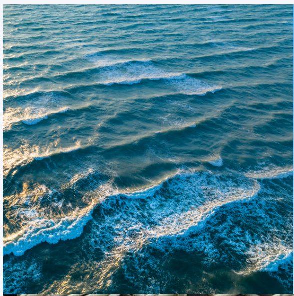

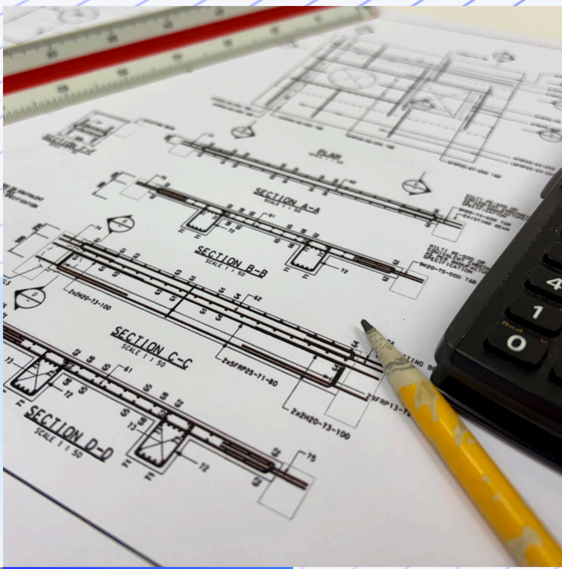

October 2025 Edition

DESIGN GUIDELINES FOR THE USE OF

Fibre-Reinforced Polymer (FRP) in Marine Structures

Civil Engineering Office
Civil Engineering and Development Department
The Government of the Hong Kong Special Administrative Region

CEDD

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FOREWORD

The marine environment in Hong Kong is characterized by high humidity, salinity and temperature variances, which imposes significant challenges on the durability of our marine structures. Traditional construction materials like steel are often susceptible to corrosion and deterioration, which may lead to costly maintenance. In recent years, Fibre-Reinforced Polymer (FRP) materials have emerged as a viable and innovative alternative, offering excellent corrosion resistance, high strength-to-weight ratio, and enhanced durability.

The Design Guidelines for the Use of FRP in Marine Structures present recommended standards and methodologies for the design of FRP-strengthening and FRP-reinforced concrete marine structures in Hong Kong. It consists of four Sections. Section 1 mainly covers design considerations and requirements for FRP-strengthening of existing marine structures with the use of externally bonded reinforcement (EBR); Section 2 mainly covers design considerations and requirements of FRP-reinforced concrete marine structures with the use of FRP reinforcement bars; and Section 3 & 4 set the material specifications, testing requirements, and quality assurance standards for FRP materials used in marine structures.

This document was developed by the Civil Engineering Office in conjunction with the Hong Kong Polytechnic University with reference to the latest local and overseas design publications and experiences, in consultation with other Government departments, engineering practitioners and professional bodies. An advisory committee was formed to oversee the development of the document. Many individuals and organizations made very useful comments, which have been taken into account in drafting the document. An independent review was undertaken by expert before the document was finalized. All contributions are gratefully acknowledged.

Practitioners are encouraged to comment at any time to the Civil Engineering Office on the contents of this document, so that improvements can be made to future editions.

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CONTENTS

Page No.

3. MATERIAL SPECIFICATION ON STRENGTHENING PRODUCTS FOR MARINE STRUCTURES

LIST OF TABLES

LIST OF FIGURES

ABBREVIATIONS

CF	Carbon Fibre
CFRP	Carbon Fibre-Reinforced Polymer
EBR	Externally Bonded Reinforcement
FRP	Fibre-Reinforced Polymer
GF	Glass Fibre
GFRP	Glass Fibre-Reinforced Polymer
IC	Intermediate Crack
RC	Reinforced Concrete
SLS	Serviceability Limit State
ULS	Ultimate Limit State

GLOSSARY OF TERMS

Adhesive

material that possesses enough adhesive strength to bond FRP reinforcement to a concrete surface.

Average value

a value provided by the manufacturer, less than or equal to the average test results of the specimens tested according to a specific method.

Batch

a batch is the quantity of GFRP reinforcing bars delivered to site under one delivery order, of one nominal diameter, and one production lot and produced by the same manufacturer. A batch shall not exceed linear length of 60,000 metres for all diameters.

Carbon fibre (CF)

fibre produced by heating organic precursor materials containing a substantial amount of carbon, such as rayon, polyacrylonitrile, or pitch in an inert environment.

Characteristic value

A value provided by the manufacturer, less than or equal to the average value minus three standard deviations of the specimens tested according to a specified method.

Externally bonded reinforcement (EBR)

adhesively bonded FRP reinforcement installed externally to a concrete surface.

Fabric reinforcement

Reinforcing fibres in fabric form.

Fibre-reinforced polymer (FRP)

fibre-polymer composite material comprising industrially manufactured fibres embedded in a polymer matrix.

Fibre-reinforced polymer (FRP) lamina

a single layer of composite material made by combining fabric reinforcement and saturating resin matrix.

Fibre-reinforced polymer (FRP) reinforcement

assembly of profiled or roughened fibre reinforced polymer bars, embedded in or connected to concrete members.

Fibre-reinforced polymer (FRP) system

composite comprising fibres with an accompanying adhesive material that is bonded to an adequately prepared concrete strata for the purpose of strengthening a structural concrete member, including CFRP system and GFRP system.

FRP jacket

closed FRP wrapping that is continuous along the strengthened member within the region to be strengthened.

FRP strip

unidirectional FRP reinforcement industrially prefabricated in various rectangular flat shapes used as EBR.

Fibre sheets

textile surface structure comprising dry parallel fibre bundles arranged in one or more directions.

Glass fibre (GF)

filament drawn from an inorganic fusion typically comprising silica-based material that has cooled without crystallizing.

Glass fibre reinforced polymer (GFRP)

fibre-polymer composite material comprising industrially manufactured glass fibres embedded in a polymer matrix.

Glass fibre reinforced polymer (GFRP) reinforcing bar

a straight or bent element with a solid, round cross-section in its straight portion, featuring surface enhancements designed to create a mechanical interlock with concrete.

Interlaminar shear

force tending to produce a relative displacement along the plane of the interface between two laminae.

In-situ lay-up

manufacturing process where dry fabric fibre reinforcement is impregnated on site with a saturating resin matrix and then cured in place.

Lay-up

process of placing reinforcing material and resin system in position for molding.

Nominal bar diameter

a standard diameter of a bar

Ply

a single layer of fibre reinforcement.

1Production lot

determined by the manufacturer, as any batch of bar produced from start to finish with the same constituent materials used in the same proportions without changing any production parameter, such as cure temperature or line speed.

Roving

a parallel bundle of continuous yarns, tows, or fibres with little or no twist.

Tow

an untwisted bundle of continuous filaments.

Wet lay-up

a manufacturing process where dry fabric fibre reinforcement is impregnated on-site with a saturating resin matrix and then cured in-place.

GLOSSARY OF SYMBOLS

Latin upper case letters

A	Length of major axis (for elliptical cross-section)
A_{c}	Cross-sectional area of concrete
$A_{c,eff}$	Effective concrete area
A_{f}	Cross-sectional area of EBR, the cross-sectional area of longitudinal FRP reinforcement within the effective tension area
A_{fa}	Cross-sectional area of FRP reinforcement
A_{fc}	Cross-sectional area of longitudinal reinforcement in the compression chord; confinement reinforcement
A_{ft}	Cross-sectional area of longitudinal reinforcement in the tension chord; transverse reinforcement
$A_{ft,min}$	Minimum transverse reinforcement
A_{fl}	Cross-sectional area of longitudinal FRP reinforcement
A_{fw}	Cross-sectional area of shear reinforcement
$A_{fw,min}$	Minimum cross-sectional area of shear reinforcement
$A_{f,min}$	Minimum cross-sectional area of FRP reinforcement
$A_{f,min,h}$	Minimum amount of horizontal reinforcement
$A_{f,min,v}$	Minimum amount of vertical reinforcement
$A_{f,req}$	Cross-sectional area of required longitudinal FRP reinforcement
$A_{f,req\ span}$	Amount of required flexural non-prestressed reinforcement
$A_{f,v}$	Amount of vertical reinforcement
$A_{f,web}$	Reinforcement area to be provided in the web over a height limited by the neutral axis and the centroid of reinforcement with a spacing not exceeding 300 mm, to control cracking
A_{k}	Area enclosed by centrelines of connecting walls of cross-section
A_{s}	Total cross-sectional area of the longitudinal steel reinforcement
A_{sa}	Cross-sectional area of ordinary reinforcement
A_{sc}	Cross-sectional area of steel compression reinforcement
A_{se}	Converted area of tensile reinforcement considering the contribution of EBR
A_{si}	Cross-sectional area of longitudinal reinforcement located at the i^{th} layer
A_{sl}	Effective area of tensile reinforcement
A_{st}	Cross-sectional area of steel tension reinforcement
B	Length of minor axis (for elliptical cross-section)
C_{C}	Long term strength reduction factor to account for creep
C_{e}	Long term strength reduction factor to account for environmental condition
C_{t}	Long term strength reduction factor to account for temperature condition
D	Diameter of the cross-section (for circular section)

D_f	FRP stress distribution factor accounting for the non-uniform distribution of FRP stress along the shear crack at the ultimate limit state with shear failure
D_{pile}	Diameter of pile at footing base
D_{upper}	Largest value of the upper sieve size D in an aggregate for the coarsest fraction of aggregates in the concrete permitted by the specification of concrete
E_2	Slope of the linear second portion of stress-strain model for FRP-confined concrete
E_c	Modulus of elasticity of unconfined concrete
$E_{c,eff}$	Effective modulus of elasticity of concrete accounting for creep deformations
E_{cm}	Secant modulus of elasticity of concrete
$E_{c,eff}$	Effective modulus of elasticity of concrete accounting for creep deformations
E_f	Mean modulus of elasticity in longitudinal direction of EBR CFRP
E_{fR}	Design value of modulus of elasticity of FRP reinforcement
E_{fwR}	Design value of modulus of elasticity of FRP shear reinforcement
E_s	Design value of modulus of elasticity of ordinary reinforcing steel
EI	Bending stiffness
F_{Ed}	Design value of actions
I_{cr}	Second moment of area of cracked concrete section
I_e	Second moment of area for calculation of deflection
I_{e+}	Second moment of area at location of maximum positive moment for calculation of deflection
I_{e1-}	Second moment of area at location of maximum negative moment at the near end of the span for calculation of deflection
I_{e2-}	Second moment of area at location of maximum negative moment at the far end of the span for calculation of deflection
I_f	Second moment of area of the bars about the centroid of the cross-section
L	Bond length of FRP shear reinforcement
L_a	Anchorage length of FRP flexural reinforcement
L_d	Distance from the maximum moment section or from the edge of the pure bending zone of the beam/slab to the end of EBR
L_e	Effective bond length of EBR
L_l	Overlap length of EBR
M_1	Lesser end moment on a compression member
M_2	Greater end moment on a compression member
M_{cr}	Cracking moment of the section in presence of the simultaneous axial force N_{Ed}, which may be calculated on the basis of the concrete tensile strength f_{ctm} assuming linear stress distribution and neglecting any contribution from reinforcement
M_{Ec}	Maximum moment in member due to service loads
M_{Ed}	Design value of the applied internal bending moment

$M_{\mathrm{Ed,min}}$	Bending strength of the section with $A_{\mathrm{f,min}}$ in presence of the simultaneous axial force N_{Ed}
M_{s}	Applied moment due to all sustained loads plus the maximum moment induced in a fatigue loading cycle
M_{sa}	Internal bending moment at the serviceability limit state
M_{sc}	Slab moment that is resisted by the column at a joint
M_{Rd}	Moment capacity
$M_{\mathrm{R,min}}$	Bending strength of the section with $A_{\mathrm{f,min}}$ in presence of the simultaneous axial force N_{Ed}
$N_{\mathrm{c,0}}$	Design value of axial compressive strength at zero eccentricity (compression positive)
$N_{\mathrm{c,max}}$	Maximum value of axial compressive strength of member (compression positive)
N_{Ed}	Design value of the applied axial force
$N_{\mathrm{t,max}}$	Maximum value of axial tensile strength of member
R	Equivalent radius of FRP-confined concrete
R_{d}	Design value of the resistance
S_{DL}	Characteristic dead load effects
S_{LL}	Characteristic live load effects
T_{a}	Characteristic value of the tensile pullout strength of a single nail or bolt anchored in concrete in the direction normal to the tension face of the member
$V_{\mathrm{Rd,f}}$	Contribution of the FRP shear reinforcement to the shear strength
$V_{\mathrm{Rd,f0}}$	Shear contribution of the FRP shear reinforcement to the shear strength of the reinforced concrete (RC) member without considering the effect of shear interaction

