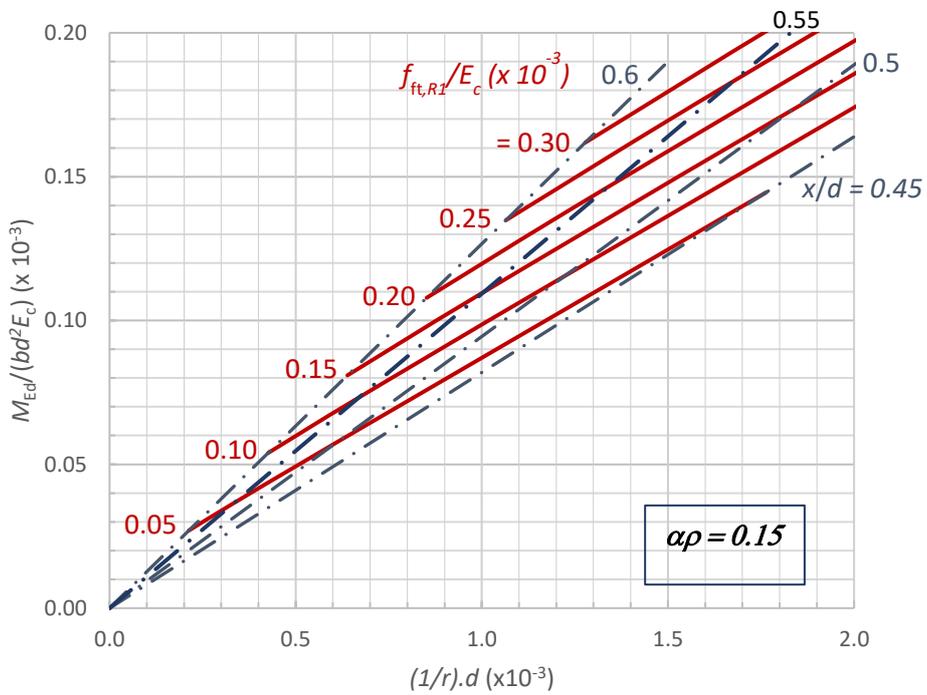


Chart B-3(d) Moment-Curvature Relation ($d/h = 0.90$; $\alpha\rho = 0.15$)

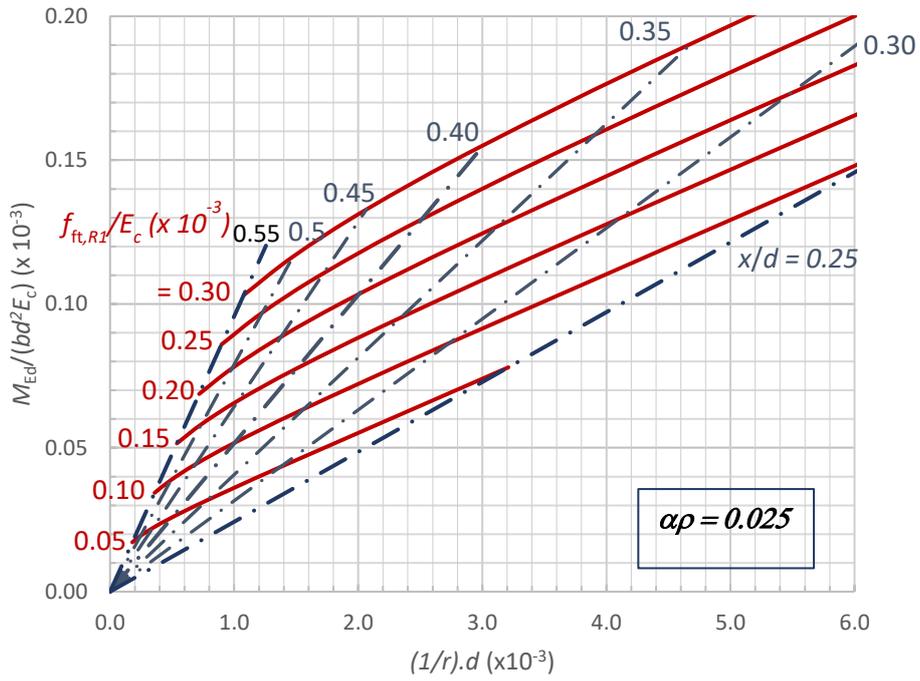


Chart B-4(a) Moment-Curvature Relation ($d/h = 0.95$; $\alpha\rho = 0.025$)

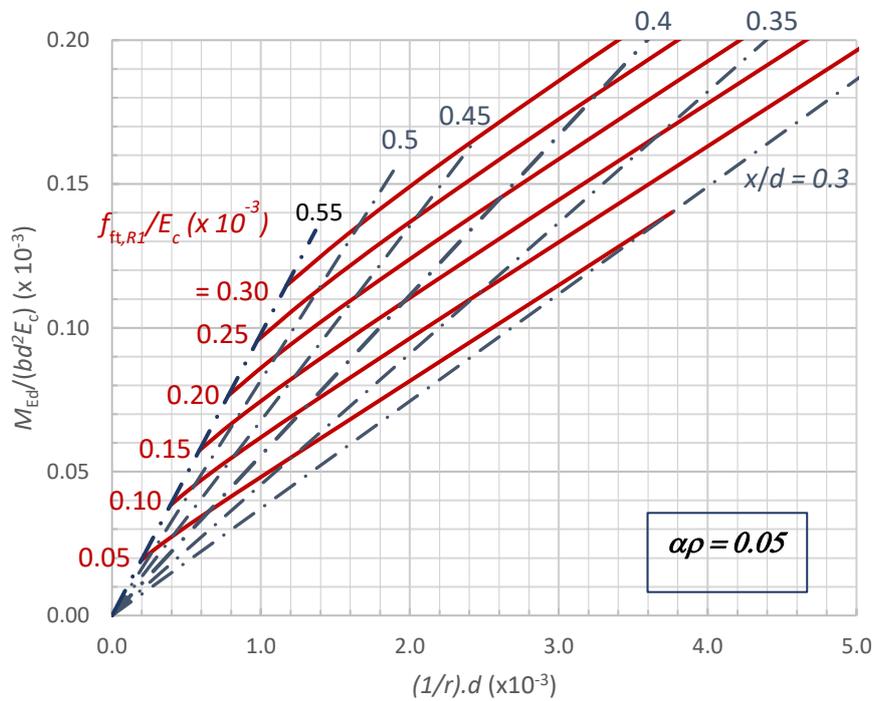


Chart B-4(b) Moment-Curvature Relation ($d/h = 0.95$; $\alpha\rho = 0.050$)

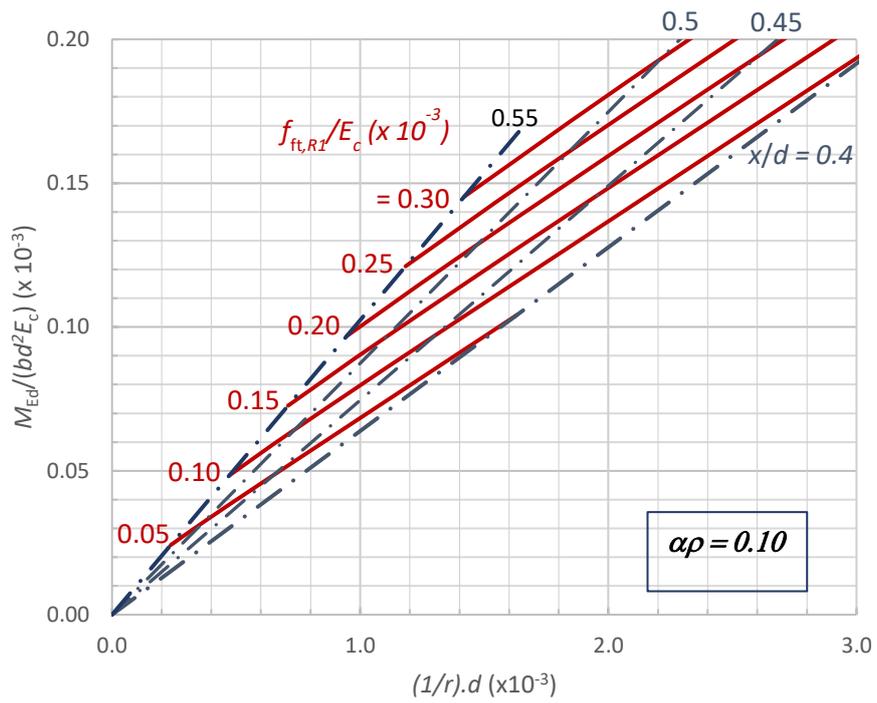


Chart B-4(c) Moment-Curvature Relation ($d/h = 0.95$; $\alpha\rho = 0.10$)

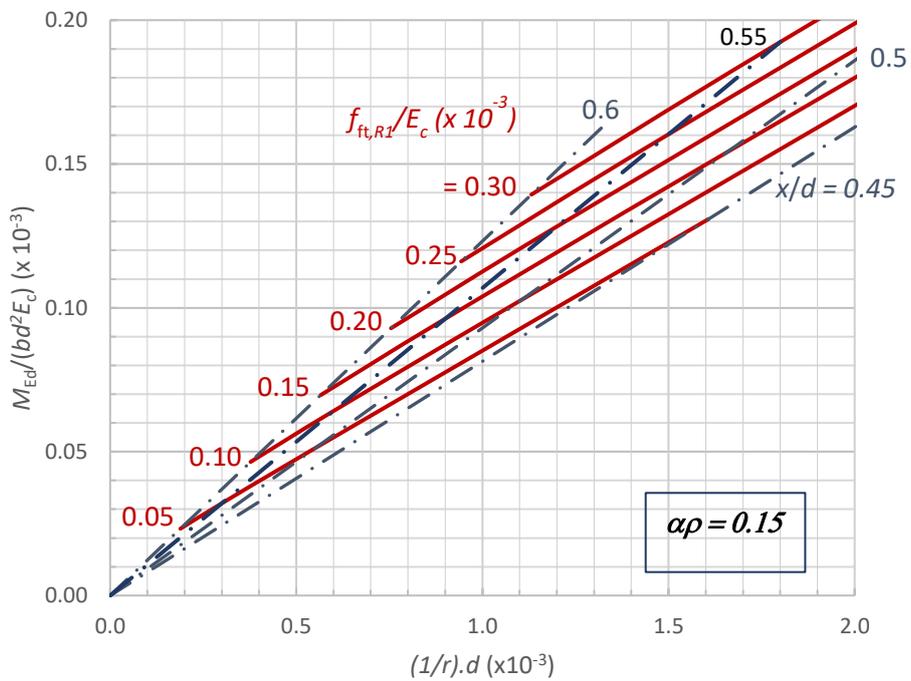


Chart B-4(d) Moment-Curvature Relation ($d/h = 0.95$; $\alpha\rho = 0.15$)

C. Design Charts for Bending

Reproduced from Design Guide (Tan 2022), Appendix A.

The design charts are developed for moment resistance of 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. C-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 . **For fibre concrete sections without conventional bars (i.e., $A_s f_{yd} / b d f_{ck} = 0$), the charts are also applicable for $\alpha_{cc,f} = 0.85$.** For other values of $\alpha_{cc,f}$, Chart D-5 in Appendix D may be used to determine M_{Rd} .

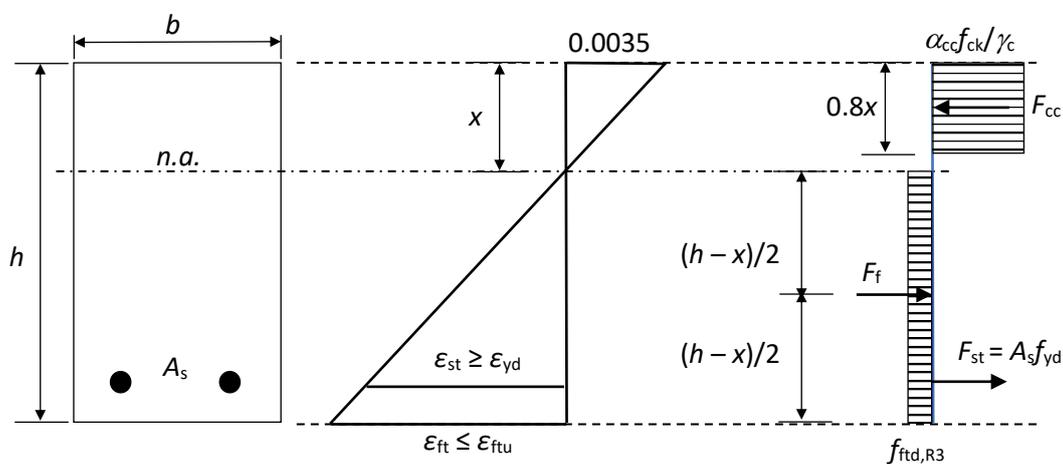


Fig. C-1 Strain and stress distributions

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

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

$$M_{Ed} = F_f [0.6x + \frac{1}{2}(h-x)] + A_s f_{yd}(d - 0.4x) \quad (C.2)$$

which simplify to:

$$F_f = 0.453bxf_{ck} - A_s f_{yd} \quad (C.3)$$

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

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

$$M_{Ed} = (0.453bx f_{ck} - A_s f_{yd}) (0.5h + 0.1x) + A_s f_{yd} (d - 0.4x) \quad (C.5)$$

or

$$M_{Ed}/bd^2 f_{ck} = [0.453(x/d) - A_s f_{yd}/bd f_{ck}][0.5/(d/h) + 0.1(x/d)] + A_s f_{yd}/bd f_{ck} [1 - 0.4(x/d)] \quad (C.6)$$

Also,

$$F_f = b(h - x) f_{ftd,R3} \quad (C.7)$$

From (C.3) and (C.7),

$$b(h - x) f_{ftd,R3} = 0.453bx f_{ck} - A_s f_{yd} \quad (C.8)$$

Hence,

$$A_s f_{yd}/bd f_{ck} = 0.453(x/d) - [1/(d/h) - (x/d)] (f_{ftd,R3}/f_{ck}) \quad (C.9)$$

For a set of values of d/h and $f_{ftd,R3}/f_{ck}$, the value of $A_s f_{yd}/bd f_{ck}$ can be obtained from Eq. (C.9) for a specified value of x/d , and subsequently the value of $M_{Ed}/bd^2 f_{ck}$ can be calculated from Eq. (C.6). Using this procedure, Design Charts C-1 to C-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.

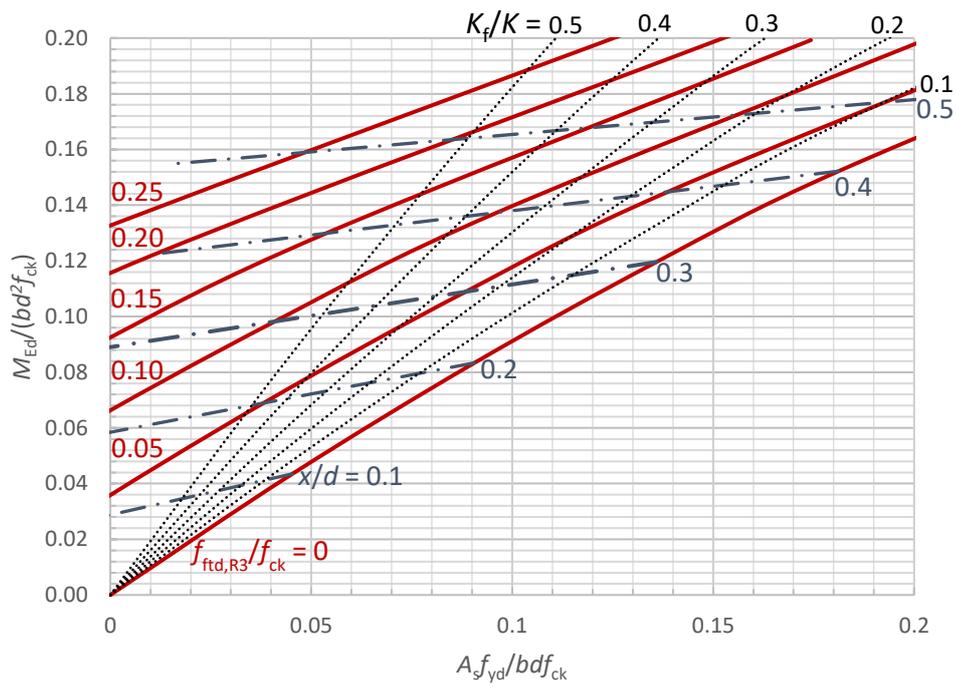


Chart C-1 Design for Bending ($d/h = 0.80$)

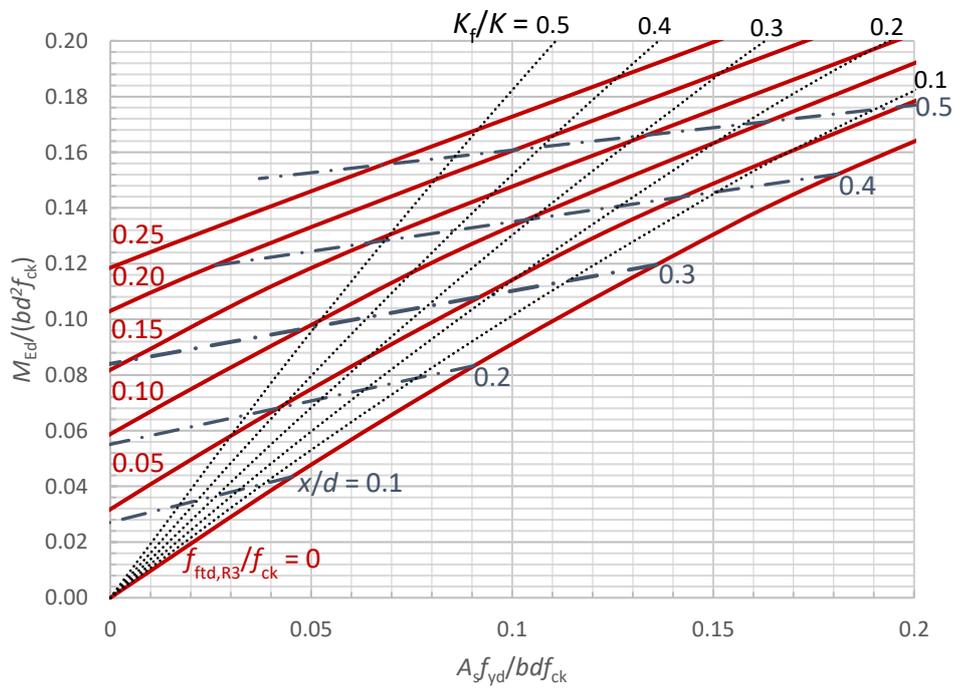


Chart C-2 Design for Bending ($d/h = 0.85$)

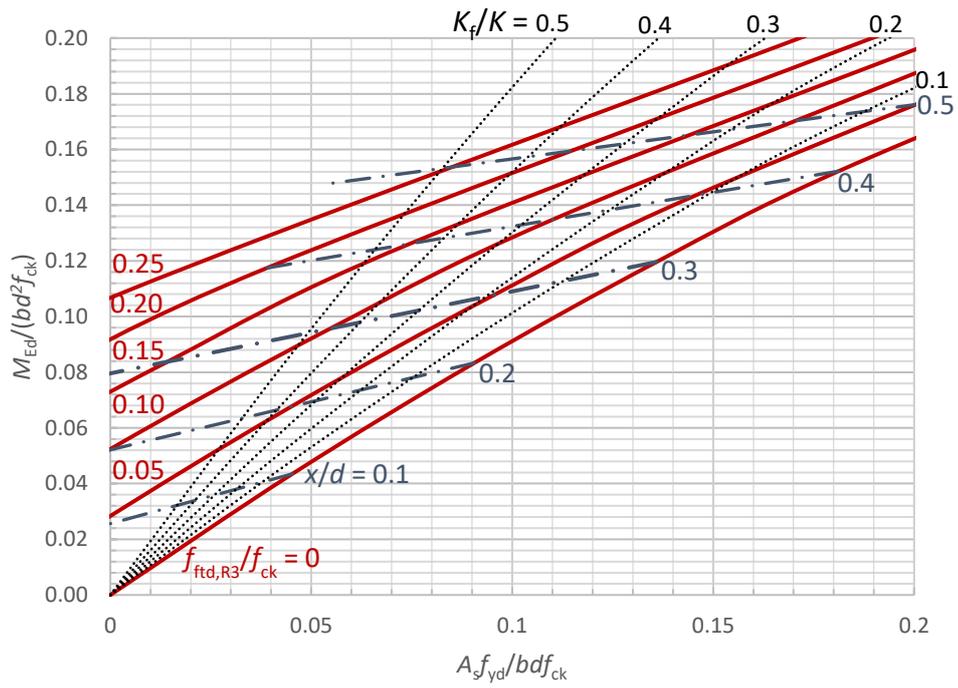


Chart C-3 Design for Bending ($d/h = 0.90$)

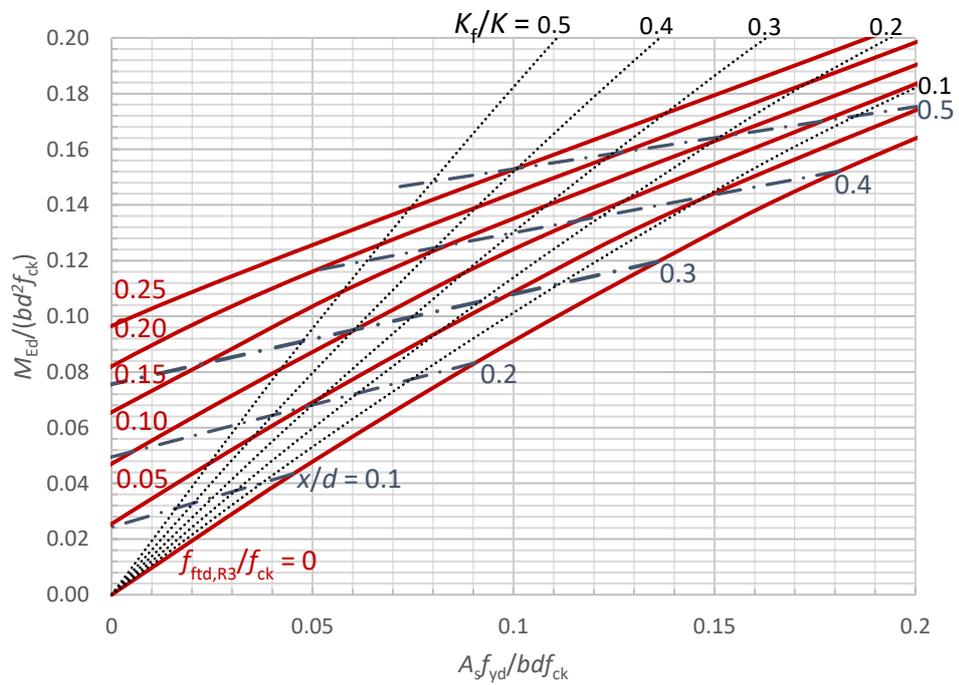


Chart C-4 Design for Bending ($d/h = 0.95$)