Latin lower case letters

a_{l}	Distance by which moment curve is shifted to account for shear effect
$a_{\mathrm{s}i}$	Distance from the point of the resultant force of the longitudinal steel reinforcement located at the i^{th} layer to the extreme compression face
a'	Distance from the compression face to the centroid of steel compression reinforcement
b	Overall width of the cross-section
b_0	Perimeter of critical section for two-way shear in slabs and footings
b_1	Dimension of the critical section b_0 measured in the direction of the span for which moments are determined
b_2	Dimension of the critical section b_0 measured in the direction perpendicular to b_1
$b_{\mathrm{eff,s}}$	Effective slab width
b_{f}	Width of the EBR in the direction perpendicular to the fibre orientation
b'_{f}	Flange width of T-section
$b_{\mathrm{min}}, b_{\mathrm{max}}$	Minimum and maximum side lengths of the section (sub-section)
b_{t}	Breadth of the section at the tension face

b_{w}	Minimum width of the cross-section; Width of the web of T-shaped section; Dimension of the cross-section in the direction perpendicular to h
b_{wa}	Minimum width of the cross-section between tension and compression chords and neutral axis
c	Concrete cover of reinforcement (to the surface of the bar)
c_{b}	Lesser of: (a) concrete cover of reinforcement (to the surface of the bar), and (b) one-half the centre-to-centre spacing of bars being developed
$c_{\mathrm{min,bc}}$	Minimum concrete cover c due to bond requirements and control of cracking due to thermal cycling
$c_{\mathrm{min,dur}}$	Minimum concrete cover c due to durability requirement
c_{s}	Clear distance between parallel reinforcement bars
d	Effective depth of steel tension reinforcement (i.e. distance from the compression face to the centroid of steel tension reinforcement)
d_{c}	Effective depth of a cross-section
d_{dg}	Size parameter describing the crack and the failure zone roughness taking account of concrete type and its aggregate properties
d_{f}	Effective depth of FRP tension reinforcement (i.e., distance from the compression face to the centroid of FRP tension reinforcement)
d_{fl}	Distance from the compression face to the lower end of the FRP shear reinforcement
d_{ft}	Distance from the compression face to the top end of the FRP shear reinforcement
e_{a}	Additional load eccentricity
e_0	Equivalent load eccentricity
e_1, e_2	Load eccentricities at the two ends of a column, where e2 always has the larger absolute value of the two and is always taken to be positive
e_{i}	Initial load eccentricity
f_{Ack}	Characteristic compressive strength of the adhesive
f_{Atk}	Characteristic tensile strength of the adhesive
f_{anch}	Contribution by additional anchorage of mechanical fasteners to the characteristic intermediate crack (IC) debonding stress
$f_{\mathrm{bd,100a}}$	Long term bond strength of FRP reinforcement
f_{bfRd}	Design bond strength of the anchorage
f_{cd}	Design value of concrete compressive strength
$f_{\mathrm{cd,c}}$	Design compressive strength of FRP-confined concrete
f_{ck}	Characteristic concrete cylinder compressive strength at age t_{ref}
f_{ctd}	Design value of the tensile strength of concrete
f_{cm}	Mean concrete cylinder compressive strength at age t_{ref}
f_{ctm}	Mean axial tensile strength of concrete at age t_{ref}
$f_{\mathrm{ct,eff}}$	Mean value of the tensile strength of the concrete effective at the time when cracking may first be expected to occur
f_{dbic}	Characteristic intermediate crack (IC) debonding stress
f_{fd}	Tensile design strength in the GFRP longitudinal reinforcement for columns corresponding to a strain of 0.01

$f_{\mathrm{frpd,e}}$	Maximum tensile stress in EBR
f_{ftd}	Design tensile strength of FRP reinforcement
f_{ftk0}	Characteristic tensile strength of FRP reinforcement at the rupture strain $\varepsilon_{\mathrm{ftk0}}$
$f_{\mathrm{ftk,100a}}$	Characteristic long term tensile strength of FRP reinforcement
f_{fud}	Design long-term tensile strength of EBR
f_{fuk}	Characteristic short-term tensile strength of EBR
f_{fwd}	Effective stress of EBR intersected by the shear crack at the ultimate limit state
$f_{\mathrm{fwk,100a}}$	Characteristic long term tensile strength of FRP shear reinforcement
f_{fwRd}	Design tensile strength of FRP shear reinforcement
f_{yd}	Design yield strength of steel tension reinforcement
f'_{yd}	Design yield strength of steel compression reinforcement
f_{yk}	Characteristic value of yield strength of reinforcement or, if yield phenomenon is not present, the characteristic value of 0.2% proof strength
f_{ywd}	Design yield strength of steel shear reinforcement
h	Dimension of the cross-section in the plane of bending under consideration (for rectangular section)
h_{c}	Overall depth of a cross-section or of a part of a cross-section
h_{f}	Height of the FRP reinforcement bonded on the web of member
h'_{f}	Flange thickness of T-section
$\mathrm{h_{fe}}$	Effective height of EBR
i	Radius of gyration
k_{el}	Effective length factor for compression members
$k_{\mathrm{s\sigma}}$	Shape factor for the ultimate compressive strength of FRP-confined concrete
$k_{\mathrm{s\varepsilon}}$	Shape factor for the ultimate axial strain of FRP-confined concrete
l	Length
l_0	Effective length of column
l_{bd}	Design value of anchorage length of reinforcing FRP
$l_{\mathrm{bd,tot}}$	Design anchorage length measured along the centre line of bars with bends and hooks in tension
l_{s}	Actual lap length
l_{sd}	Design value of lap length
m	Number of layers of longitudinal steel bars arranged along the direction of h
n	Axial compression ratio of column
n_{a}	Total number of nails or bolts used in the shear span (from the section considered to the end of FRP strip)
n_{f}	Number of layers of fibre sheets
r	Radius of the cross-section (for circular section)
r_{b}	Bending radius of bent bar
r_{c}	Radius of the rounded corners (for rectangular section)
r_{cs}	Ratio of rise to the side length for expanded rectangular cross-sections

r_s	Radius of the circle formed by the geometric centres of the longitudinal steel bars
s	Spacing of the shear reinforcement or confinement reinforcement measured along the longitudinal axis of the member
$s_{bu,max}$	Maximum longitudinal spacing of bent-up bars
s_f	Centre-to-centre spacing of FRP strips
$s_{l,max}$	Maximum longitudinal spacing of shear assemblies/stirrups
$s_{l,surf,max}$	Maximum spacing of surface reinforcement in beams with downstand
$s_{max,col}$	Maximum spacing of transverse reinforcement along the column
$s_{stir,max}$	Maximum spacing for torsion assemblies / stirrups
$s_{slab,max}$	Maximum spacing of bars for slabs
$s_{tr,max}$	Maximum transverse spacing of shear legs
t	Length over which the support reaction is distributed
t_{eff}	Effective wall thickness
t_f	Thickness of the cross-section of EBR
t_{fl}	Thickness of a single layer of FRP
t_{ref}	Age of concrete at which the concrete strength is determined in days
u	Perimeter of concrete cross-section, having area A_c
u_k	Perimeter of the area A_k
υ	Strength reduction factor for concrete cracked due shear or other actions
$w_{k,cal}$	Calculated crack width
$w_{lim,cal}$	Limiting crack width to be compared with the calculated crack width $w_{k,cal}$
x_{sb}	Depth of the equivalent stress block for concrete in compression
x_{sb}	Depth of the compression zone assuming a stress block
x_u	Depth of the neutral axis at ultimate limit state
y_t	Distance from centroidal axis of gross section, neglecting reinforcement, to tension face
z	Lever arm for the shear stress calculation

Greek letters

α	Inclination of reinforcement across interface
α_1	Stress factor of the equivalent stress block for concrete in compression
α_{f}	Angle of inclination of fibres in the FRP reinforcement to the longitudinal axis of the member
$\alpha_{\mathrm{FRP,th}}$	Coefficient of thermal expansion of FRP reinforcement
β_1	Depth factor of the equivalent stress block for concrete in compression
β_{dns}	Ratio of maximum factored sustained axial load to maximum factored axial load associated with the same load combination
β_{L}	FRP bond length factor
β_{j}	Confinement stiffness ratio of FRP-confined concrete
β_{jb}	Lower limit of confinement stiffness ratio for sufficiently-confined concrete
β_{w}	FRP-to-concrete width ratio factor
γ_{BA}	Partial factor for the bond strength between EBR and concrete
γ_{C}	Partial factor for concrete
γ_{FRP}	Partial factor for FRP reinforcing material
γ_{f}	Partial factor for the tensile strength of EBR
γ_{fa}	Factor used to determine the fraction of M_{sc} transferred by slab flexure at slab-column connections
γ_{sv}	Parameter to account for the variation in stiffness along the length of the flexural member
δ	Moment magnification factor used to reflect effects of member curvature between ends of a compression member
ε_{c}	Axial compressive strain of concrete
$\varepsilon_{\mathrm{cu}}$	Ultimate compressive strain of concrete
$\varepsilon_{\mathrm{cu,c}}$	Design ultimate axial strain of FRP-confined concrete
$\varepsilon_{\mathrm{fRd}}$	Strain of FRP reinforcement at design tensile strength f_{fRd}
$\varepsilon_{\mathrm{ftk0}}$	Rupture strain of FRP reinforcement
$\varepsilon_{\mathrm{ftk,100a}}$	Long term rupture strain of FRP reinforcement
$\varepsilon_{\mathrm{fuk}}$	Characteristic ultimate strain of EBR
$\varepsilon_{\mathrm{fud}}$	Long-term design strain of EBR
$\varepsilon_{\mathrm{fwRd}}$	Strain of FRP shear reinforcement at design tensile strength f_{fwRd}
$\varepsilon_{\mathrm{ru}}$	Design ultimate strain of the FRP jacket in the hoop direction
$\varepsilon_{\mathrm{ruk}}$	Characteristic ultimate strain of the FRP jacket in the hoop direction
ε_{t}	Transition strain in stress-strain curve of FRP-confined concrete
η	Amplification factor for the load eccentricity to account for the slenderness effect
η_{BA}	Environmental reduction factor applied to the bond strength between EBR and concrete
η_{cc}	Factor to account for the difference between the undisturbed compressive strength of a cylinder and the effective compressive strength developed in the structural member
ζ	Time-dependent factor for sustained load

ξ_b	Ratio of the compression depth to the depth of cross-section at balanced strain condition
θ	Angle between the compression field and the member axis
λ	Ratio between the bond length and the effective bond length of FRP shear reinforcement
$\lambda_{\triangle}$	Multiplier used for additional deflection due to long-term effects
μ	Ratio between the shear contribution of steel shear reinforcement and the shear contribution of FRP strips when they are both fully mobilised
μ_a	Frictional coefficient between two rough concrete surfaces
ρ	Reinforcement ratio
ρ_{fb}	Reinforcement ratio producing balanced strain conditions
ρ_{lf}	Longitudinal reinforcement ratio for FRP reinforcement
ρ_w	Shear reinforcement ratio
$\rho_{w,min}$	Minimum shear reinforcement ratio
$\rho_{w,req}$	Required shear reinforcement ratio
$\rho_{w,stir}$	Minimum ratio of shear and torsion reinforcement in the form of stirrups
σ_c	Axial compressive stress of concrete
σ_f	Serviceability value of stress in the FRP reinforcement, determined on the basis of a cracked section
$\sigma_{f,lim}$	Limiting value of the serviceability stress in the FRP reinforcement in order to comply with a given limiting crack width
$\sigma_{f,md}$	Design maximum tensile stress of FRP for the ultimate limit state of flexural failure of member
$\sigma_{f,max}$	Maximum stress reached in the FRP intersected by the critical shear crack
$\sigma_{fe,m1}$	Assumed tensile stress of FRP when the concrete at the extreme compression face reaches ultimate strain
σ_{ftd}	Design value of stress in the FRP reinforcement to develop the full sectional strength
σ_s	Serviceability value of steel stress, determined on the basis of a cracked section
σ_{si}	Stress of longitudinal reinforcement located at the i^{th} layer
$\sigma_{s,lim}$	Limiting value of the serviceability steel stress in order to comply with a given limiting crack width
τ_{ave}	Average interfacial shear strength of the lap joint
τ_{max}	Maximum shear bond strength
τ_{Rd}	Design shear resistance of the member without shear strengthening
$\tau_{Rdc,min}$	Minimum shear stress resistance allowing to avoid a detailed verification for shear (average shear stress over a cross-section)
$\tau_{Rd,c}$	Shear stress resistance of members without shear reinforcement (average shear stress over a cross-section)
$\tau_{Rd,f}$	Shear resistance of a member with FRP reinforcement
$\tau_{Rd,FRP}$	Design shear resistance of a section with FRP shear strengthening
$\tau_{t,Rd}$	Torsional shear stress resistance

$\tau_{\mathrm{t,Rd,sw}}$	Torsional shear stress resistance governed by design strength of shear reinforcement
$\tau_{\mathrm{t,Rd,sl}}$	Torsional shear stress resistance governed by design strength of longitudinal reinforcement
$\tau_{\mathrm{t,Rd,max}}$	Torsional shear stress resistance governed by crushing of the concrete in the compression field
ν	Strength reduction factor for concrete cracked due to shear or other actions
ϕ_{f}	Diameter of FRP reinforcement bar
$\emptyset_{\mathrm{l}}$	Diameter of longitudinal bars in column
$\emptyset_{\mathrm{l,max}}$	Maximum diameter of longitudinal bars in column
ϕ_{trans}	Diameter of transverse bars
ψ_{f}	Reduction factor accounting for the unfavourable effect of shear interaction between internal steel shear reinforcement and external FRP shear reinforcement
ψ_{t}	Factor used to modify anchorage length for casting location in tension

1. FRP-STRENGTHENING OF EXISTING MARINE STRUCTURES

1.1 Purpose and Scope

BS EN 1992 is referred to in Port Works Design Manual (PWDM) for the design of reinforced concrete structures. This document applies to the strengthening of existing marine concrete structures assessed in accordance with EN 1992-1-1:2023 and PWDM, making use of fibre reinforced polymer (FRP) reinforcement which is externally bonded to the surface of existing structures. Worked examples are provided in **Annex A** to illustrate the application of the design methods.