D. Design Charts for Bending with Axial Force

The N-M design charts are developed for symmetrically reinforced fibre concrete rectangular sections with $d/h = 0.80, 0.85, 0.90, \text{ and } 0.95$ (corresponding to $d'/h = 0.20, 0.15, 0.10, \text{ and } 0.05$, respectively) and $A_s f_{yd} / b d f_{ck} = 0 \text{ to } 1.0$. The method of strain compatibility is used with an equivalent rectangular stress block for concrete in compression with $\alpha_{cc} = 0.85$, and a constant stress distribution for fibre concrete in tension, as shown in Fig. D-1. **For each set values of d/h and $A_s f_{yd} / b d f_{ck}$ the value of $f_{ftd,R3} / f_{ck}$ is varied from 0 to 0.3, in steps of 0.1.**

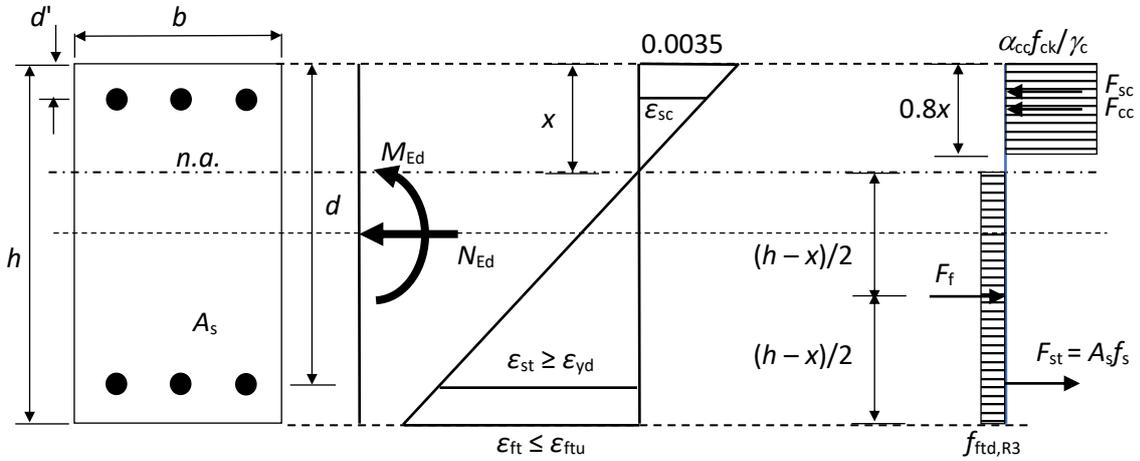


Fig. D-1 Strain and stress distributions

To derive the charts, the stress resultants acting on the section are first obtained as:

$$F_{cc} = b(0.8x) \left(\frac{0.85f_{ck}}{1.5} \right) = 0.453bx f_{ck} \quad (D.1)$$

$$F_{sc} = \frac{A_s}{2} f_s' = \frac{A_s}{2} E_s \left(\frac{x-d'}{x} \right) 0.0035 \leq \frac{A_s}{2} f_{yd} \quad (D.2)$$

$$F_{st} = \frac{A_s}{2} f_s = \frac{A_s}{2} E_s \left(\frac{d-x}{x} \right) 0.0035 \leq \frac{A_s}{2} f_{yd} \quad (D.3)$$

$$F_f = b(h-x) f_{ftd,R3} \quad (D.4)$$

Next, force and moment equilibrium give:

$$N = F_{cc} + F_{sc} - F_{st} - F_f \quad (D.5)$$

$$M = F_{cc} \left(\frac{h}{2} - 0.4x \right) + F_{sc} \left(\frac{h}{2} - d' \right) + F_{st} \left(d - \frac{h}{2} \right) - F_f \left(\frac{h}{2} - \frac{h-x}{2} \right) \quad (D.6)$$

Substituting the values of F_{cc} , F_{sc} , F_{st} , and F_f from (D.1) to (D.4) into Eqs. (D.5) and (D.6) and dividing all items in the resulting equations by bhf_{ck} and bh^2f_{ck} , respectively, gives:

$$\frac{N}{bhf_{ck}} = 0.453 \left(\frac{x}{h}\right) + \frac{A_s f_s'}{2bh f_{ck}} - \frac{A_s f_s}{2bh f_{ck}} - \left(1 - \frac{x}{h}\right) \frac{f_{ftd,R3}}{f_{ck}} \quad (D.7)$$

$$\frac{M}{bh^2f_{ck}} = 0.453 \left(\frac{x}{h}\right) \left(\frac{1}{2} - 0.4 \frac{x}{h}\right) + \frac{A_s f_s'}{2bh f_{ck}} \left(\frac{1}{2} - \frac{d'}{h}\right) + \frac{A_s f_s}{2bh f_{ck}} \left(\frac{d}{h} - \frac{1}{2}\right) - \left(1 - \frac{x}{h}\right) \frac{f_{ftd,R3}}{f_{ck}} \left(\frac{1}{2} \cdot \frac{x}{h}\right) \quad (D.8)$$

For a specified value of $\frac{x}{h}$, the values of $\frac{N}{bhf_{ck}}$ and $\frac{M}{bh^2f_{ck}}$ can be obtained for a given set values of $\frac{d}{h}$, $\frac{A_s f_{yd}}{bh f_{ck}}$, and $\frac{f_{ftd,R3}}{f_{ck}}$ from (d.7) and (D.8) respectively. By varying the value of $\frac{x}{h}$, the N - M design charts are obtained as shown in Charts D-1 to D-4. In each chart, the values of ε_{ft} corresponding to selective values of the design ultimate fibre strain ε_{ftu} . (SS 674, Figs. 3.2 and 3.3) are shown.

It should be noted that as fibre contribution in compression is not accounted for, the N_{Ed}/bhf_{ck} versus M_{Ed}/bh^2f_{ck} curve does not depend on the value of $f_{ftd,R3}/f_{ck}$ where the section is completely in compression, i.e., above the decompression point where $\varepsilon_{ft} = 0$.

Following the same procedure, the N - M design charts can be obtained for plain and lightly reinforced fibre concrete sections, as shown in Charts D-5(a), (b), (c) and (d), with $\alpha_{cc} = 0.6$, 0.65, 0.75 and 0.85, respectively.

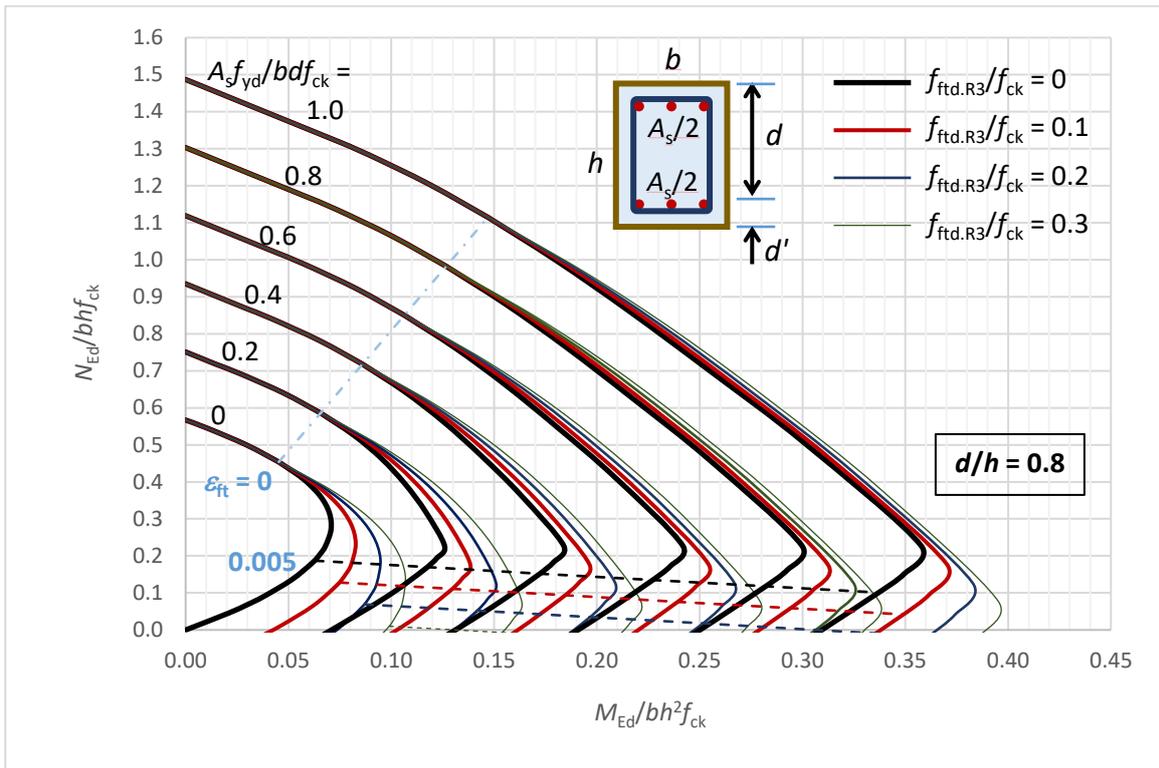


Chart D-1(a) N-M Diagram ($d/h = 0.80$)

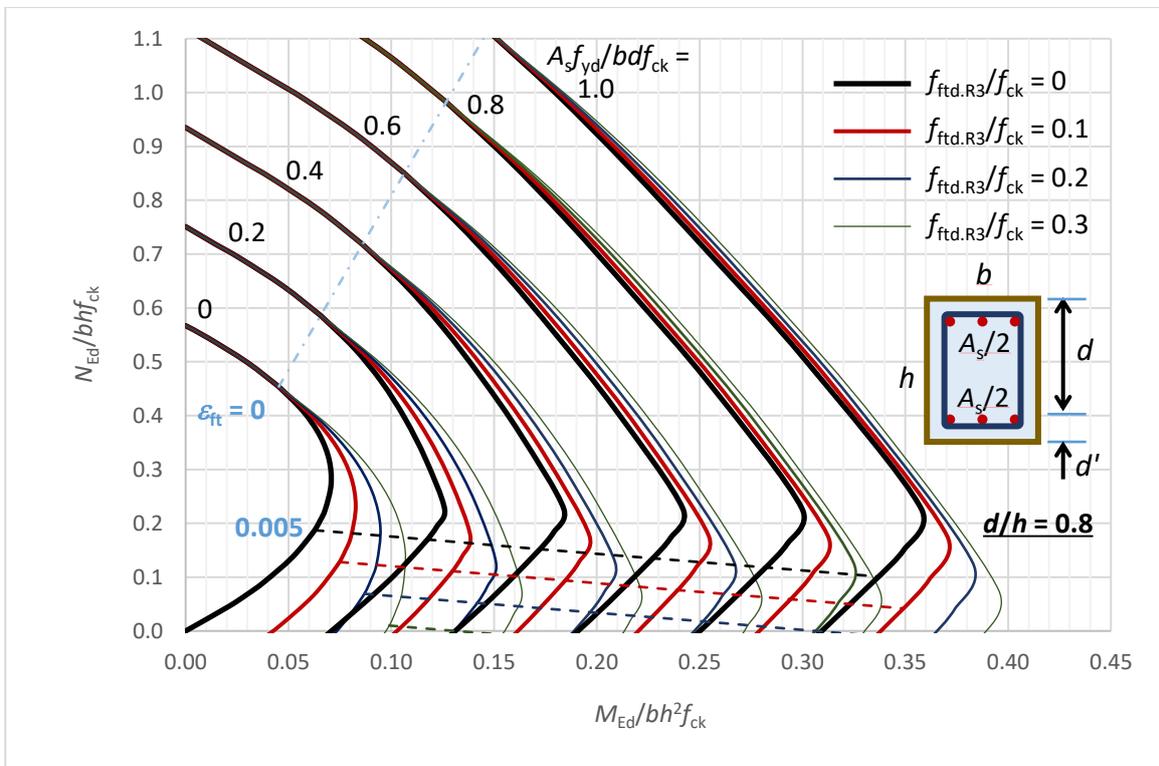


Chart D-1(b) N-M Diagram (*enlarged*) ($d/h = 0.80$)