The externally bonded reinforcement (EBR) may be in the form of:
—prefabricated carbon FRP (CFRP) or glass FRP (GFRP) strips;
— in-situ lay-up carbon fibre (CF) or glass fibre (GF) sheets.

NOTE 1 For assessment of existing structures, Annex I of EN 1992-1-1:2023 shall be considered unless specifically omitted or supplemented in this document.

1.2 Terms, definitions and symbols

For the purposes of this document, the terms, definitions and symbols are given in sections of abbreviations, glossary of terms and glossary of symbols.

1.3 Basis of design

(1) Partial factors for EBR and its bond strength with concrete (i.e., γ_{f} and γ_{BA}) given in **Clause J.4 of EN 1992-1-1:2023** shall be applied.

NOTE γ_{f} is the partial factor for the tensile strength of EBR, and γ_{BA} is the partial factor for the bond strength between EBR and concrete.

(2) The load considerations refer to **Section 5 of PWDM Part 1-General Design Considerations for Marine Works**.

(3) To prevent collapse of the structure in the event of FRP failure, strengthening limits are imposed so that without the FRP strengthening system, the structure is still able to carry a certain level of loads as given below:

$$(R_{\mathrm{d}})_{\mathrm{existing}} \geq (1.1S_{\mathrm{DL}} + 0.75S_{\mathrm{LL}})_{\mathrm{new}} \tag{1}$$

where

$(R_d)_{existing}$ is the design value of the resistance of the un-strengthened existing member, determined by **Clause 8 of EN 1992-1-1:2023**;

S_{DL} is the characteristic dead load effect;

S_{LL} is the characteristic live load effect;

The subscript "new" denotes the new load effects applied to the strengthened member.

(4) The following points may be considered when selecting between CFRP and GFRP strengthening systems:

— GFRP strengthening systems are superior to CFRP strengthening systems in terms of impact resistance and deformation capacity;
— CFRP strengthening systems are superior to GFRP systems in terms of fatigue resistance and stiffness;
— CFRP should not be placed in direct contact with steel;
— when GFRP strengthening systems are used, the glass fibres shall be properly protected by a durable resin matrix and shall not be directly exposed to alkaline and acidic environments.

1.4 Materials

1.4.1 General

(1) FRP strengthening systems suitable for design in accordance with this document shall comply with **Section 3 of this document**.

(2) This document provides design rules for structural members strengthened with FRP reinforcement within the following limits of declared properties:

— interlaminar shear strength of FRP strips determined according to EN ISO 14130 shall be equal to or larger than the adhesive bond strength for any system;
— mean modulus of elasticity of CFRP strips: $E_f \geq 150{,}000$ MPa;
— elastic stiffness per unit width of CF sheets: $E_f \cdot A_f / b_f \geq 20$ kN/mm;
— mean modulus of elasticity of GFRP strips: $E_f \geq 40{,}000$ MPa;
— elastic stiffness per unit width of GF sheets: $E_f \cdot A_f / b_f \geq 12$ kN/mm;
— total GF cross-section per unit width of GF sheets in the total of all layers determined in the direction of the tension action effect applied to the system: 200 mm^2/m $\leq A_f / b_f \leq$ 3,600 mm^2/m;
— requirements for CF sheets and the adhesive follow those given in **Clause J.5.1(3) of EN 1992-1-1:2023**.

(3) **Clause J.5.1(4) of EN 1992-1-1:2023** is adopted.

1.4.2 Properties

(1) The following properties of FRP strips and fibre sheets are required for design of EBR strengthening systems in accordance with this standard:

f_{fuk}	characteristic short-term tensile strength of the EBR;
η_f	environmental reduction factor applied to the tensile strength of the EBR;
η_{BA}	environmental reduction factor applied to the bond strength between FRP and concrete;
E_f	mean value of modulus of elasticity of the EBR in the longitudinal direction;
ε_{fuk}	characteristic ultimate strain;
A_f	cross-sectional area.

(2) The cross-sectional area of FRP strips is determined following **Clause J.5.2(2) of EN 1992-1-1:2023**.

(3) For fibre sheets, the thickness t_f, which is an effective value, has to be determined considering the number of layers as follows:

$$t_f = {n_f}^{k_f} \cdot \frac{A_f}{b_f} \tag{2}$$

where

n_f	is the number of layers;
A_f/b_f	is the cross-sectional area of the fibres per meter of a single layer of fibre sheet;
k_f	=0.85 for $n_f \geq 3$, or 1 otherwise.

1.4.3 Design assumptions

(1) The value of design long-term tensile strength of the EBR (FRP strips and fibre sheets) shall be taken as:

$$f_{fud} = \frac{\eta_f \cdot f_{fuk}}{\gamma_f} \tag{3}$$

where η_f is an environmental reduction factor applied to the tensile strength of the EBR for the relevant exposure conditions.

(2) η_f and η_{BA} may be taken as the values in **Table 1** unless more accurate information is available based on test data for the EBR.

Table 1 — Environmental reduction factors

Exposure condition	Fibre type/ bond	Value of η_f or η_{BA}
Aerated zone – XS1	Glass	0.65
	Carbon	0.85
	Bond between EBR and concrete	0.92
Submerged zone – XS2 Tidal and splash zones – XS3	Glass	0.50
	Carbon	0.83
	Bond between EBR and concrete	0.85

(3) The long-term design strain of EBR, $\varepsilon_{\mathrm{fud}}$, shall be determined by:

$$\varepsilon_{\mathrm{fud}} = \frac{f_{\mathrm{fud}}}{E_{\mathrm{f}}} \tag{4}$$

(4) The design stress-strain diagram given in **Clause J.5.3(4) of EN 1992-1-1:2023** is adopted.

1.5 Durability

(1) The durability of the strengthening system should be ensured, for the design life of the structure, considering the exposure classes in accordance with **PWDM**, with additional protective measures if necessary.
(2) The considerations for ensuring the durability of the strengthening system shall be in accordance with **Clause J.6(2) of EN 1992-1-1:2023**.
(3) The consideration of thermal effects shall be in accordance with **Clause J.6(3) of EN 1992-1-1:2023**.
(4) Available protective coatings in **Clause 4.1.3 of ACI PRC-440.2-23** may be used to protect the EBR from damaging environmental and mechanical effects.
(5) The inspection and maintenance of coatings shall be in accordance with **Clause 9.3.4 of ACI PRC-440.2-23**.
(6) The durability of resins used in FRP shall be in accordance with **Clause 9.3.4 of ACI PRC-440.2-23**. The use of epoxy as resins is recommended for marine structures.
(7) The crack injection may be applied according to **Clause 6.4.1.2 of ACI PRC-440.2-23**.

1.6 Structural analysis

Structural analysis shall be performed in accordance with **Clause J.7 of EN 1992-1-1: 2023**.

1.7 Ultimate Limit States (ULS)

1.7.1 Bending with or without axial force

1.7.1.1 Flexural strengthening with Externally Bonded Reinforcement (EBR)

(1) The basic assumptions for determining the ultimate moment resistance of members strengthened in flexure shall be in accordance with **Clause J.8.1.1(1) of EN 1992-1-1:2023**.
(2) The strain state of the existing reinforcement and concrete members shall be determined

using the elastic analysis based on a cracked section under the effects of actions during the installation of the FRP system. The strain of strengthened members shall be determined according to **Clause J.8.1.1(2) of EN 1992-1-1:2023**.

(3) The concrete strength range for which this document is applicable is consistent with **Clause J.8.1.1(3) of EN 1992-1-1:2023**.

(4) The plane section assumption and a linear elastic stress-strain relationship for FRP are adopted, in accordance with **Clause J.8.1.1(4) of EN 1992-1-1:2023**.

(5) Failure of FRP-strengthened beams/slabs without the steel tension bars reaching yielding is not allowed as such a failure is highly brittle.

(6) The following flexural failure modes following the yielding of the steel in tension should be investigated for determining the flexural strength of an FRP-strengthened section: (a) Rupture of EBR; (b) FRP debonding due to the anchorage failure of EBR strengthening system; (c) Concrete crushing.

1.7.1.2 Compressive strengthening with closed FRP wrapping

(1) The design provisions in this document are applicable to RC columns that have been strengthened with an FRP jacket, which is continuous along the column height within the region designated for strengthening, and the fibre direction should be in or close to the hoop direction (which is perpendicular to the longitudinal axis) of the strengthened column (**Figure 1**).

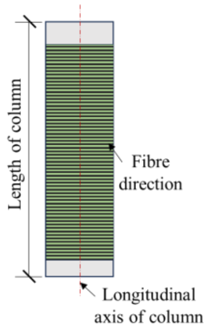
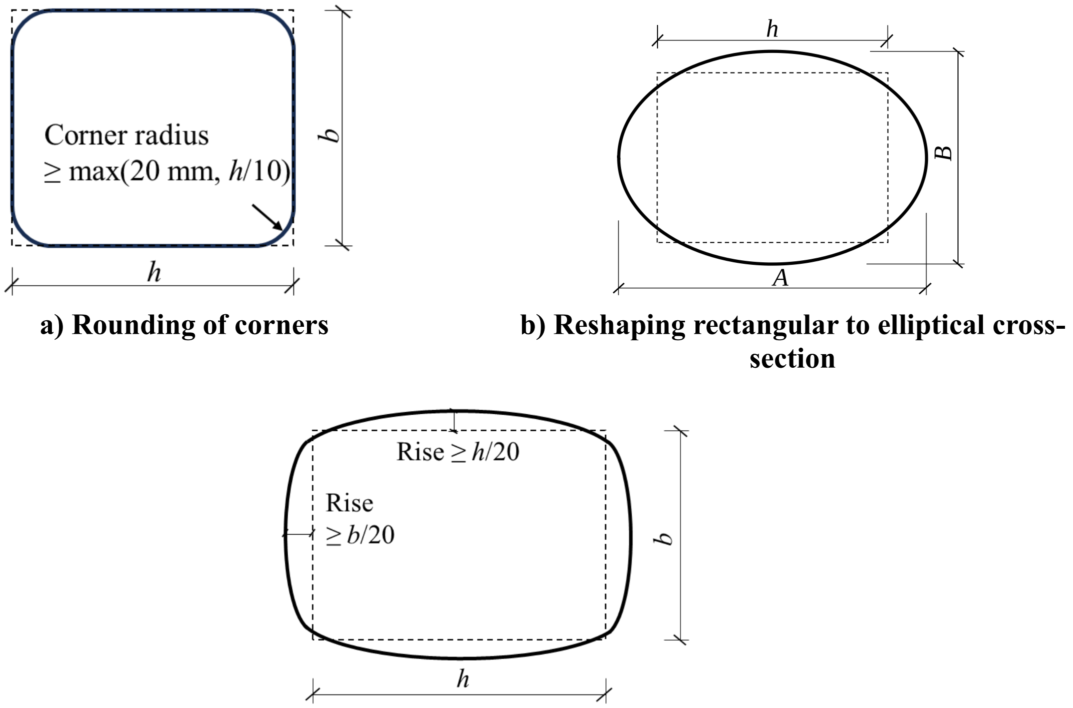


Figure 1 — Arrangement of closed FRP wrapping on columns

(2) When strengthening a rectangular column, its sharp corners shall be rounded in accordance with the requirements shown in **Figure 2a)**. If the height of a rectangular cross-section $h > 600$ mm, its section sides shall be reshaped to an elliptical cross-section [**Figure 2 b)**] or modified to a curvilinear shape (referred to as expanded rectangular section hereinafter) to enhance the effectiveness of FRP confinement [**Figure 2 c)**]. For FRP-strengthened columns with an expanded rectangular cross-section, the original width and height of the cross-section shall be utilised in design calculations.



a) Rounding of corners

b) Reshaping rectangular to elliptical cross-section

c) Modifying side shape to expanded rectangular section

Figure 2 — Strengthening of rectangular RC columns

(3) The level of concrete confinement provided by the FRP jacket is characterized by the confinement stiffness ratio:

$$\beta_{\mathrm{j}} = \frac{E_{\mathrm{f}} t_{\mathrm{f}}}{\eta_{\mathrm{cc}} f_{\mathrm{ck}} R} \tag{5}$$

where

E_{f}	is the modulus of elasticity of the FRP jacket in the hoop direction;
t_{f}	is the thickness of the FRP jacket;
R	is a geometric parameter which is specified in **Table 2 of this document**;
η_{cc}	is a factor determined according to **Formula (5.4) of EN 1992-1-1:2023**;
f_{ck}	is the characteristic concrete cylinder compressive strength.

(4) β_{jb} is the lower limit of β_{j} for sufficiently-confined concrete and is defined in **Table 2 of this document**. The concrete is sufficiently confined when $\beta_{\mathrm{j}} > \beta_{\mathrm{jb}}$; otherwise, the concrete is insufficiently confined.

Table 2 — Definition of *R* and β_{jb}

Parameter	Circular cross-section	Elliptical cross-section	Rectangular cross-section and expanded rectangular cross-section
R	r	$0.5A$	$0.5\sqrt{b^2 + h^2}$
β_{jb}	6.5	$6.5/k_{\mathrm{s}\sigma}$	$6.5/k_{\mathrm{s}\sigma}$
NOTE r is the radius of a circular cross-section, b and h are the width and the height of a rectangular cross-section (or an expanded rectangular cross-section) respectively, $k_{\mathrm{s}\sigma}$ is the shape factor for the compressive strength, which is defined in **Clause 1.7.1.2(8) of this document**.			

(5) The confining effect provided by EBR may be considered in the design of axially-loaded members under the following conditions, where:

— geometric parameter $R \geq 75$ mm;
— height-to-width ratio of rectangular sections is not greater than 1.5;
— corner radius for rectangular cross-sections is $r_c \geq \max(20 \text{ mm}, h/10)$;
— slenderness satisfies the condition $l_0/h \leq 12.5\text{-}375\varepsilon_{\mathrm{ruk}}$.

NOTE l_0 is the effective length of column determined according to **Clause 14.4.5.1 of EN 1992-1-1:2023**, h is the dimension of the cross-section in the plane of bending under consideration; $\varepsilon_{\mathrm{ruk}}$ is the characteristic ultimate strain of the FRP jacket in the hoop direction. $\varepsilon_{\mathrm{ruk}}$ may be taken as $0.5\varepsilon_{\mathrm{fuk}}$ for CFRP or $0.7\varepsilon_{\mathrm{fuk}}$ for GFRP, unless more accurate information is available.

(6) The degree of enhancement in the concrete strength due to FRP confinement needs to be limited as:

$$\frac{f_{\mathrm{cd,c}}}{f_{\mathrm{cd}}} \leq 1.75 \tag{6}$$

where $f_{\mathrm{cd,c}}$ is the design compressive strength of FRP-confined concrete, determined by **Formula (10) of this document**.

(7) For the design of FRP-confined concrete cross-sections, the following stress-strain relationship may be used:

$$\sigma_{\mathrm{c}} = \begin{cases} E_{\mathrm{c,eff}}\varepsilon_{\mathrm{c}} - \dfrac{\left(E_{\mathrm{c,eff}} - E_2\right)^2}{4f_{\mathrm{cd}}}{\varepsilon_{\mathrm{c}}}^2, & 0 \leq \varepsilon_{\mathrm{c}} \leq \varepsilon_{\mathrm{t}} \\ f_{\mathrm{cd}} + E_2\varepsilon_{\mathrm{c}} \quad , & \varepsilon_{\mathrm{t}} \leq \varepsilon_{\mathrm{c}} \leq \varepsilon_{\mathrm{cu,c}} \end{cases} \tag{7}$$

where

σ_{c} is the axial compressive stress;
ε_{c} is the axial compressive strain;
E_2 is the slope of the linear second portion, determined by:

$$E_2 = \frac{f_{\mathrm{cd,c}} - f_{\mathrm{cd}}}{\varepsilon_{\mathrm{cu,c}}} \tag{8}$$

ε_{t} is the strain at which the parabolic first portion meets the linear second portion;

$$\varepsilon_{\mathrm{t}} = \frac{2f_{\mathrm{cd}}}{E_1 - E_2} \tag{9}$$

$E_{\mathrm{c,eff}}$ is effective modulus of elasticity of concrete accounting for creep deformations, determined by **Formula (9.1) of EN 1992-1-1:2023**;
$\varepsilon_{\mathrm{cu,c}}$ is the design ultimate axial strain of FRP-confined concrete, determined by **Formula (16) of this document**.

The corresponding stress-strain diagram of FRP-confined concrete in compression is displayed in **Figure 3**.

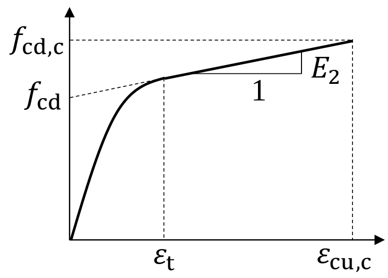


Figure 3 — Stress-strain diagram of FRP-confined concrete in compression

(8) The compressive strength of confined concrete in columns $f_{cd,c}$ may be calculated as follows:

$$f_{cd,c} = f_{cd} + 3.5\frac{E_f t_f}{R}\left(k_{s\sigma} - \frac{6.5}{\beta_j}\right)\varepsilon_{ru} \tag{10}$$

where

ε_{ru} is the design ultimate strain of the FRP jacket in the hoop direction, determined by:

$$\varepsilon_{ru} = \frac{\eta_f \cdot \varepsilon_{ruk}}{\gamma_f} \tag{11}$$

$k_{s\sigma}$ is the shape factor for the ultimate compressive strength,

For circular sections,

$$k_{s\sigma} = 1 \tag{12}$$

For rectangular sections,

$$k_{s\sigma} = \left(1.25\frac{r_c}{b} + \frac{r_c}{h} + 0.33\right)\left(\frac{b}{h}\right)^{0.4} \tag{13}$$

For elliptical cross-sections,

$$k_{s\sigma} = 1.6 - 0.6\left(\frac{A}{B}\right) \tag{14}$$

For expanded rectangular cross-sections,

$$k_{s\sigma} = \left(1.25\frac{r_c}{b} + \frac{r_c}{h} + 0.33\right)\left(\frac{b}{h}\right)^{0.4}\left(1 + 2.5r_{cs}^{0.5}\right) \tag{15}$$

where

r_c is the radius of the rounded corner;
r_{cs} is the ratio of rise to the side length for an expanded rectangular cross-section;
A is the length of major axis for an elliptical cross-section;
B is the length of minor axis for an elliptical cross-section.

(9) The ultimate strain of confined concrete in columns $\varepsilon_{cu,c}$ may be calculated as follows:

$$\varepsilon_{cu,c} = 0.0035 + 0.6k_{s\varepsilon}{\beta_j}^{0.8}{\varepsilon_{ru}}^{1.45} \tag{16}$$

where

$k_{s\varepsilon}$ is the shape factor for the ultimate axial strain,

For circular sections,

$$k_{s\varepsilon} = 1 \tag{17}$$

For rectangular sections,

$$k_{s\varepsilon} = \left(1.25\frac{r_c}{b} + \frac{r_c}{h} + 0.33\right)\left(\frac{h}{b}\right)^{0.5} \tag{18}$$

For elliptical cross-sections,

$$k_{s\varepsilon} = 1.3 - 0.3\left(\frac{A}{B}\right) \tag{19}$$

For expanded rectangular cross-sections

$$k_{\mathrm{s}\varepsilon} = \left(1.25\frac{r_{\mathrm{c}}}{b} + \frac{r_{\mathrm{c}}}{h} + 0.33\right)\left(\frac{h}{b}\right)^{0.5} \left(1 + 1.5r_{\mathrm{cs}}^{0.03}\right) \tag{20}$$

(10) The consideration of confining effect of FRP in creep calculations shall be in accordance with **Clause J.8.1.2(5) of EN 1992-1-1:2023**.

1.7.1.3 Simplified method for member strength evaluation

1.7.1.3.1 Simplification for stress distribution of concrete in the compression zone

(1) The assumption of equivalent rectangular stress block distribution in **Clause 8.1.2(2) of EN 1992-1-1:2023** may apply to the simplified method for strength evaluation. The stress value of the equivalent stress block is taken to be $\alpha_1 f_{\mathrm{cd}}$ (for unconfined concrete) or $\alpha_1 f_{\mathrm{cd,c}}$ (for FRP-confined concrete), while the depth of the equivalent stress block x_{sb} is taken to be $\beta_1 x_{\mathrm{u}}$, where x_{u} is the depth of the neutral axis at the ultimate limit state.

(2) The stress factor α_1 and the depth factor β_1 of the equivalent stress block may be determined as follows:

— for RC members strengthened in flexure with EBR,

$$\begin{cases} \alpha_1 = 0.5 + 0.5\sigma_{\mathrm{f,md}}/\sigma_{\mathrm{fe,m1}} \\ \beta_1 = 0.8 \end{cases} \tag{21}$$

where

$\sigma_{\mathrm{f,md}}$ is the design maximum tensile stress of FRP for the ultimate limit state of flexural failure of the member, determined by **Formula (28) of this document**;

$\sigma_{\mathrm{fe,m1}}$ is the assumed tensile stress of FRP when the concrete at the extreme compression face reaches ultimate strain, determined by **Clause 1.7.1.3.2(3) of this document**.

NOTE the values of coefficients calculated by **Formula (21) of this document** apply when the strength class for concrete is no higher than C40/50, as shown in **Table 5.1 of EN 1992-1-1:2023**.

— For RC columns strengthened with closed FRP wrapping subjected to eccentrically applied load,

$$\begin{cases} \alpha_1 = 1.17 - 0.2f_{\mathrm{cd,c}}/f_{\mathrm{cd}} \\ \beta_1 = 0.9 \end{cases} \tag{22}$$

1.7.1.3.2 Strength of RC members strengthened in flexure with EBR

(1) The flexural strength of rectangular or T-shaped RC members strengthened in flexure with EBR may be determined by:

— when the depth of equivalent stress block x_{sb} is larger than the thickness of flange h_{f}' (Figure 4),

$$\alpha_1 f_{\mathrm{cd}} b_{\mathrm{w}} x_{\mathrm{sb}} + \alpha_1 f_{\mathrm{cd}}(b_{\mathrm{f}}' - b_{\mathrm{w}})h_{\mathrm{f}}' = f_{\mathrm{yd}} A_{\mathrm{st}} - f_{\mathrm{yd}}' A_{\mathrm{sc}} + \sigma_{\mathrm{f,md}} A_{\mathrm{f}} \tag{23}$$

$$M_{\mathrm{Ed}} \leq \alpha_1 f_{\mathrm{cd}} b_{\mathrm{w}} x_{\mathrm{sb}} \left(d - \frac{x_{\mathrm{sb}}}{2}\right) + \alpha_1 f_{\mathrm{cd}} (b'_{\mathrm{f}} - b_{\mathrm{w}}) h'_{\mathrm{f}} \left(h_0 - \frac{h'_{\mathrm{f}}}{2}\right) + f'_{\mathrm{yd}} A_{\mathrm{sc}} (d - a') + \sigma_{\mathrm{f,md}} A_{\mathrm{f}} (d_{\mathrm{f}} - d) \tag{24}$$

where

M_{Ed}	is the design value of the applied internal bending moment;
b_{w}	is the width of rectangular section or the width of the web of T-shaped section;
d	is the effective depth of steel tension reinforcement (i.e., distance from the compression face to the centroid of steel tension reinforcement);
d_{f}	is the effective depth of FRP tension reinforcement (i.e., distance from the compression face to the centroid of FRP tension reinforcement);
b'_{f}	is the flange width of T-section;
h'_{f}	is the flange thickness of T-section;
x_{sb}	is the height of the equivalent rectangular stress diagram of the concrete compression zone;
a'	is the distance from the compression face to the centroid of steel compression reinforcement;
A_{st}, A_{sc}	are the cross-sectional areas of steel tension and compression reinforcement, respectively;
A_{f}	is the cross-sectional area of EBR on the tension face of strengthened RC member;
f_{cd}	is the design value of concrete compressive strength;
f_{yd}	is the design yield strength of tension reinforcement (positive value);
f'_{yd}	is the design yield strength of compression reinforcement(positive value);
α_1	is determined by **Formula (21) of this document**;
$\sigma_{\mathrm{f,md}}$	is determined by **Formula (28) of this document**.