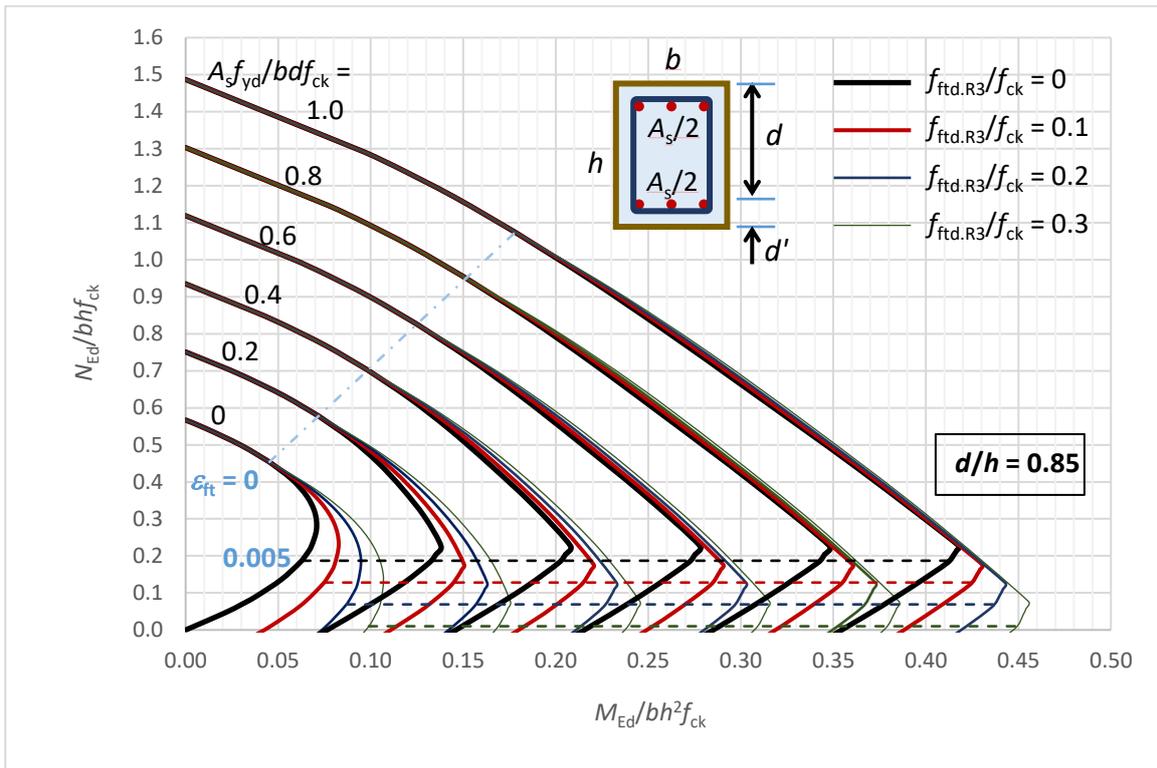


Chart D-2(a) N-M Diagram ($d/h = 0.85$)

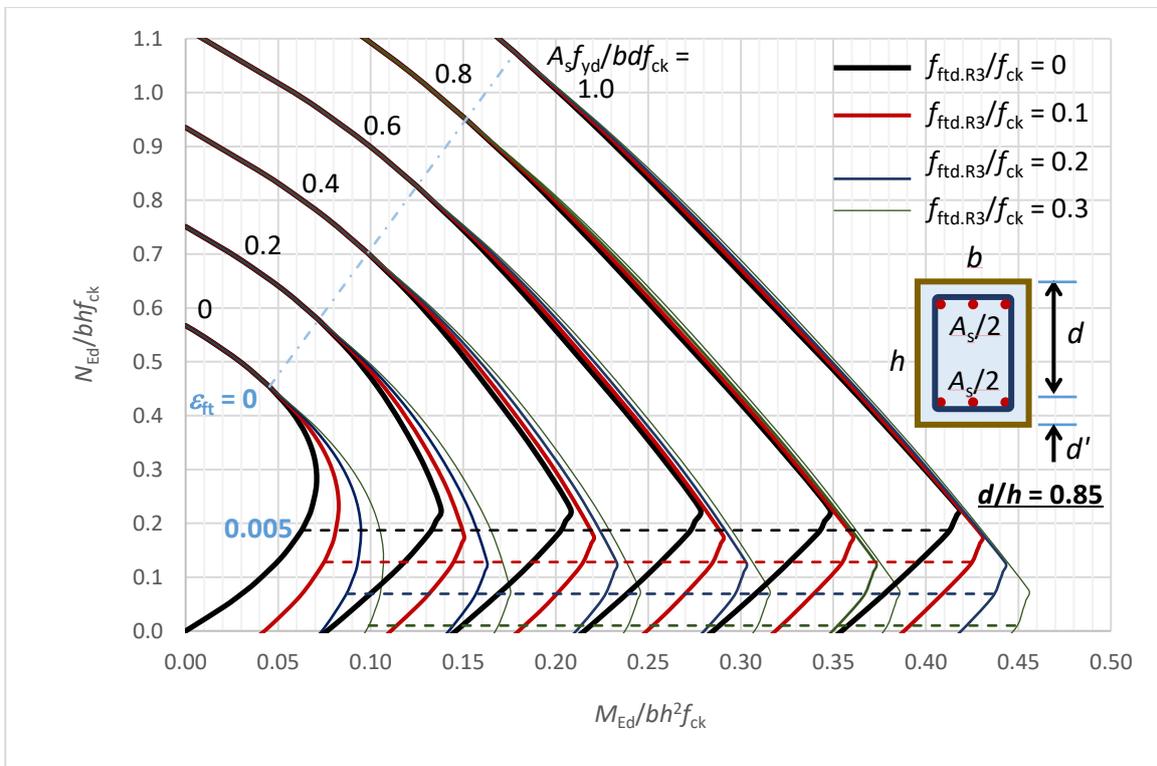


Chart D-2(b) N-M Diagram (*enlarged*) ($d/h = 0.85$)

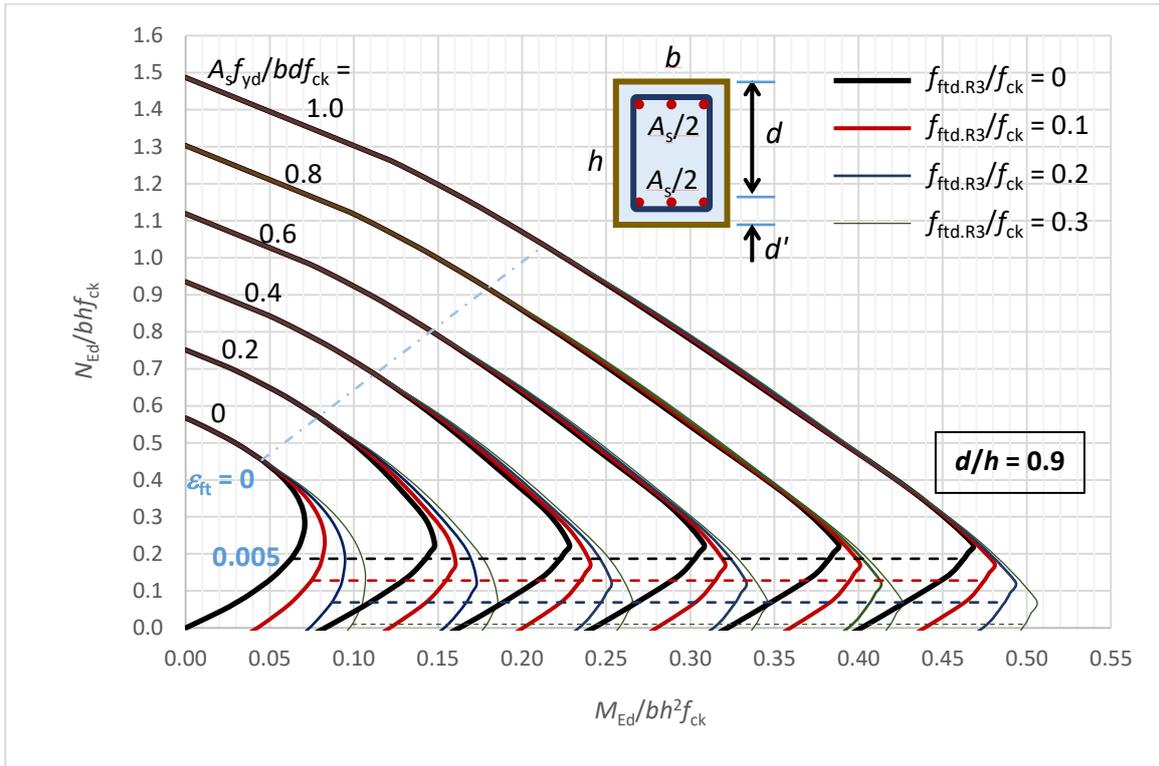


Chart D-3(a) N-M Diagram ($d/h = 0.90$)

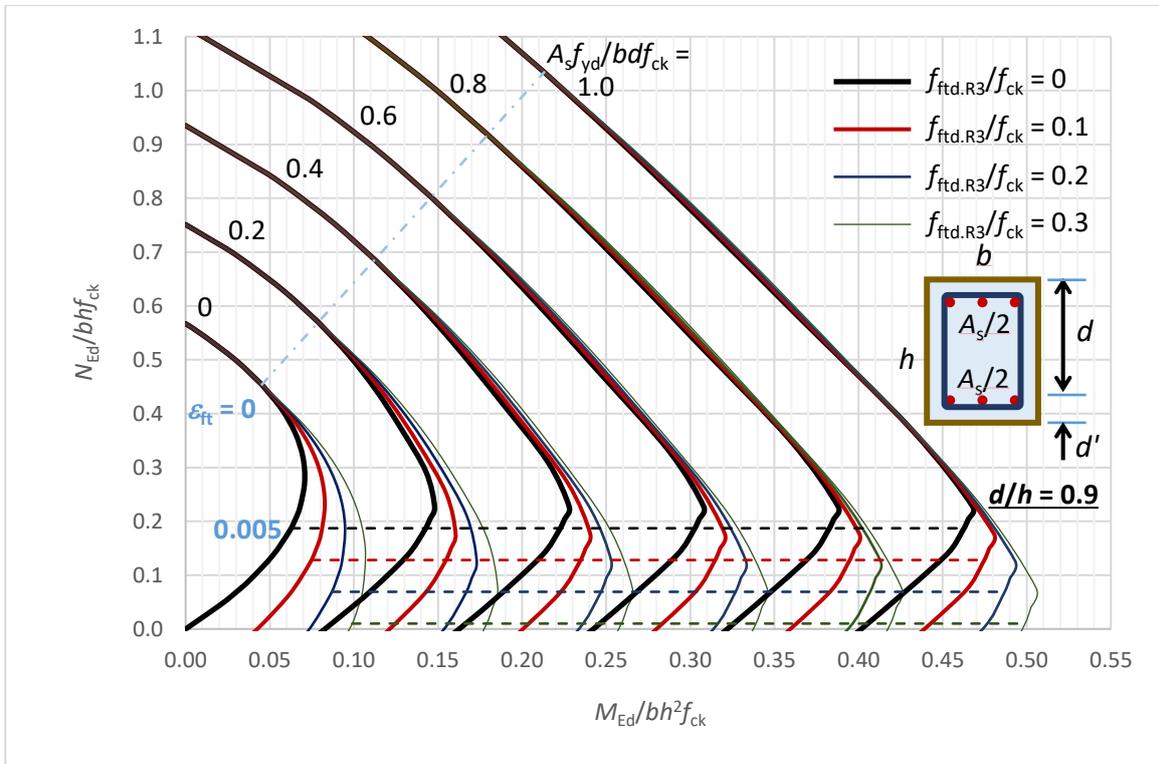


Chart D-3(b) N-M Diagram (*enlarged*) ($d/h = 0.90$)