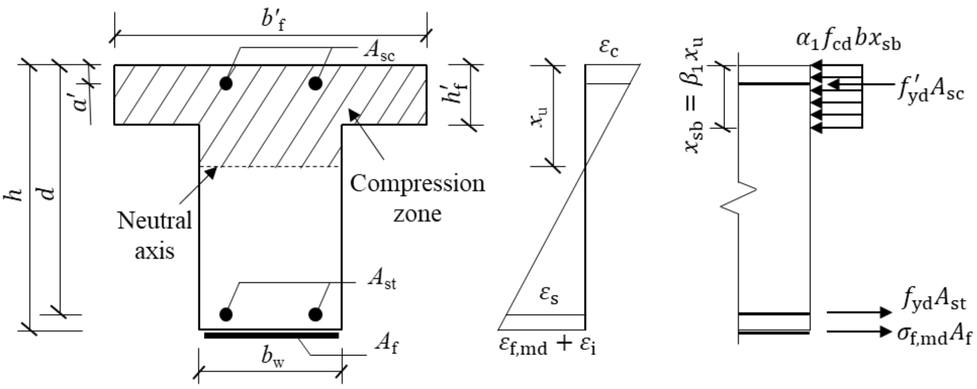


Figure 4 — Stress and strain distributions when $x_{\mathrm{sb}} > h'_{\mathrm{f}}$

— when the depth of equivalent stress block x_{sb} is smaller than the thickness of flange h'_{f} and larger than $2a'$ (Figure 5),

$$\alpha_1 f_{\mathrm{cd}} b'_{\mathrm{f}} x_{\mathrm{sb}} = f_{\mathrm{yd}} A_{\mathrm{st}} - f'_{\mathrm{yd}} A_{\mathrm{sc}} + \sigma_{\mathrm{f,md}} A_{\mathrm{f}} \tag{25}$$

$$M_{\mathrm{Ed}} \leq \alpha_1 f_{\mathrm{cd}} b'_{\mathrm{f}} x_{\mathrm{sb}} \left(d - \frac{x_{\mathrm{sb}}}{2}\right) + f'_{\mathrm{yd}} A_{\mathrm{sc}} (d - a') + \sigma_{\mathrm{f,md}} A_{\mathrm{f}} (d_{\mathrm{f}} - d) \tag{26}$$

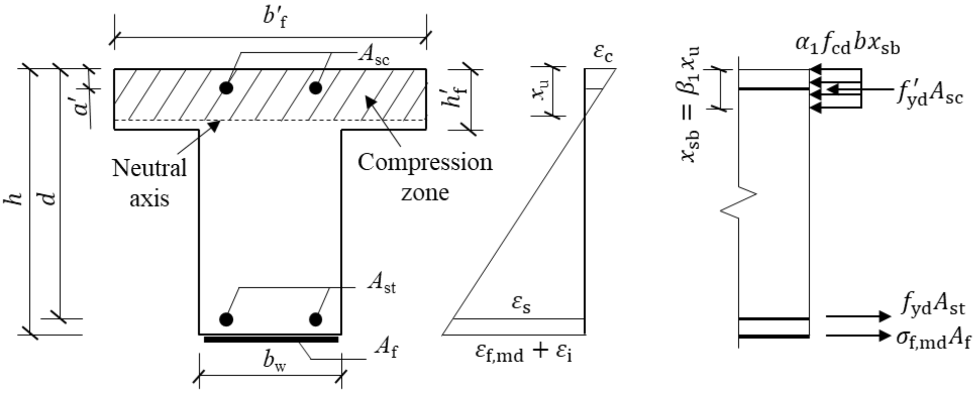


Figure 5 — Stress and strain distributions when $2a' < x_{sb} \leq h'_f$

— when the depth of equivalent stress block x_{sb} is smaller than $2a'$,

$$M_{Ed} \leq f_{yd} A_{st}(d - a') + \sigma_{f,md} A_f (d_f - a') \quad (27)$$

(2) Consistent with the flexural failure modes to be investigated mentioned in Clause 8.1.1(6), the design maximum tensile stress $\sigma_{f,md}$ in the FRP reinforcement for the ultimate limit state of flexural failure of member is determined by:

$$\sigma_{f,md} = \min\{f_{fud}, f_{bfRd}, \sigma_{fe,m1}\} \quad (28)$$

where

f_{fud}	is the design long-term tensile strength of EBR;
f_{bfRd}	is the design bond strength of the anchorage, determined according to **Section 1.10.1.1 of this document**;
$\sigma_{fe,m1}$	is determined by **Clause 1.7.1.3.2(3) of this document**.

(3) The assumed tensile stress of FRP when the concrete at the extreme compression face reaches ultimate strain $\sigma_{fe,m1}$ may be determined as follows:

— when the depth of equivalent stress block x_{sb} is larger than the thickness of flange h'_f,

$$f_{cd}[b_w x_{sb} + (b'_f - b_w)h'_f] = f_{yd} A_{st} - f'_{yd} A_{sc} + \sigma_{fe,m1} A_f \quad (29)$$

$$x_{sb} = \frac{\beta_1 \varepsilon_{cu}}{\varepsilon_{cu} + \frac{\sigma_{fe,m1}}{E_f} + \varepsilon_i} h \quad (30)$$

where

ε_{cu}	is the ultimate compressive strain of concrete;
ε_i	is the initial strain at the tension face of the concrete due to pre-existing loading, calculated on the basis of a cracked section.

— when the depth of equivalent stress block x_{sb} is smaller than the thickness of flange h'_f,

$$f_{cd} b'_f x_{sb} = f_{yd} A_{st} - f'_{yd} A_{sc} + \sigma_{fe,m1} A_f \quad (31)$$

$$x_{\mathrm{sb}} = \frac{\beta_1 \varepsilon_{\mathrm{cu}}}{\varepsilon_{\mathrm{cu}} + \frac{\sigma_{\mathrm{fe,m1}}}{E_{\mathrm{f}}} + \varepsilon_{\mathrm{i}}} h \tag{32}$$

(4) For rectangular section, $b_{\mathrm{f}}' = b_{\mathrm{w}}$ and $h_{\mathrm{f}}' = 0$. For one-way slabs, b_{w} and h may respectively be taken as 1,000 mm and the slab thickness for strength evaluation, while the EBR shall be evenly distributed.

(5) The depth of the equivalent stress block for un-strengthened and strengthened members should satisfy the following requirement to ensure the yielding of steel tension bars at the ultimate limit states,

$$x_{\mathrm{sb}} \leq \xi_{\mathrm{b}} d = \frac{\beta_1}{1 + f_{\mathrm{yd}}/(E_{\mathrm{s}} \varepsilon_{\mathrm{cu}})} d \tag{33}$$

1.7.1.3.3 Strength of RC columns strengthened with closed FRP wrapping

(1) The additional load eccentricity e_{a} of RC columns strengthened with closed FRP wrapping shall be defined as:

$$e_{\mathrm{a}} = \max(20\mathrm{mm}, h/30) \tag{34}$$

where

h is the dimension of the cross-section in the plane of bending under consideration.

(2) The slenderness effect of FRP-strengthened columns may be omitted, when the slenderness ratio satisfies the following equation:

$$\frac{l_0}{h} \leq \frac{15 \frac{e_2 - e_1}{h} + 5}{\frac{f_{\mathrm{cd,c}}}{f_{\mathrm{cd}}} (1 + 30 \varepsilon_{\mathrm{ruk}})} \tag{35}$$

where

l_0 is the effective length of column;

e_1, e_2 are the load eccentricities at the two ends of a column respectively, where e_2 always has the larger absolute value of the two and is always taken to be positive.

In this case, the amplification factor to account for the slenderness effect $\eta = 1$.

(3) When considering the slenderness effect, the amplification factor η shall be calculated as:

$$\eta = 1 + \frac{1.25 \varepsilon_{\mathrm{cu,c}} + 0.0017}{\frac{8.3 e_{\mathrm{i}}}{d}} \left(\frac{l_0}{h}\right)^2 \zeta_1 \zeta_2 \tag{36}$$

where

d is the effective depth of the cross-section, taken as $r + r_s$ for circular sections, where

r is the radius of the cross-section;

r_s is the radius of the circle formed by the geometric centres of the longitudinal steel bars.

e_i is the initial load eccentricity, determined by

$$e_i = e_0 + e_a \tag{37}$$

where

e_0 is the equivalent load eccentricity, calculated as

$$e_0 = 0.6e_2 + 0.4e_1 \geq 0.4e_2 \tag{38}$$

e_a is the additional load eccentricity, defined as **Formula (34) of this document**.

$$\zeta_1 = \frac{0.8f_{cd,c}A_c}{N_{Ed}};\ \text{if}\ \zeta_1 > 1,\ \text{take}\ \zeta_1 = 1 \tag{39}$$

where

A_c is the cross-sectional area of concrete;

N_{Ed} is the design value of the applied axial force.

$$\zeta_2 = (1.15 + 30\varepsilon_{ru}) - (0.01 + 6\varepsilon_{ru})\frac{l_0}{h};\ \text{if}\ \zeta_2 > 1,\ \text{take}\ \zeta_2 = 1 \tag{40}$$

(4) For FRP-strengthened circular RC columns with six or more uniformly-distributed longitudinal steel bars, the strength of the column under eccentric compression may be calculated by:

$$\begin{cases} N_{Ed} \leq \alpha\alpha_1 f_{cd,c}A_c\left(1 - \dfrac{\sin 2\pi\alpha}{2\pi\alpha}\right) + (\alpha_c - \alpha_t)f_{yd}A_s \\ N_{Ed}e_{max} \leq \dfrac{2}{3}\alpha_1 f_{cd,c}A_c r\dfrac{\sin^3\pi\alpha}{\pi} + f_{yd}A_s r_s\dfrac{\sin\pi\alpha_c + \sin\pi\alpha_t}{\pi} \end{cases} \tag{41}$$

where

A_s is the total cross-sectional area of the longitudinal steel reinforcement;

$2\pi\alpha$ is the ratio of the central angle corresponding to the compression zone of the concrete [**Figure 6 a)**];

$$e_{max} = \max(\eta e_i, e_2 + e_a) \tag{42}$$

$$\alpha_c = 1.25\alpha - 0.125;\ 0 \leq \alpha_c \leq 1 \tag{43}$$

$$\alpha_t = 1.125 - 1.5\alpha;\ 0 \leq \alpha_t \leq 1 \tag{44}$$

(5) For FRP-strengthened rectangular and expanded rectangular columns, the strength of the column under eccentric compression may be calculated from:

$$\begin{cases} N_{Ed} \leq \alpha_1 f_{cd,c}bx_{sb} + \displaystyle\sum_{i=1}^{m}\sigma_{si}A_{si} \\ N_{Ed}e_{max} \leq \alpha_1 f_{cd,c}bx_{sb}\left(\dfrac{h}{2} - \dfrac{x_{sb}}{2}\right) + \displaystyle\sum_{i=1}^{m}\sigma_{si}A_{si}\left(\dfrac{h}{2} - a_{si}\right) \end{cases} \tag{45}$$

where

b is the dimension of the cross-section in the direction perpendicular to h;

x_{sb} is the depth of the equivalent stress block for concrete in compression;

m is the number of layers of longitudinal steel bars arranged along the direction of h;

A_{si} is the cross-sectional area of longitudinal reinforcement located at the i^{th} layer [**Figure 6 b)**];

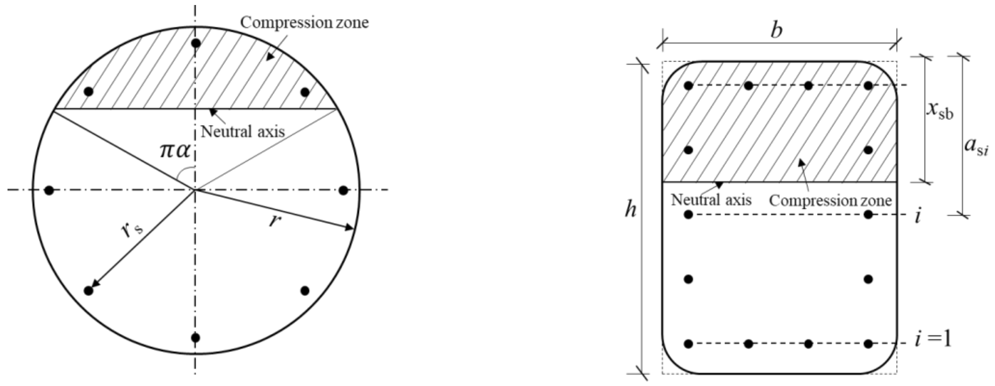
a_{si} is the distance from the point of the resultant force of the longitudinal steel reinforcement located at the i^{th} layer to the extreme compression face;

σ_{si} is the stress of longitudinal reinforcement located at the i^{th} layer, determined by:

$$-f_{\mathrm{yd}} \le \sigma_{si} = \varepsilon_{\mathrm{cu,c}}\left(1 - \frac{\beta_1 a_{si}}{x_{\mathrm{sb}}}\right) E_{\mathrm{s}} \le f_{\mathrm{yd}} \tag{46}$$

where the value of σ_{si} is positive for the longitudinal bar in compression, while negative in tension;

α_1 is determined by **Formula (22) of this document**.



a) Circular cross-section with uniform steel reinforcement around the perimeter

b) Rectangular section with steel reinforcement along section height

Figure 6 — Cross-sections of FRP-strengthened columns

(6) The strength of un-strengthened circular RC columns may also be calculated using **Formula (41) of this document** by setting $f_{\mathrm{cd,c}} = f_{\mathrm{cd}}$ and $\alpha_{\mathrm{c}} = \alpha$, $\alpha_{\mathrm{t}} = 1.25 - 2\alpha$ instead of **Formulae (43),(44) of this document**. The strength of un-strengthened rectangular and expanded rectangular RC columns may also be calculated using **Formula (45) of this document** by setting $f_{\mathrm{cd,c}} = f_{\mathrm{cd}}$. The calculation of $e_{\max}$ for FRP-strengthened columns can still be used to conservatively account for the slenderness effect of un-strengthened columns.

(7) The strength of the column in the direction of the load eccentricity and that of the column in the direction perpendicular to the load eccentricity should both be calculated, and the smaller value of the two should be taken as the design strength of the column.

1.7.2 Shear

1.7.2.1 General verification procedure

Shear resistance of un-strengthened RC members shall be determined in accordance with **Clause J.8.2.1 of EN 1992-1-1:2023**.

1.7.2.2 Detailed verification for strengthened members not requiring design shear reinforcement

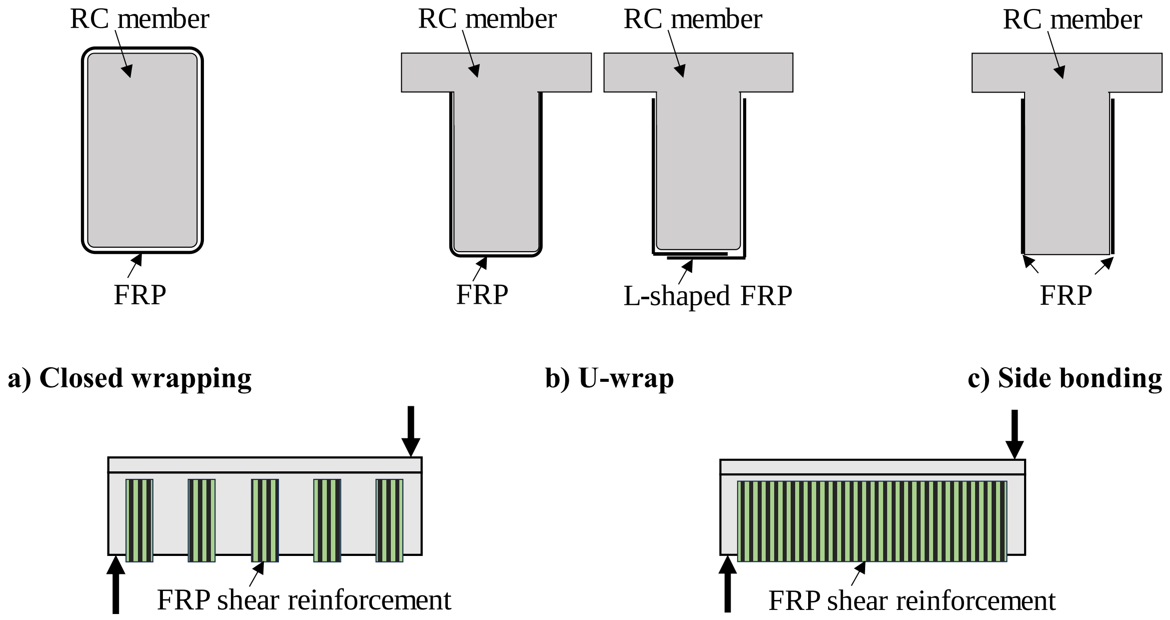
Clause J.8.2.2 of EN 1992-1-1:2023 is followed for determining the value of A_{sl} used in **Formula (8.27) or (8.33) of EN 1992-1-1:2023**.

1.7.2.3 Members requiring design shear reinforcement

(1) Shear resistance of internal reinforcement shall be calculated in accordance with **Clause J.8.2.3(1) of EN 1992-1-1:2023**.

(2) The shear strengthening types for which this document is applicable are consistent with **Clause J.8.2.3(2) of EN 1992-1-1:2023**.

(3) The provisions apply to shear strengthening systems displayed in **Figure 7**.



a) Closed wrapping **b) U-wrap** **c) Side bonding**

d) Discrete application of strengthening **e) Continuous application of strengthening**

Figure 7 — Types of shear strengthening schemes

(4) The shear resistance of a section strengthened with EBR may be taken as:

$$\tau_{\mathrm{Rd,FRP}} = \tau_{\mathrm{Rd}} + \frac{V_{\mathrm{Rd,f}}}{(b_{\mathrm{w}} \cdot z)} \leq 0.5 \cdot \nu \cdot f_{\mathrm{cd}} \tag{47}$$

where

$V_{\mathrm{Rd,f}}$	is the contribution of the FRP shear reinforcement to the shear strength, determined by **Formula (48) of this document** for RC beam or **Formula (51) of this document** for RC column;
b_{w}	is the width of the cross-section of beams;
z	is the lever arm for the shear stress calculation defined as $z = 0.9d$; where d refers to the effective depth of the member (i.e. distance from the compression face to the centroid of steel tension reinforcement);
τ_{Rd}	is shear resistance of the un-strengthened member , determined according to **Clause 8.2 of EN 1992-1-1:2023**;
ν	is concrete strength reduction factor due to shear cracking, and $\nu = 0.5$ may be adopted when using the angles of the compression field given in **Clause 8.2.3(4) of EN 1992-1-1:2023**.

NOTE For simplified calculation of **Formula (47) of this document**, the width of the cross-section b_{w} and the effective depth of the flexural steel tension reinforcement d for circular sections may be taken as 1.76*r* and 1.6*r*, respectively, where *r* is the radius of the cross-section.

(5) The contribution of the FRP shear reinforcement to the shear strength of the RC beam is determined by:

$$V_{\mathrm{Rd,f}} = 2\psi_{\mathrm{f}} f_{\mathrm{fwd}} t_{\mathrm{f}} b_{\mathrm{f}} \frac{h_{\mathrm{fe}}}{s_{\mathrm{f}}} (\cot\theta + \cot\alpha_{\mathrm{f}}) \cdot \sin\alpha_{\mathrm{f}} \tag{48}$$

where

f_{fwd} is the average stress (also referred to as the effective stress) of EBR intersected by the shear crack at the ultimate limit state, which is determined according to **Section 1.7.2.4 of this document**;

b_{f} is the width of the EBR in the direction perpendicular to the fibre orientation;

t_{f} is the thickness of the FRP strips;

s_{f} is the centre-to-centre spacing between FRP strips along the longitudinal axis of the member;

α_{f} is the angle of inclination of fibres in the FRP reinforcement to the longitudinal axis of the member;

h_{fe} is the effective height of the FRP, determined by

$$h_{\mathrm{fe}} = h_{\mathrm{f}} - (h - 0.9d) \tag{49}$$

where

h_{f} is the height of the FRP reinforcement bonded on the web, determined by:

$$h_{\mathrm{f}} = d_{\mathrm{fl}} - d_{\mathrm{ft}} \tag{50}$$

where

d_{fl} is the distance from the compression face to the lower end of the FRP;

d_{ft} is the distance from the compression face to the top end of the FRP and = 0 for complete wrapping.

h is the total height of member.

ψ_{f} is the reduction factor accounting for the unfavourable effect of shear interaction between internal steel shear reinforcement and external FRP shear reinforcement, determined according to **Section 1.7.2.5 of this document**;

θ is the angle between the compression field and the member axis. Unless a more rigorous analysis is undertaken, θ should be taken as 45 degrees for the calculation of τ_{Rd} and $V_{\mathrm{Rd,f}}$.

The relevant dimensional parameters are displayed in **Figure 8** as an illustration.

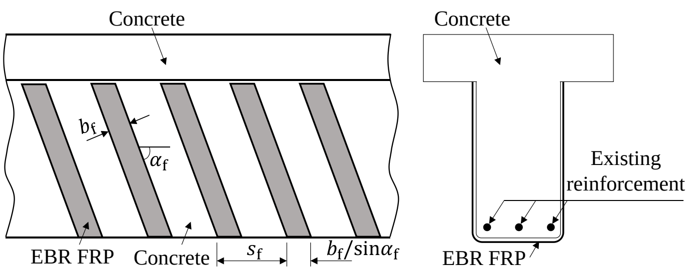


Figure 8 — Definition of dimensional parameters for shear strengthening

(6) For RC columns strengthened with closed FRP wrapping, the contribution of EBR to the shear strength may be determined by:

$$V_{\mathrm{Rd,f}} = \frac{K_{\mathrm{s}}(1-n)b_{\mathrm{f}}t_{\mathrm{f}}}{s_{\mathrm{f}}} f_{\mathrm{fwd}} h \tag{51}$$

where

f_{fwd} is determined by **Formulae (52)-(54) of this document**;
h is the diameter of cross-section (circular section), or the sectional dimension in the direction of shear force (rectangular section);
n is the axial compression ratio of column, taken as $N_{\mathrm{Ed}}/f_{\mathrm{cd}}A_{\mathrm{c}}$.
K_{s} is the shape coefficient, taken as 1.57 for circular section, or 2 for rectangular section

For continuous FRP jackets along the height of the column, $s_{\mathrm{f}} = b_{\mathrm{f}}$.

1.7.2.4 Determination of effective stress of EBR

(1) The average stress in FRP strips at the ultimate limit state shall be evaluated by:

$$f_{\mathrm{fwd}} = D_{\mathrm{f}}\sigma_{\mathrm{f,max}} \tag{52}$$

where

D_{f} is the FRP stress distribution factor;
$\sigma_{\mathrm{f,max}}$ is the maximum stress reached in the FRP intersected by the critical shear crack.

(2) When the shear failure is controlled by the tensile rupture of FRP, the values of D_{f} and $\sigma_{\mathrm{f,max}}$ may be determined by:

$$D_{\mathrm{f}} = 0.5\left(1 + \frac{d_{\mathrm{ft}}}{h_{\mathrm{fe}} + d_{\mathrm{ft}}}\right) \tag{53}$$

$$\sigma_{\mathrm{f,max}} = \min\left(0.8 \times f_{\mathrm{fud}}, 0.8 \times \frac{0.015 \cdot \eta_{\mathrm{f}} \cdot E_{\mathrm{f}}}{\gamma_{\mathrm{f}}}\right) \tag{54}$$

NOTE The reduction factor of 0.8 accounts for the detrimental corner effect on the tensile strength of the FRP.

(3) When the shear failure is controlled by FRP debonding, the values of D_{f} and $\sigma_{\mathrm{f,max}}$ may be determined by:

$$D_{\mathrm{f}} = \begin{cases} \dfrac{2\left(1-\cos\dfrac{\pi\lambda}{2}\right)}{\pi\lambda\sin\dfrac{\pi\lambda}{2}}, & \lambda \le 1 \\ 1 - \dfrac{\pi - 2}{\pi\lambda}, & \lambda > 1 \end{cases} \tag{55}$$

$$\sigma_{\mathrm{f,max}} = \min(0.8 \times f_{\mathrm{fud}}, f_{\mathrm{bfRd}}) \tag{56}$$

where

$$\lambda = L/L_{\mathrm{e}} \tag{57}$$

where

L is the bond length of FRP shear reinforcement, determined by:

$$L = \begin{cases} \dfrac{h_{\mathrm{fe}}}{\sin\alpha_{\mathrm{f}}}, & \mathrm{U-wraps} \\ \dfrac{h_{\mathrm{fe}}}{2\sin\alpha_{\mathrm{f}}}, & \text{side bonding} \end{cases} \tag{58}$$

L_e is the effective bond length of EBR, determined by

$$L_{\mathrm{e}} = \sqrt{\frac{E_{\mathrm{f}} t_{\mathrm{f}}}{\sqrt{f_{\mathrm{ck}}}}} \tag{59}$$

f_{bfRd} is the design bond strength of the anchorage, determined according to **Section 1.10.1.2 of this document**.

(4) For the closed wrapping EBR shear strengthening schemes, f_{fwd} shall be evaluated for the FRP rupture failure mode. For U-wrap and side bonding schemes, both of FRP rupture and FRP debonding failure modes shall be considered and the lower prediction of f_{fwd} shall be used.

1.7.2.5 Effect of interaction between steel and FRP shear reinforcement

(1) For the U-wrap and the closed wrapping EBR shear strengthening schemes, $\psi_{\mathrm{f}} = 1.0$.

(2) For the side bonding EBR shear strengthening scheme, the value of ψ_{f} shall be calculated by:

$$\psi_{\mathrm{f}} = \frac{B}{(B+\mu)} \tag{60}$$

where

μ is the ratio between the shear contribution of steel shear reinforcement and the shear contribution of FRP strips when they are both fully mobilised, determined by:

$$\mu = \frac{\tau_{\mathrm{Rd}} \cdot b_{\mathrm{w}} \cdot z}{V_{\mathrm{Rd,f0}}} \tag{61}$$

where

$V_{\mathrm{Rd,f0}}$ is the contribution of the FRP shear reinforcement to the shear strength of the RC member, excluding the effects of shear interaction (evaluated by **Formula (48) of this document** with $\psi_f = 1.0$).

$$B = \begin{cases} \dfrac{1.01\times10^5}{\phi_{\mathrm{trans}}^{0.834} f_{\mathrm{yk}}^{1.88}}(\lambda + 2.11), & \text{for plain transverse bars} \\ \dfrac{2.05\times10^5}{\phi_{\mathrm{trans}}^{1.13} f_{\mathrm{yk}}^{1.71}}(\lambda + 1.58), & \text{for deformed transverse bars} \end{cases} \tag{62}$$

where

ϕ_{trans} is the diameter of transverse bars;

f_{yk} is the characteristic value of yield strength of steel transverse bars.

1.7.3 Torsion and combined actions

The contribution of FRP strengthening shall be assessed in accordance with **Clause J.8.3 of EN 1992-1-1:2023**.

1.7.4 Punching

This document does not apply to strengthening for punching shear.

1.7.5 Design with strut-and-tie models and stress fields

The contribution of FRP strengthening shall be assessed in accordance with **Clause J.8.5 of EN 1992-1-1:2023**.