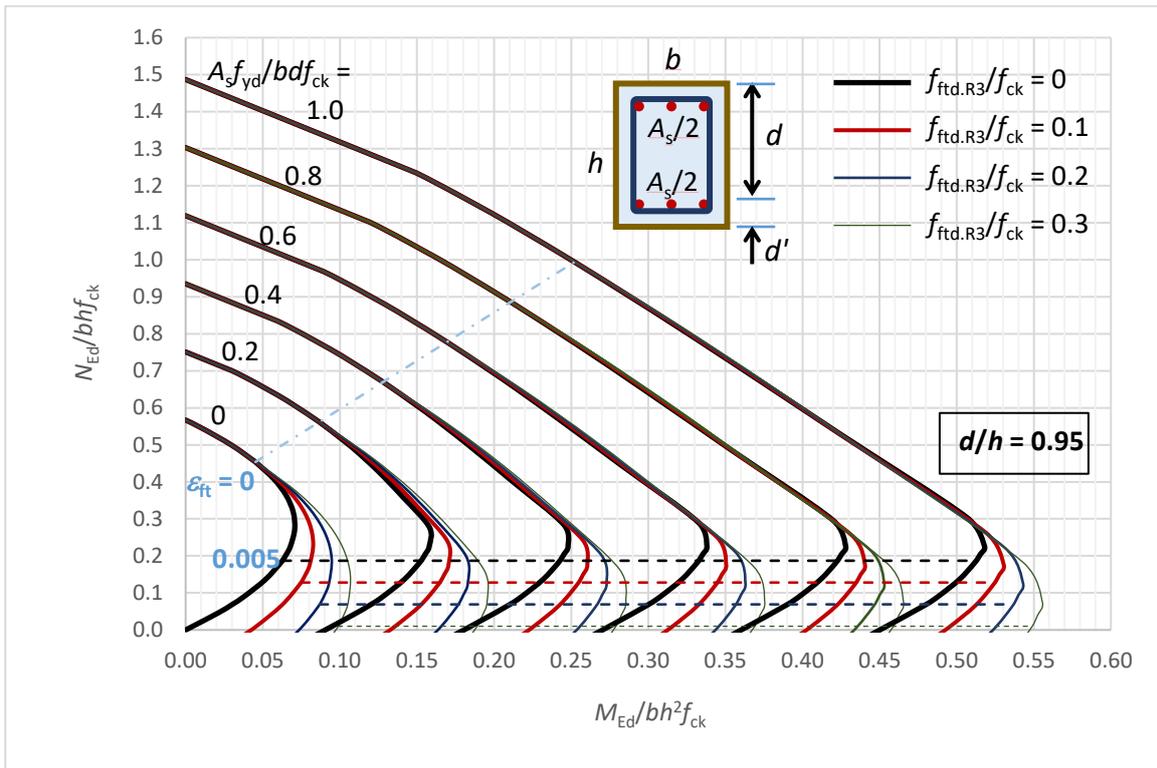


Chart D-4(a) N-M Diagram ($d/h = 0.95$)

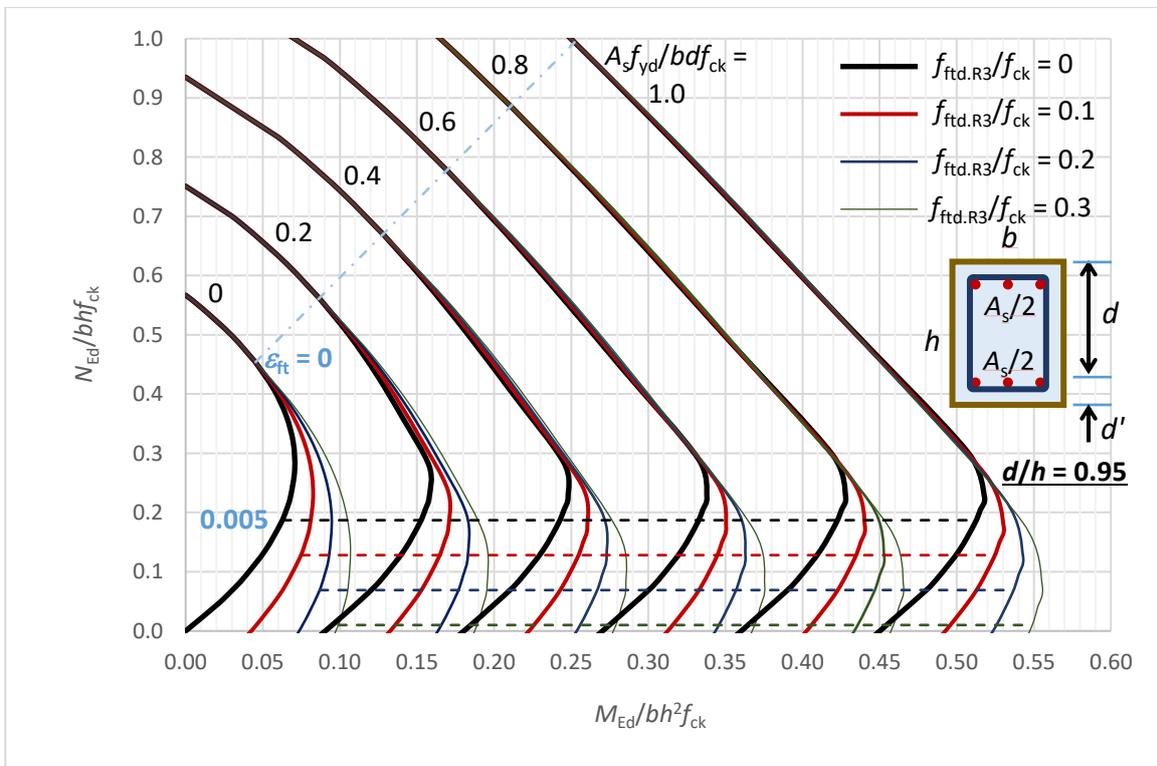


Chart D-4(b) N-M Diagram (*enlarged*) ($d/h = 0.95$)

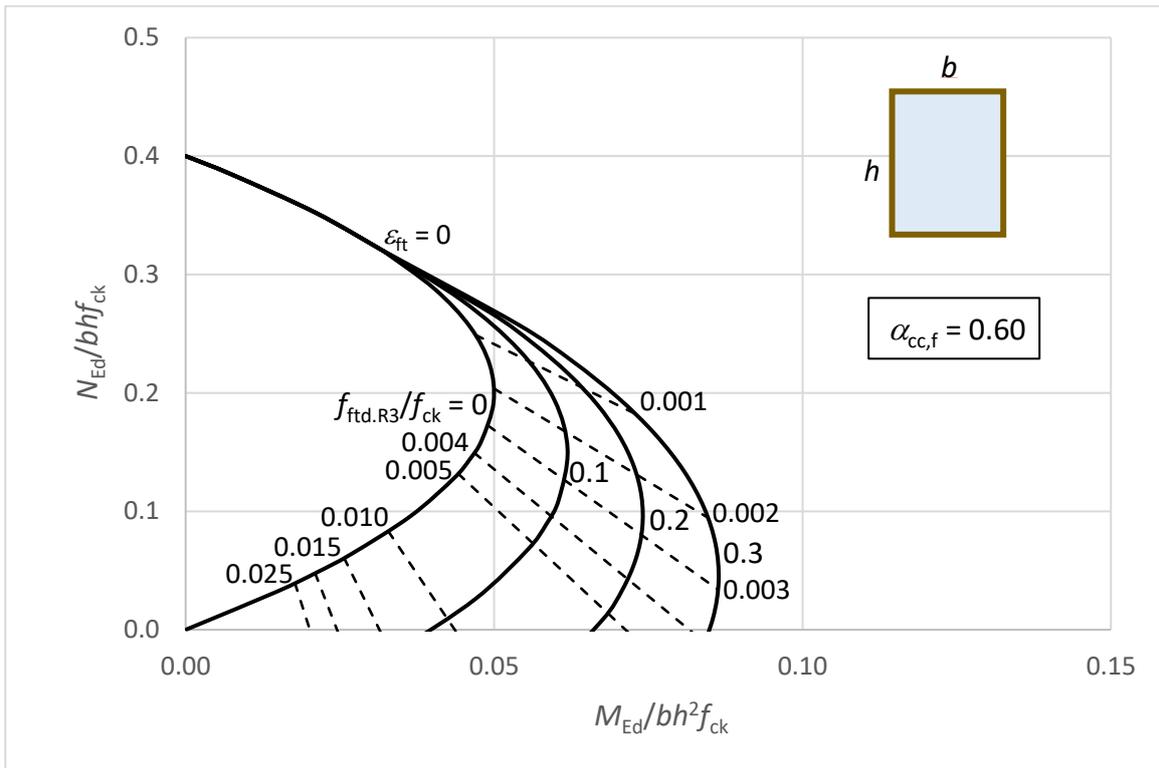


Chart D-5(a) N-M Diagram ($\alpha_{cc,f} = 0.60$)

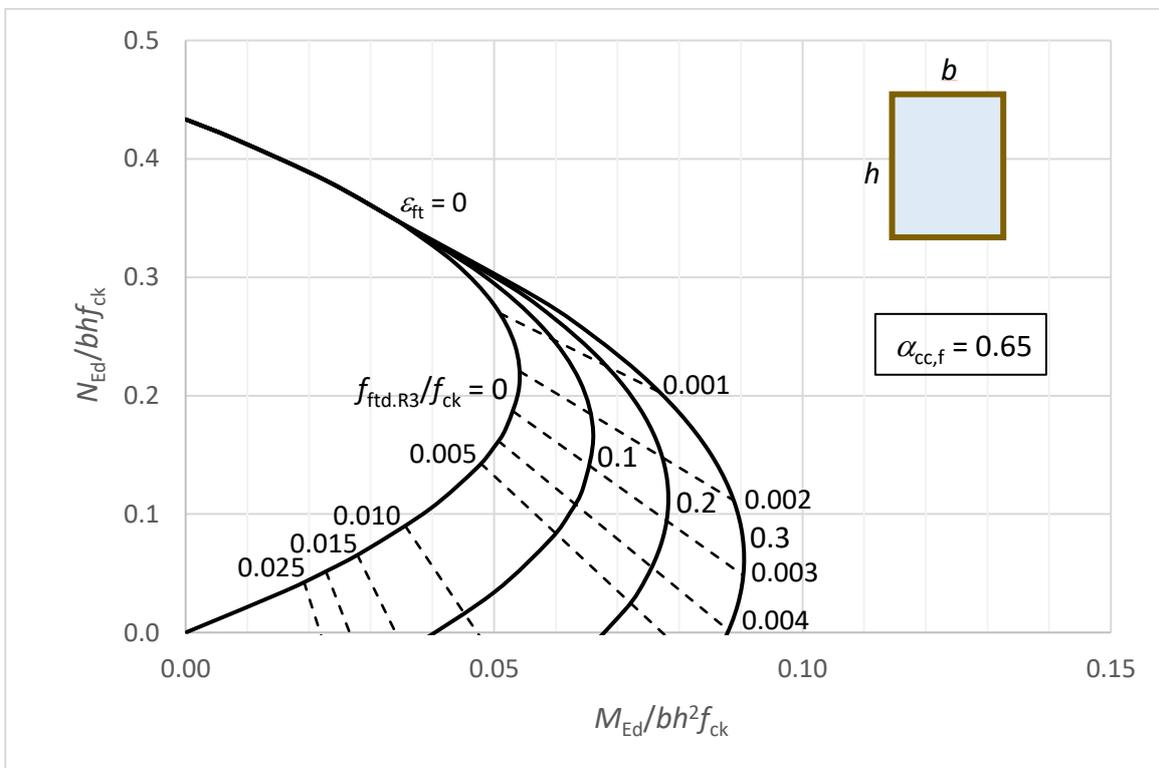


Chart D-5(b) N-M Diagram ($\alpha_{cc,f} = 0.65$)

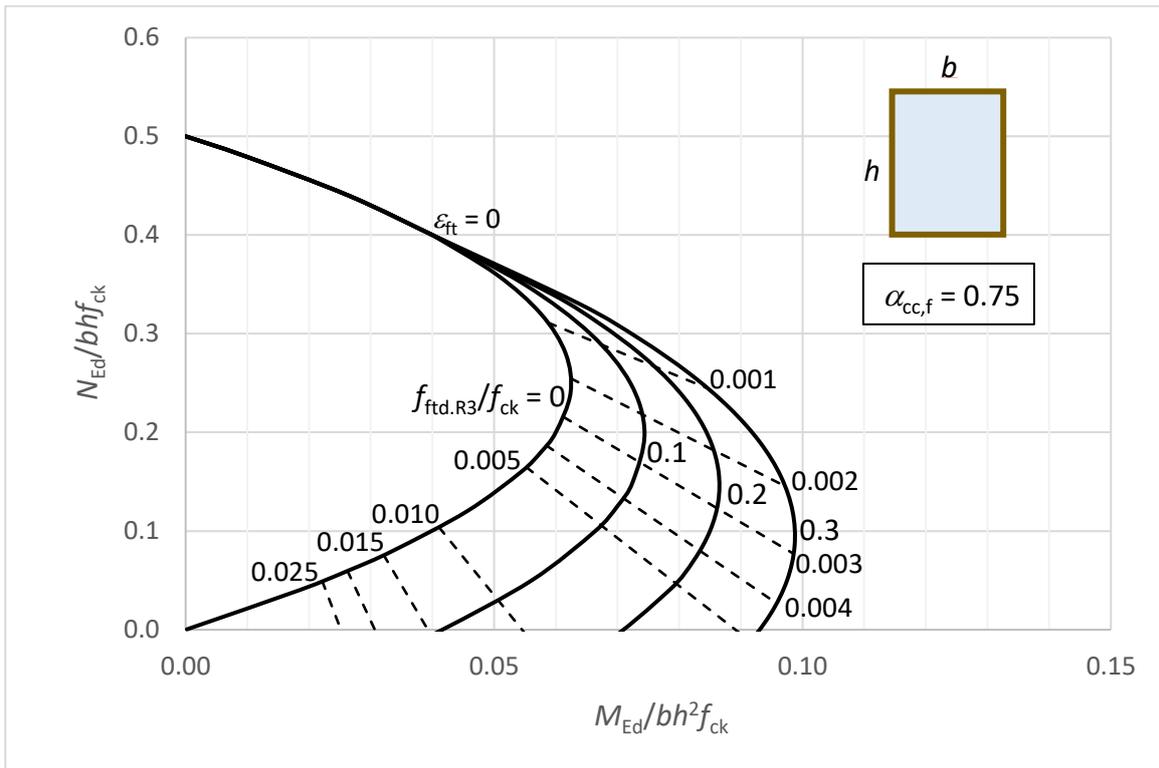


Chart D-5(a) N-M Diagram ($\alpha_{cc,f} = 0.75$)

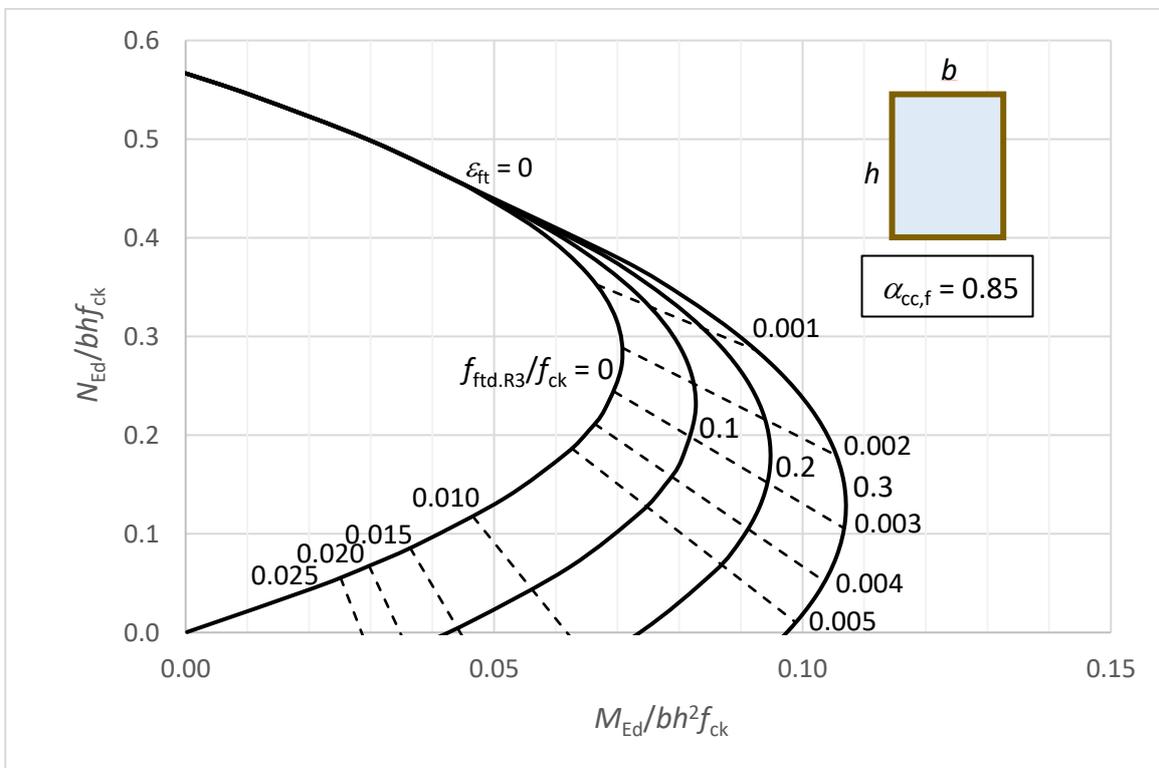


Chart D-5(d) N-M Diagram ($\alpha_{cc,f} = 0.85$)

E. Design Table and Chart for Shear

The design shear resistance, $v_{Rd,cf}$ (MPa), of fibre-reinforced concrete is given by:

$$v_{Rd,cf} = (0.18/\gamma_c) \cdot k \cdot [100\rho(1+7.5f_{ft,R3}/f_{ctk}) \cdot f_{ck}]^{1/3} \geq 0.035k^{3/2}f_{ck}^{1/2} \quad (\text{SS 674, Eq. 6.2})$$

where $f_{ft,R3} = 0.37 f_{R,3}$ (SS 674, Eq. 3.2).

The values of $v_{Rd,cf}$, including the lower bound values, for different values of f_{ck} and $f_{R,3}$, are tabulated in Table E-1 and shown graphically in Chart E-1, for the case where $k = 2$ and $\rho = 0.01$. For other cases, the values of $v_{Rd,cf}$ can be obtained by multiplying the values from the table or graph by the factors shown in the footnotes.

Table E-1 Values of $v_{Rd,cf}$ * (with $k = 2$, and $\rho = 0.01$)

f_{ck} (MPa) \ $f_{R,3}$ (MPa)	25	30	35	40	45	50
0	0.746	0.746	0.746	0.746	0.746	0.746
1	1.018	0.994	0.975	0.959	0.946	0.935
2	1.193	1.157	1.129	1.106	1.086	1.069
3	1.327	1.284	1.250	1.221	1.197	1.176
4	1.439	1.390	1.351	1.318	1.290	1.266
5	1.535	1.482	1.438	1.402	1.372	1.345
6	1.621	1.563	1.517	1.478	1.444	1.416
7	1.699	1.637	1.587	1.546	1.511	1.480
8	1.770	1.705	1.652	1.609	1.571	1.539
$v_{Rd,cf}(\text{min})^{**}$	0.495	0.542	0.586	0.626	0.664	0.700

*Multiply value by $(k/2)(100\rho)^{1/3}$ where $\rho \leq 0.02$, for $k < 2$ (or $d > 200$ mm) and/or $\rho \neq 0.01$

**Multiply value by $(k/2)^{3/2}$ for $k < 2$ (or $d > 200$ mm)

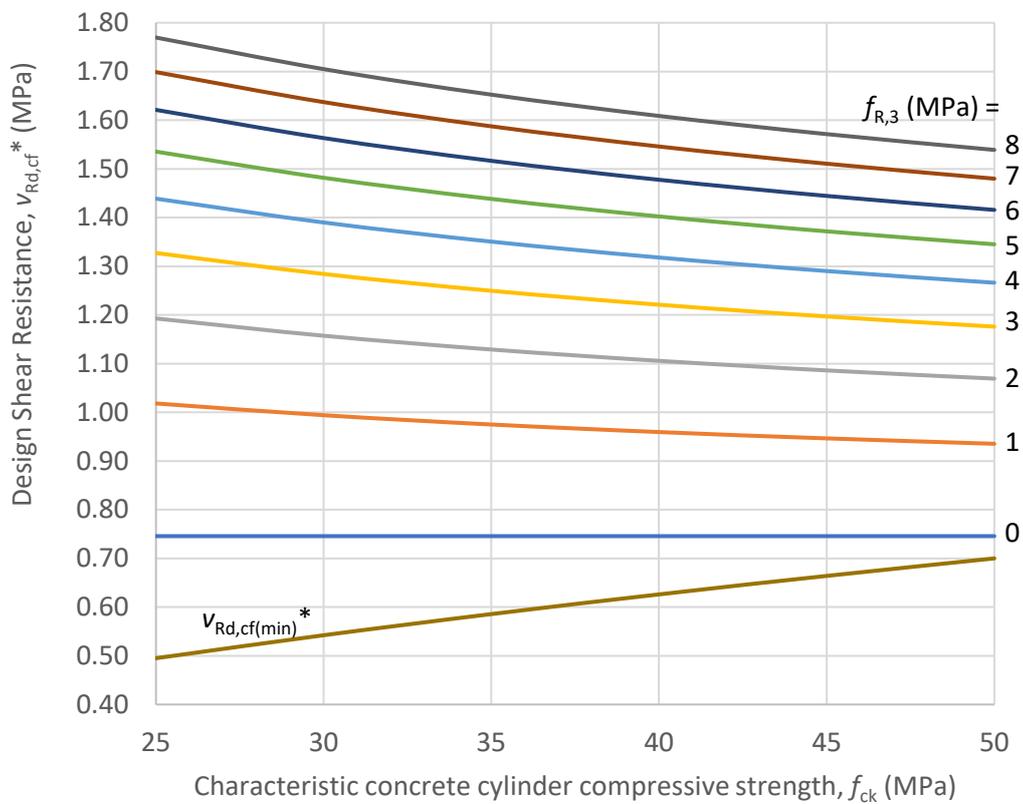


Chart E-1 Values of $v_{Rd,cf}$ (with $k = 2$, and $\rho = 0.01$)

NOTE:

*Multiply value by $(k/2)(100\rho)^{1/3}$ where $\rho \leq 0.02$, for $k < 2$ (or $d > 200$ mm) and/or $\rho \neq 0.01$;

**Multiply value by $(k/2)^{3/2}$ for $k < 2$ (or $d > 200$ mm)]

F. Charts for Crack Width Determination

The crack width is calculated as:

$$w_k = s_{r,max} \cdot (\varepsilon_{sm} - \varepsilon_{cm}) \quad (\text{SS 674, Eq. 7.5}) \quad (\text{F.1})$$

where

$$s_{r,max} = k_3 \cdot c + k_1 \cdot k_2 \cdot k_4 \cdot (1 - k_f) \cdot \frac{\phi}{\rho_{p,eff}} \quad (\text{SS 674, Eq. 7.8}) \quad (\text{F.2})$$

and

$$(\varepsilon_{sm} - \varepsilon_{cm}) = \frac{\sigma_s - (k_t + (1 - k_t) \cdot k_f) \cdot \frac{f_{ct,eff}}{\rho_{p,eff}} \cdot (1 + \alpha_e \cdot \rho_{p,eff})}{E_s} \geq 0.6 \cdot \frac{\sigma_s}{E_s}$$

(SS 674, Eq. 7.6) (F.3)

The relations among various terms in Eqs. (F.1) to (F.3) are shown in Design Chart F-1 as follows:

1st Quadrant, Q1 (top-right):

$(\varepsilon_{sm} - \varepsilon_{cm})$ versus $s_{r,max}$ for different values of w_k

2nd Quadrant, Q2 (top-left):

$(\varepsilon_{sm} - \varepsilon_{cm})$ versus $K_{f2} \cdot \frac{f_{ct,eff}}{\rho_{p,eff}} \cdot (1 + \alpha_e \cdot \rho_{p,eff})$ for different values of σ_s ;

where $K_{f2} = k_t + (1 - k_t) \cdot k_f$.