1.8 Serviceability Limit States (SLS)

1.8.1 General

(1) The serviceability limit states considered for concrete members strengthened in flexure with EBR include stress limitations and deflection control.
(2) When FRP strengthening is applied to increase the shear strength of concrete members, no serviceability limit state needs to be checked.
(3) For FRP-strengthened concrete columns, the serviceability limit states to be checked include the stress limitations under service loads.
(4) The stresses and strains of the FRP-strengthened reinforced concrete section under service loads may be calculated using elastic analysis based on an uncracked section or a cracked section.

1.8.2 Stress limitations and crack control

(1) For concrete and steel reinforcement, the stress limitations under service loads follow the limits given in **Table 9.1 and Table 9.2 of EN 1992-1-1:2023**. In addition, for compressive strengthening, the service stress in longitudinal steel compression reinforcement should remain below $0.6f_{\mathrm{yk}}$.
(2) In cases of flexural strengthening with the requirement of fatigue verification according to **Clause 10 of EN 1992-1-1:2023**, the creep rupture and fatigue stress for FRP needs to be limited according to **Clause 10.2.9 of ACI PRC-440.2-23**.

1.8.3 Deflection control

(1) Deflections of beams or slabs strengthened with EBR may be estimated by ignoring the slip between the FRP and concrete and transforming the area of FRP to steel by taking account of the modulus ratio:

$$A_{\mathrm{se}} = A_{\mathrm{s}} + \frac{E_{\mathrm{f}}}{E_{\mathrm{s}}} A_{\mathrm{f}} \tag{63}$$

where

A_{se} is the converted area of tensile reinforcement considering the contribution of

FRP;

A_s is the area of tensile steel reinforcement;

A_f is the area of tensile FRP reinforcement.

(2) The total deflection of the flexural member after strengthening should include the pre-existing deflection and the deflection caused by increased loads after strengthening, determined by **Clause 9.3 of EN 1992-1-1:2023**.

1.9 Fatigue

The FRP-strengthened flexural members that need fatigue verification as required by **Clause 10.1(1) of EN 1992-1-1:2023** shall pass the fatigue checks in **Clause 1.8.2(2) of this document** and **Clause J.10 of EN 1992-1-1:2023**.

1.10 Bond and anchorage of EBR strengthening systems

1.10.1 Debonding failure and bond strength — flexure

1.10.1.1 General

(1) Anchorage of the strengthening system to the concrete surface of a member in flexure shall be provided to avoid the following failure mechanisms:

- intermediate crack debonding;
- end debonding;
- shear induced separation.

(2) The failure modes of end debonding and shear induced separation should be prevented from becoming the critical failure mode.

1.10.1.2 Intermediate crack (IC) debonding

(1) The characteristic IC debonding stress may be determined using **Formula (64)** by taking account of the FRP-to-concrete bond strength and additional anchorage using fibre anchors or mechanical fasteners.

$$f_{\mathrm{dbic}} = 0.114(4.41 - \alpha)\tau_{\max}\sqrt{\frac{E_{\mathrm{f}}}{t_{\mathrm{f}}}} + f_{\mathrm{anch}} \tag{64}$$

where

$\tau_{\max}$ is the maximum shear bond strength, determined by

$$\tau_{\max} = 1.5\beta_{\mathrm{w}} f_{\mathrm{ctm}} \tag{65}$$

where

β_{w} is the FRP-to-concrete width ratio factor, determined by:

$$\beta_{\mathrm{w}} = \sqrt{\frac{2 - b_{\mathrm{f}}/b_{\mathrm{t}}}{1 + b_{\mathrm{f}}/b_{\mathrm{t}}}} \tag{66}$$

b_t is the breadth of the section at the tension face.

$$\alpha = 3.41 L_{\mathrm{ee}}/L_{\mathrm{d}} \tag{67}$$

where

L_{d} is the distance from the maximum moment section or from the edge of the pure bending zone of the beam/slab to the end of EBR (mm).

$$L_{\mathrm{ee}} = 0.228\sqrt{E_{\mathrm{f}} t_{\mathrm{f}}} \quad \text{(mm)} \tag{68}$$

f_{anch} is the contribution by additional anchorage of mechanical fasteners determined by:

$$f_{\mathrm{anch}} = \mu_{\mathrm{a}} T_{\mathrm{a}} n_{\mathrm{a}} / A_{\mathrm{f}} \tag{69}$$

where

μ_{a} is the frictional coefficient between two rough concrete surfaces. A conservative value of μ_{a} = 0.7 may be adopted when a more accurate value is not available;

T_{a} is the characteristic value of the tensile pullout strength of an individual nail or bolt anchored in concrete, measured in the direction perpendicular to the tension face of the member;

n_{a} is the total quantity of nails or bolts installed within the shear span;

A_{f} is the cross-sectional area of EBR;

f_{anch}=0 if no additional anchorage is used. ($f_{\mathrm{anch}} A_{\mathrm{f}}$) is limited to 200 kN for a single FRP strip due to the limitation of available test data.

(2) The design bond strength of the anchorage is determined by:

$$f_{\mathrm{bfRd}} = \frac{\eta_{\mathrm{BA}} f_{\mathrm{dbic}}}{\gamma_{\mathrm{BA}}} \tag{70}$$

1.10.1.3 Anchorage measures for end debonding prevention

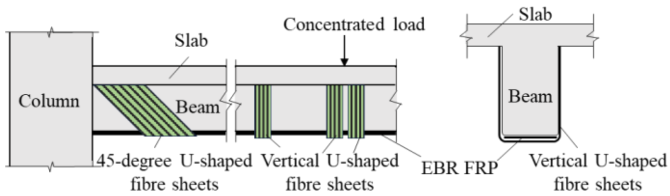
(1) EBR used for flexural strengthening of a beam may extend to the edge of the support for the beam. 45-degree diagonal or vertical U-shaped fibre sheets shall be applied at the end of EBR (**Figure 9**).

(2) For FRP-strengthened beam, when 45-degree diagonal U-shaped fibre sheets are used for end anchorage, their width should not be less than 0.8 times the beam height and 0.5 times the width of EBR, and their thickness should not be less than 0.5 times the thickness of EBR [**Figure 9(a)**].

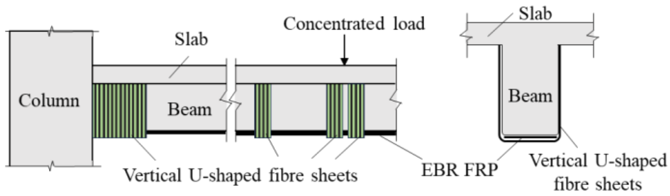
(3) For FRP-strengthened beam, when vertical U-shaped fibre sheets are used for end anchorage, their width should not be less than 1.2 times the beam height and 0.5 times the width of EBR, and their thickness should not be less than 0.5 times the thickness of EBR [**Figure 9(b)**].

(4) For FRP-strengthened beam, vertical U-shaped fibre sheets with the width of no less than 100 mm should be applied on both sides of a concentrated load or a secondary beam. The U-shaped fibre sheets should extend to the top of the beam or the bottom surface of the slab on the beam, with a thickness not less than 0.33 mm. Vertical U-shaped fibre sheets of the above size may also be placed in other locations along the beam, with the clear spacing of no larger than 3

times the beam height [**Figure 9**].



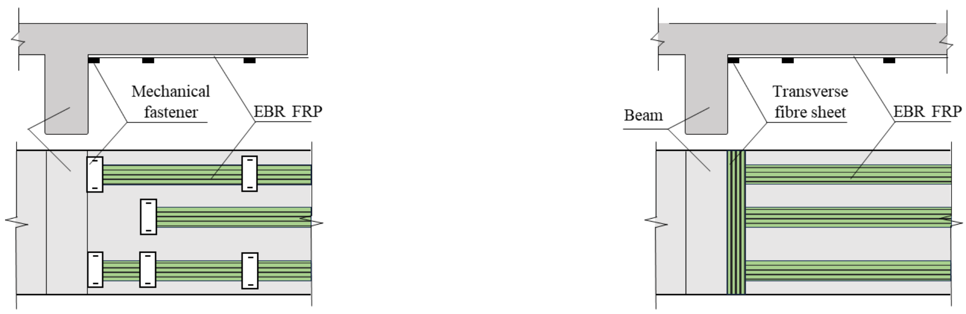
a) 45-degree U-shaped fibre sheets for end anchorage of EBR



b) Vertical U-shaped fibre sheets for end anchorage of EBR

Figure 9 — Anchorage measures for flexural strengthening of beams with EBR

(5) When the width of member surface strengthened in flexure with EBR is larger than 500 mm, the EBR may extend to the edge of support for the member. Anchorage measures shall be applied to the ends of EBR; mechanical fasteners or transverse fibre sheets with the width of no less than 200 mm may be used (**Figure 10**). When fibre sheets are used for flexural strengthening, the width and thickness of transverse fibre strips for anchorage shall respectively be no less than 0.5 times the width and 0.5 times the thickness of the fibre sheets for strengthening. When FRP strips are used for flexural strengthening, the cross-sectional area of transverse fibre sheets for anchorage shall be no less than 0.25 times the cross-sectional area of the FRP strips for strengthening.



a) Mechanical fasteners for anchorage **b) Transverse fibre sheets for anchorage**

Figure 10 — Anchorage measures for flexural strengthening of slabs with EBR

(6) Reliable anchorage measures should be used to connect EBR on the negative bending region of beams to the support of the strengthened beam. If EBR is designed to pass around a column, EBR should be applied within a range of 4 times the slab thickness on both sides of the beam (**Figure 11**).

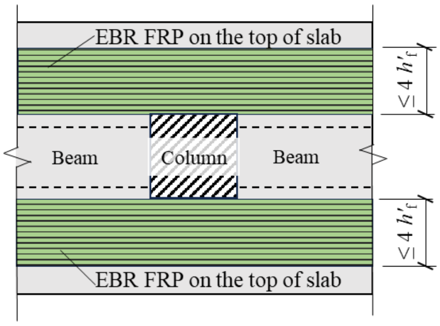


Figure 11 — The bonding range for strengthening the negative bending region of beams

1.10.1.4 Strength check for shear capacity

The shear capacity of the strengthened member shall be checked according to the provisions of **Section 1.7.2 of this document**. The shear capacity shall be larger than the shear demand corresponding to the ultimate flexural strength of the strengthened member.

1.10.2 Debonding failures and bond strength — shear

The following bond strength model may be used to determine the bond strength for EBR in shear strengthening:

$$f_{\mathrm{bfRd}} = \frac{\eta_{\mathrm{BA}}\alpha\beta_{\mathrm{w}}\beta_{\mathrm{L}}}{\gamma_{\mathrm{BA}}}\sqrt{\frac{E_{\mathrm{f}}}{t_{\mathrm{f}}}\sqrt{f_{\mathrm{ck}}}} \tag{71}$$

where

$\alpha = 0.315$

β_{w} is the FRP-to-concrete width ratio factor, determined by:

$$\beta_{\mathrm{w}} = \sqrt{\frac{2-\frac{b_{\mathrm{f}}}{s_{\mathrm{f}}\sin\alpha_{\mathrm{f}}}}{1+\frac{b_{\mathrm{f}}}{s_{\mathrm{f}}\sin\alpha_{\mathrm{f}}}}} \geq \frac{\sqrt{2}}{2} \tag{72}$$

β_{L} is the FRP bond length factor, determined by:

$$\beta_L = \begin{cases} 1 & \text{if } \lambda \geq 1 \\ \sin\left(\frac{\pi\lambda}{2}\right) & \text{if } \lambda < 1 \end{cases} \tag{73}$$

where λ is determined by **Formula (57) of this document**.

The units of parameters are: E_{f} – MPa; f_{ck} – MPa; t_{f} – mm.

1.11 Detailing of members and particular rules

1.11.1 Flexural strengthening with EBR

(1) The maximum centre-to-centre spacing $s_{\mathrm{f,max}}$ of FRP strips shall be restricted according to **Clause J.12.1 of EN 1992-1-1:2023**.

(2) The clear spacing of the FRP strips shall be less than the spacing of steel tension

reinforcement and 200 mm.

(3) The length between the section where the flexural strength contribution of FRP is fully utilised and the end of EBR should not be less than (L_f + 200 mm). L_f is the distance between the sections where the flexural strength contribution of FRP is fully utilised and not required, while L_f should not be less than 1/4 of the span of strengthened slab or 1/3 of the span of strengthened beam.