3rd Quadrant, Q3 (bottom-left):

$K_{f1} \cdot \frac{\phi}{\rho_{p,eff}}$ versus $K_{f2} \cdot \frac{f_{ct,eff}}{\rho_{p,eff}} \cdot (1 + \alpha_e \cdot \rho_{p,eff})$ for different values of s^* ;

where $K_{f1} = k_1 \cdot k_2 \cdot k_4 \cdot (1 - k_f)$ and $s^* = \frac{K_{f1}/K_{f2}}{f_{ct,eff} \cdot (1 + \alpha_e \cdot \rho_{p,eff})} \cdot \phi$

4th Quadrant, Q4 (bottom-right):

$K_{f1} \cdot \frac{\phi}{\rho_{p,eff}}$ versus $s_{r,max}$ for different values of $k_3 \cdot c$.

Note that following NA to SS EN 1992:2008 provisions for flexural cracking, $k_1 = 0.8$, $k_2 = 0.4$, $k_3 = 3.4$, and $k_4 = 0.425$. Also, for crack width calculation under sustained loading, $k_t = 0.4$. In such cases, the values of K_{f1} and K_{f2} are:

$$K_{f1} = 0.8 \cdot 0.4 \cdot 0.425 \cdot (1 - k_f) = 0.17 \cdot (1 - k_f)$$

$$K_{f2} = 0.4 + (1 - 0.4) \cdot k_f = 0.4 + 0.6 \cdot k_f$$

To determine the crack width using Chart F-1, follow the steps below:

1. Calculate the values of s^* and $K_{f1} \cdot \frac{\phi}{\rho_{p,eff}}$, and determine the corresponding point in Q3.
2. From Q3, project lines parallel to the axes to Q2 and Q4.
3. Determine the points on the projected lines corresponding to the calculated steel stress σ_s , and $k_3 \cdot c$, in Q2 and Q4, respectively.
4. From the obtained points in Q2 and Q4, project lines parallel to the axes to Q1 to obtain the value of w_k .

If the obtained value of w_k is larger than the limiting crack width w_{max} , repeat the above steps by choosing a smaller value of ϕ .

Chart F-1 can be used for sections with multiple layers by appropriately determining the value of $\rho_{p,eff}$ following the recommendations of PP 6687-1-2020.

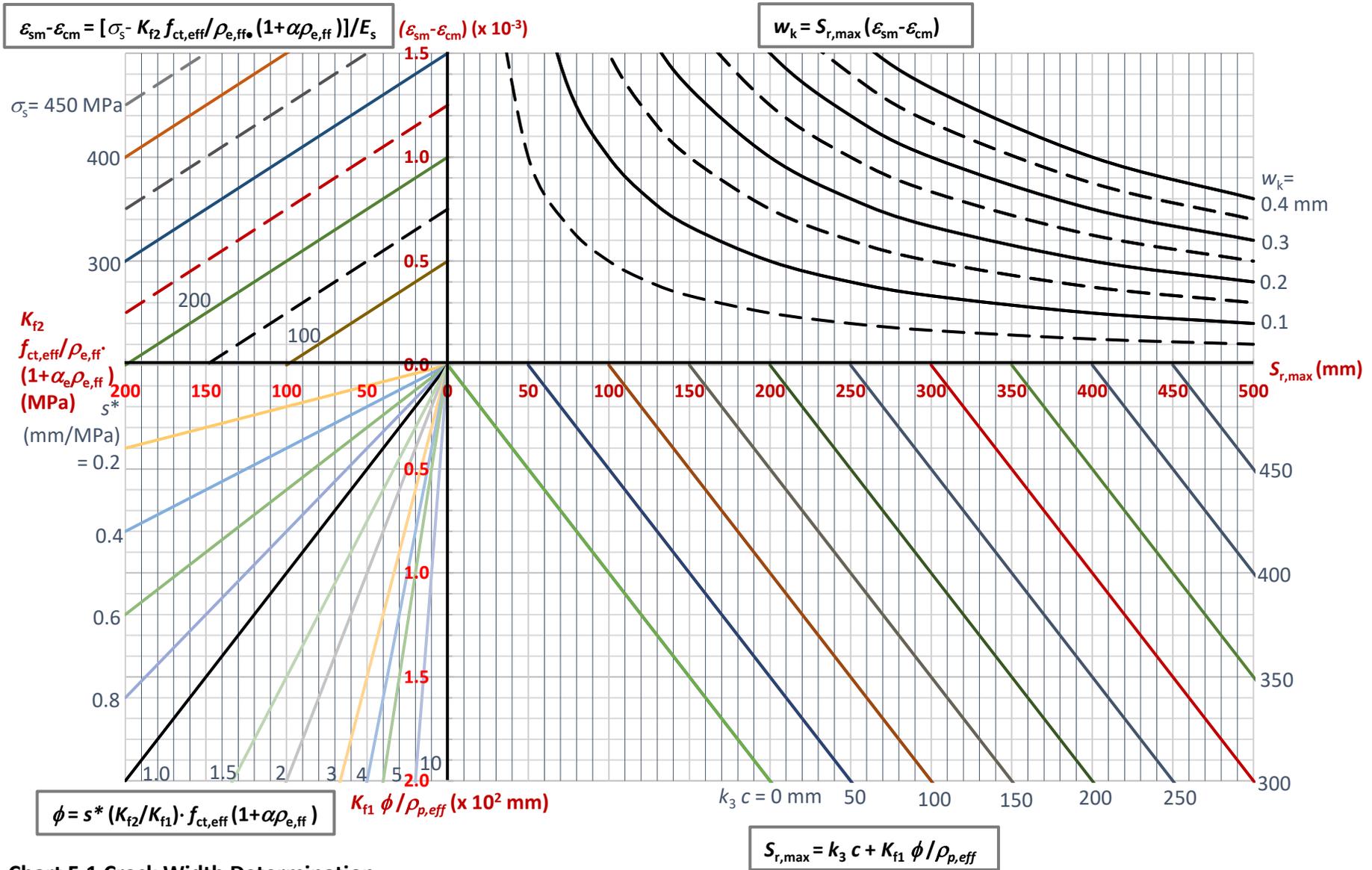


Chart F-1 Crack Width Determination

Supplementary Notes

S.1 Quality Assurance/Quality Control in Testing of Fibre Concrete to EN 14651

S1.1 Test Equipment

The test equipment must have a rigid loading frame and a load measuring range of between 3 to 300kN with a 2-3MN/mm deformation range (Zimmer and Reuter 2015). An example is shown in Fig. S1-1. Test equipment with a load measuring range of up to 600kN will be too large for testing the prism specimens to BS EN 14651.



Fig. S1-1 BS EN 14651 Beam Test Set-up

S1.2 Problems in Fibre Concrete Beam Test

When tested correctly with the appropriate test equipment in accordance with BS EN 14651 test method, the specimen will exhibit five values of flexural strength corresponding to crack-mouth opening displacement (CMOD) at limit of proportionality (LOP), 0.5 mm, 1.5 mm, 2.5 mm and 3.5 mm. Fig. S1-2 shows an acceptable result. Fig. S1-3 on the other hand shows an unacceptable result as only four values corresponding to CMOD at LOP, 0.5mm, 1.5mm and 2.5mm, were obtained. The last value corresponding to CMOD at 3.5mm is not shown on the curve because the aluminum or stainless steel plates holding the CMOD measuring device had fallen off from the prism specimen, causing the CMOD measurement to stop. This problem can be resolved by using better quality glue and ensuring that the specimen is dry before glueing the plate.



Fig. S1-2 Acceptable BS EN14651 Test Result

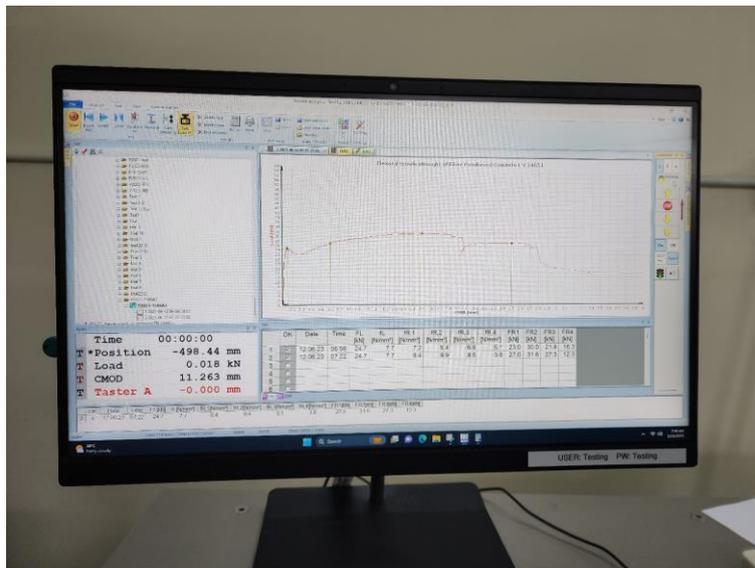


Fig. S1-3 Unacceptable BS EN 14651 Test Result

S1.3 Review of Test Results

For design calculations, the characteristic strengths shall be used. It is incorrect to use mean or average values. As an example, the characteristic strengths based on the results of 60 (n) samples taken from three batches of fibre concrete are determined following the steps below.

Step 1: Calculate the average value m_x for each desired parameter ($f_L, f_{R,1}, f_{R,2}, f_{R,3}, f_{R,4}, f_{ct}, f_{cu}$).

Step 2: Calculate standard deviation s_x for each parameter.

Step 3: Calculate coefficient of variance $V_x (= s_x / m_x)$ for each parameter.

Step 4: Determine the minimum and maximum values for each parameter.

Step 5: Calculate the characteristic value for each parameter using the following formula (*fib* Bulletin 83 (2017), Eq. 9-3):

$$X_k = m_x \{1 - k_n V_x\} \quad (S1.1)$$

where k_n is given as (*fib* Bulletin 83 (2017), Eq. 9-4):

$$k_n = t_{0.05} (1 + 1/n)^{0.5} \quad (S1.2)$$

with $t_{0.05}$ fractile of the t -distribution for the probability 0.05. Values for k_n for different number n of specimens are reported in Table 9-1 of *fib* Bulletin 83 (2017) and reproduced below in Table S1-1. The procedure assumes that the coefficient of variation V_x is unknown.

If test data are available from large production of a similar material (e.g. using the same fibre and the same fibre dosage), the coefficient of variation can be considered as known. In this case, the coefficient k_n is given as (*fib* Bulletin 83 (2017), Eq. 9-7):

$$k_n = u_{0.05} \{1 + 1/n\}^{0.5} \quad (S1.3)$$

with $u_{0.05}$ fractile of the standardized normal distribution for the probability 0.05. Values for k_n for different number n of specimens are reported in Table 9-2 of *fib* Bulletin 83 (2017) and reproduced below in Table S1-1. Values of k_n may also be specified by the project owner as part of their particular and/or project specifications.

Table S1-1: Values of k_n for the 5% characteristic value

No. of samples, n	V_x unknown	V_x known
3	3.37	1.89
4	2.63	1.83
5	2.34	1.80
6	2.18	1.77
8	2.01	1.74
9	1.96	1.73
10	1.92	1.72
12	1.87	1.71
15	1.82	1.70

In this example, the project owner specified the following: the average tensile splitting strength, the average limit of proportionality (LOP) value, the average $f_{R,1}$ value and the average $f_{R,3}$ value from tests on all three (3) batches, shall exceed the specified characteristic value by at least 1.70 times the standard deviation. As shown in Table S1-2, these requirements have been complied with.

With regards to the characteristic values, the Qualified Person (Design) may have different requirements from the Client. It is normal for the QP (Design) to have higher characteristic values than the Client. This is due to the QP (Design) having to satisfy additional structural requirements whereas the Client may just specify their minimum characteristic requirements. Both sets of requirements must be satisfied during the trial mix.

Last, in the example, the value of $f_{Rk,1}/f_{Lk}$ has to be at least equal to 0.5, in accordance with SS 674:2021. Also, the $f_{Rk,3}/f_{Rk,1}$ ratio required by the QP (Design) is 5.0/5.2 or 0.96. Both conditions are met with sufficient margins.

Table S1-2 Example with Sample Size $n = 60$

Parameter	f_l , MPa	$f_{R,1}$, MPa	$f_{R,2}$, MPa	$f_{R,3}$, MPa	$f_{R,4}$, MPa	f_{ct} , MPa ¹	f_{cu} , MPa ¹	
Average value, m_x	7.58	8.99	11.31	9.97	8.35	7.52	77.40	
Standard deviation, s_x	1.18	1.74	2.14	1.82	1.98	0.23	0.89	
Coefficient of variance, V_x	16%	19%	19%	18%	24%	3%	1%	
Minimum value	5.5	5.2	6.3	6.6	5	6.64	75.2	
Maximum value	10.4	13	16.7	14.7	14.5	8.01	79.4	
Achieved characteristic value, $X_k = m_x - 1.7 s_x$	5.6	6.0	7.7	6.9	5.0	7.1	75.9	
Required characteristic value (QP Design)	5.5	5.2	-	5.0	-	6.0	60	
Required characteristic value (Client)	5.5	5.2	-	3.1	-	4.5	60	
fib Model Code /EN 1992-1-1:2023 Classification								
f_{Lk}							5.6	
$f_{Rk,1}$							6.0	
$f_{Rk,3}$							6.9	
$f_{Rk,1}/f_{Lk}$							1.07	
$f_{Rk,3}/f_{Rk,1}$							1.15	
$\alpha_{cc,f}$ (Design Guide 2022, Table C3.3)							0.85	
Strength-Ductility Class							6d	

NOTE: f_{ct} = concrete splitting tensile strength (BS EN 12390-6);
 f_{cu} = concrete cube compressive strength (BS EN 12390-3).

References

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- BS EN 12390-6:2023. Testing hardened concrete – Tensile splitting strength of test specimens.
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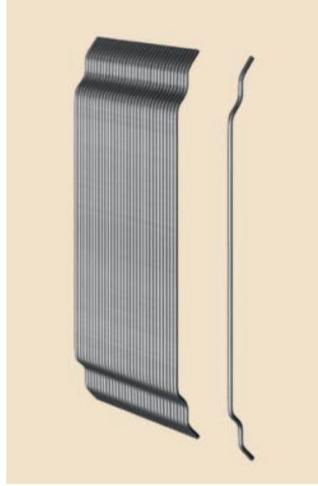
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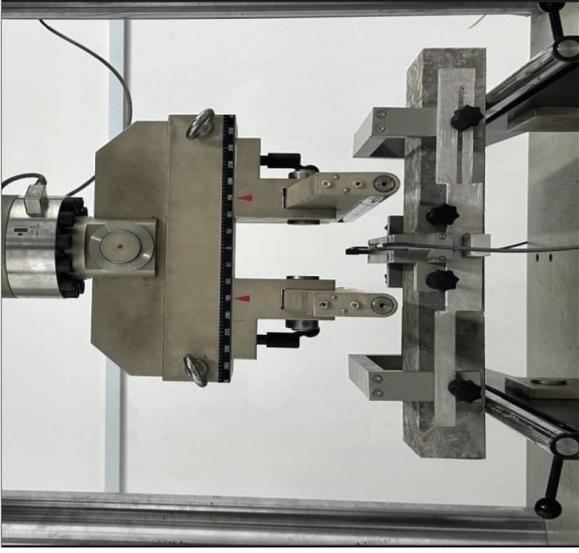
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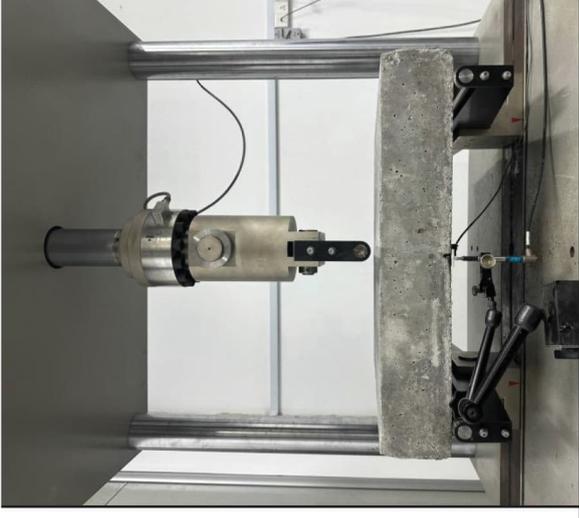
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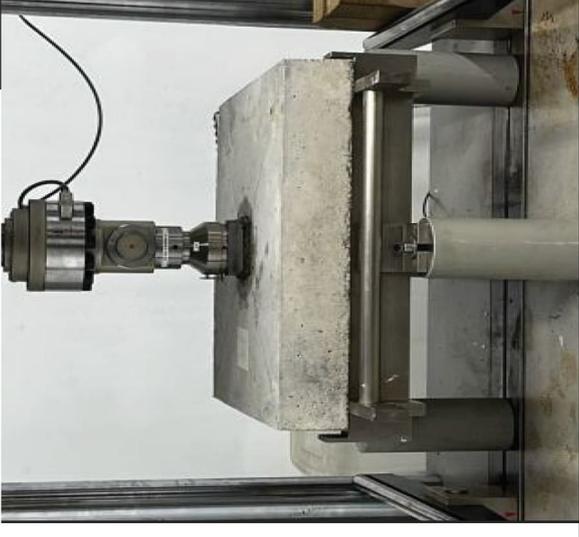
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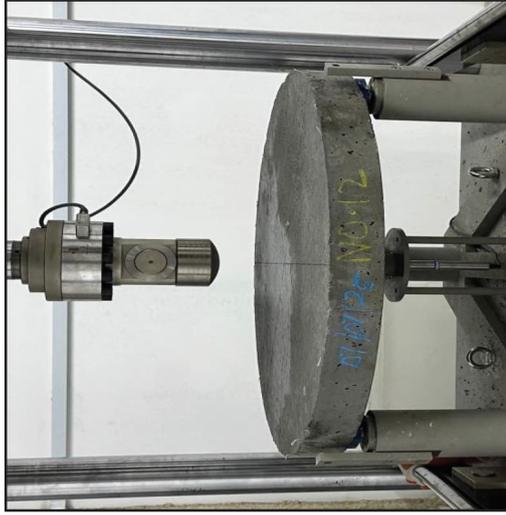
Part 3: Flexural Strength (First Peak, Ultimate and Residual) of Fibre Reinforced Beam Specimens (BS EN 14488-3–Method A)



Part 3: Flexural Strength (First Peak, Ultimate and Residual) of Fibre Reinforced Beam Specimens (BS EN 14488-3–Method B)



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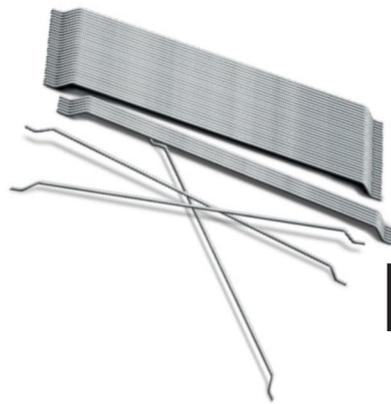
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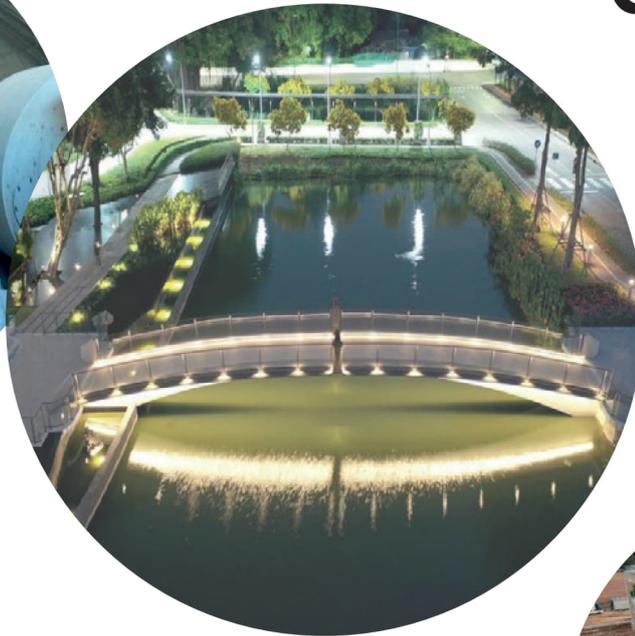


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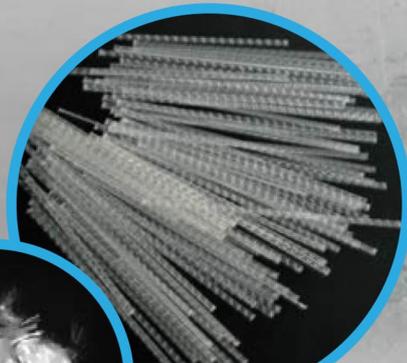
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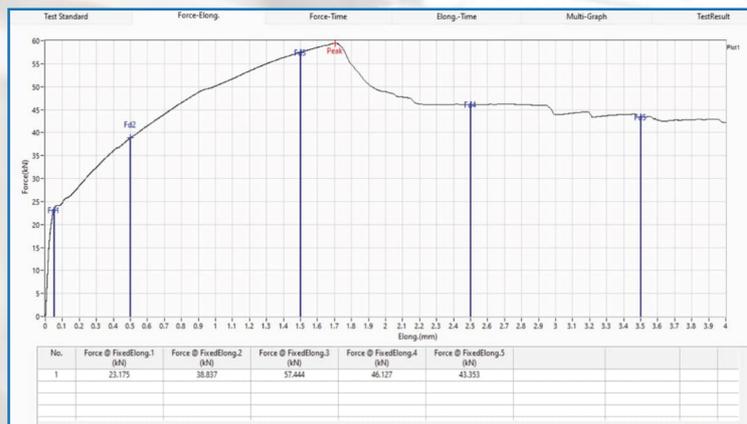
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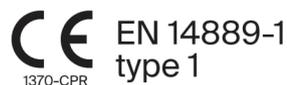
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