1.11.2 Shear strengthening with EBR

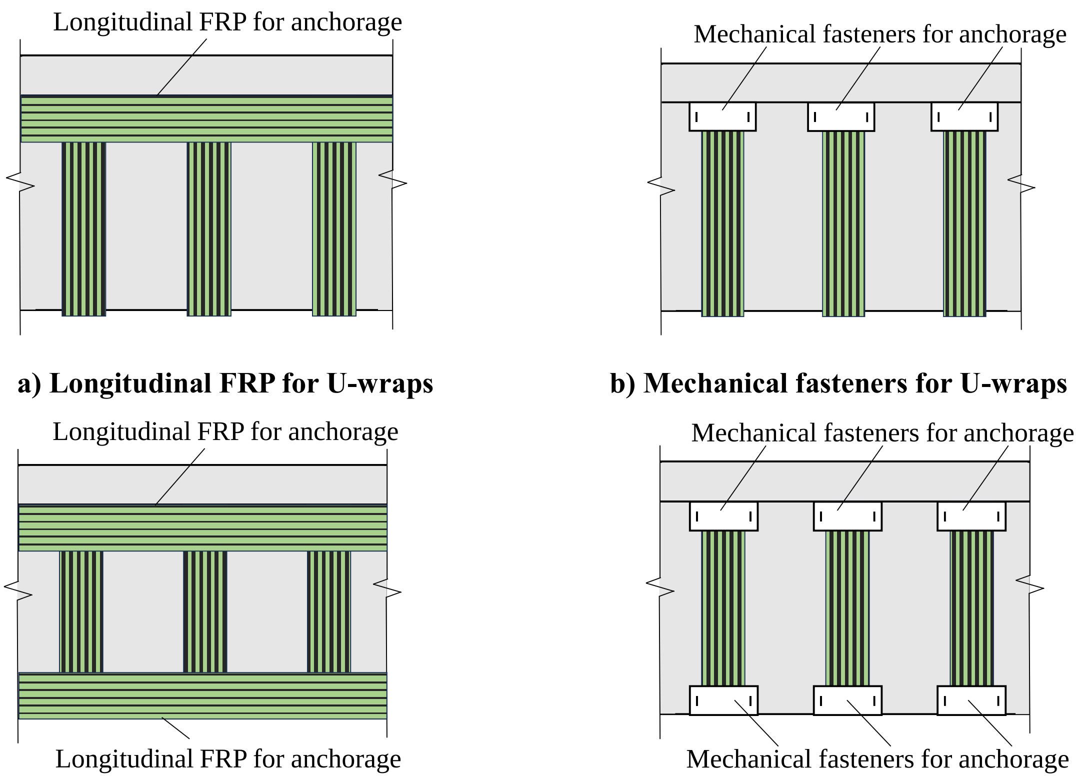
(1) The maximum clear strip spacing shall satisfy the following requirement:

$$\left(s_{\mathrm{f}}-\frac{b_{\mathrm{f}}}{\sin\alpha_{\mathrm{f}}}\right)\leq\min\left(h_{\mathrm{fe}}\frac{1+\cot\alpha_{\mathrm{f}}}{2},300\ \mathrm{mm}\right) \tag{74}$$

(2) For closed wrapping EBR shear strengthening schemes, the lap joints of EBR shall satisfy the requirements in **Clause 1.11.3 of this document**.

(3) For U-wrap EBR shear strengthening schemes, longitudinal FRP or mechanical fastener may be applied on the upper end of the EBR [**Figure 12(a),(b)**].

(4) For side bonding EBR shear strengthening schemes, longitudinal FRP or mechanical fastener may be applied on the upper and lower ends of the EBR [**Figure 12(c),(d)**].



a) Longitudinal FRP for U-wraps **b) Mechanical fasteners for U-wraps**

c) Longitudinal FRP for side bonding **d) Mechanical fasteners for side bonding**

Figure 12 — Anchorage measures for shear strengthening of beams with EBR

1.11.3 Compressive strengthening with EBR

(1) Any lap joint in the FRP jacket should include an adequate overlap, as determined by:

$$L_{\mathrm{l}} \leq \max\left(150\ \mathrm{mm}, \frac{f_{\mathrm{fuk}} t_{\mathrm{f1}}}{\tau_{\mathrm{ave}}}\right) \tag{75}$$

where

L_{l}	is the overlap length of EBR;
f_{fuk}	is the characteristic tensile strength of FRP;
t_{f1}	is the thickness of a single layer of FRP;
τ_{ave}	is the average interfacial shear strength of the lap joint and may be taken as 4 MPa.

(2)

When FRP jackets are composed of multiple layers applied separately, each layer must include a sufficient overlap in accordance with **Formula (75) of this document**. The lap joints of different layers should be spaced around the column's circumference, with a minimum separation equal to the lesser of 200 mm and $1.5L_{\mathrm{l}}$.

1.11.4 Surface preparation before strengthening

The requirements of surface preparation before strengthening are provided in **Clauses 6.2 and 6.4.2 of ACI PRC-440.2-23**.

1.12 Additional rules for precast concrete elements and structures

NOTE There are no additional requirements.

1.13 Lightly reinforced concrete structures

NOTE There are no additional requirements.

2 FRP-REINFORCED MARINE CONCRETE STRUCTURES

2.1 Purpose and Scope

BS EN 1992 is referred to in PWDM for the design of reinforced concrete structures. This document applies to the construction of new marine reinforced concrete structures assessed in accordance with **EN 1992-1-1:2023**, making use of glass fibre reinforcement (GFRP) reinforcement as an alternative to conventional steel reinforcement. Worked examples are provided in **Annex B** to illustrate the application of the design methods.

NOTE 1 This document applies only to GFRP reinforcement bars and mesh having external surface enhancement (e.g., protrusions, lugs, sand coatings, helical wrapping with fibres, deformations). Prestressed GFRP is not covered.

NOTE 2 This document does not apply to the use of GFRP reinforcement in lightweight concrete and concrete made with recycled aggregate.

NOTE 3 National choice is allowed in this standard where explicitly stated within notes. The National Annex to this standard contains the national choices to be used for buildings and civil engineering works constructed in the UK.

2.2 Terms, definitions and symbols

For the purposes of this document, the terms, definitions and symbols are given in sections of abbreviations, glossary of terms and glossary of symbols.

2.3 Verification- Partial factors for FRP reinforcement

(1) The partial factor for material γ_{FRP} given in **Table R.1 of EN 1992-1-1:2023** shall be used for FRP reinforcement.

(2) The loading considerations refer to **Clause 5 of PWDM Part 1-General Design Considerations for Marine Works**.

2.4 Materials

2.4.1 General

(1) Embedded FRP reinforcement materials suitable for design in accordance with this document shall comply with **Section 4 of this document**.
(2) **Clause R.5.1(3) of EN 1992-1-1:2023** shall be followed.

2.4.2 Properties

Clause R.5.2 of EN 1992-1-1:2023 shall be followed.

2.4.3 Design assumptions

(1) Design should be based on the nominal cross-section area of the reinforcement.
(2) The value of the design tensile strength of embedded FRP reinforcement shall be taken as:

$$f_{\mathrm{ftd}} = \frac{f_{\mathrm{ftk,100a}}}{\gamma_{\mathrm{FRP}}} \tag{76}$$

where

$f_{\mathrm{ftk,100a}}$ is the characteristic long-term strength. When not directly determined from production data, it can be obtained with **Formula (77) of this document**,

$$f_{\mathrm{ftk,100a}} = C_{\mathrm{t}} \cdot C_{\mathrm{e}} \cdot f_{\mathrm{ftk0}} \tag{77}$$

f_{ftk0} is the characteristic tensile strength of FRP reinforcement; for reinforcement with bent portion, its characteristic tensile strength shall be taken as 60% of that of straight bar unless more accurate values are determined.

C_{t} is the factor considering temperature effects:
C_t=1.0 for indoor and underground environments;
C_t=0.8 for outdoor members if heating through solar radiation cannot be excluded;

C_{e} is the coefficient between the strength after ageing and before ageing. The value shall be taken as 0.8 unless more accurate values are determined.

(3) The stress-strain relationship should be taken as illustrated in **Figure 13 of this document**.

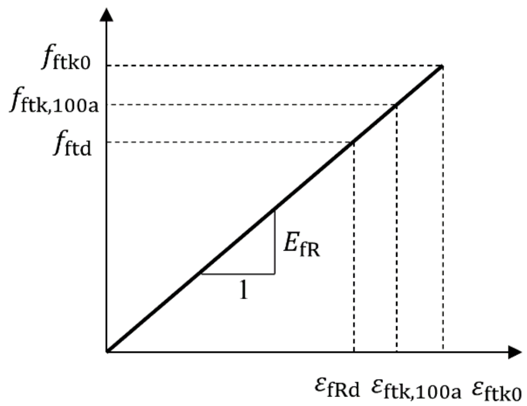


Figure 13 — Design stress-strain diagram for FRP reinforcement

(4) **Clauses R.5.3(5)-(6) of EN 1992-1-1:2023** shall be followed.

2.5 Durability – Concrete cover

(1) The value $c_{min,dur}$ = 0 for FRP reinforcement in **Formula (6.2) of EN 1992-1-1:2023**.
(2) Unless more accurate information is available based on test data, the cover for bond requirements and control of cracking due to thermal cycling for FRP reinforcement should be taken as $\mathrm{c_{min,bc}} \geq \min\ \{2\emptyset_\mathrm{f},\ 75\ \mathrm{mm}\}$. But at least the minimum cover for FRP reinforcement shall be taken as $\mathrm{c_{min,b}} \geq 1.5\emptyset_\mathrm{f}$ and $\mathrm{c_{min,b}} \geq 10$ mm.

NOTE Please seek advice when fire resistance is required.

2.6 Structural analysis

Clauses R.7(1)-(4) of EN 1992-1-1:2023 shall be followed.

2.7 Ultimate Limit States (ULS)

2.7.1 Bending with or without axial force

2.7.1.1 General

(1) When determining the ultimate moment resistance of reinforced concrete cross-sections, the following assumptions shall be made:

- plane sections remain plane;
- the tensile strength of concrete is ignored;
- the stresses in the reinforcing FRP are derived from the design stress-strain relationships in **Provisions of Clause 2.4.3 in this document**;

(2) The tensile strain in FRP reinforcement shall not exceed the design rupture strain, $\varepsilon_{\mathrm{fRd}}$.
(3) **Clause 22.2.3.3 of ACI 440.11-22** shall be followed.

2.7.1.2 Bending without axial force

(1) The flexural capacity of an FRP-reinforced flexural member is dependent on whether it is controlled by concrete crushing or FRP rupture. The controlling limit state can be determined by comparing the FRP reinforcement ratio to the balanced reinforcement ratio ρ_{fb}, which is a ratio where concrete crushing and FRP rupture occur simultaneously. ρ_{fb} shall be permitted to be calculated by:

$$\rho_{\mathrm{fb}} = \frac{\alpha_1 f_{\mathrm{cd}}}{f_{\mathrm{ftd}}} \cdot \xi_{\mathrm{b}} \tag{78}$$

$$\xi_b = \frac{\beta_1}{1 + f_{ftd}/(E_{fR}\varepsilon_{cu})} \tag{79}$$

where

f_{cd} is design value of concrete compressive strength;
f_{ftd} is design tensile strength of FRP reinforcement;
E_{fR} is design value of modulus of elasticity of FRP-reinforcement;
α_1 is the stress factor of the equivalent stress block for concrete in compression and taken as 1.0;
β_1 is the depth factor of the equivalent stress block for concrete in compression and taken as 0.8;
ξ_b is the corresponding ratio of the compression depth to the depth of cross-section;
ε_{cu} is ultimate compressive strain in the concrete.

To ensure a conservative design approach, a value of $1.5\rho_{fb}$ is adopted in this document as the threshold distinguishing the two failure modes.

(2) The stress in the FRP reinforcement (σ_{ftd}) under different reinforcement ratios can be calculated as follows:

$$\sigma_{ftd} = \begin{cases} f_{ftd} & (\rho_{lf} < 1.5\rho_{fb}) \\ (\rho_{lf}/\rho_{fb})^{-0.55} f_{ftd} & (\rho_{lf} \geq 1.5\rho_{fb}) \end{cases} \tag{80}$$

where

ρ_{lf} is the longitudinal reinforcement ratio for FRP reinforcement.

(3) For the two distinct failure modes, the design flexural strength (M_{Rd}) of the section under pure bending can be calculated as follows:

$$M_{Rd} = \varphi A_f \sigma_{ftd} (d_c - \frac{x_{sb}}{2}) \tag{81}$$

$$x_{sb} = \begin{cases} \left[\frac{0.14}{1 + 400(\sigma_{ftd}/E_{fR})} + \frac{\rho_{lf}\sigma_{ftd}}{f_{cd}}\right] d_c & (\rho_{lf} < 1.5\rho_{fb}) \\ \frac{\rho_{lf}\sigma_{ftd}}{\alpha_1 f_{cd}} d_c & (\rho_{lf} \geq 1.5\rho_{fb}) \end{cases} \tag{82}$$

where

φ is the reduction factor considering the brittleness of failure caused by FRP rebar rupture; φ = 0.85 when $\rho_{lf} < 1.5\rho_{fb}$, and φ = 1 when $\rho_{lf} \geq 1.5\rho_{fb}$
d_c is the effective depth of a cross-section;
x_{sb} is the depth of the equivalent rectangular stress block in the concrete compression zone.

2.7.1.3 Bending with axial force

(1) Design value of axial compressive strength, N_{Ed}, shall not exceed $N_{c,max}$, in accordance with **Table 3 of this document**, where $N_{c,0}$ is design value of axial compressive strength at zero eccentricity and is calculated by **Formula (83) of this document**.

Table 3 — Maximum axial strength $N_{c,max}$

Transverse reinforcement	$N_{c,max}$
Ties	$0.80N_{c,0}$
Spirals	$0.85N_{c,0}$

(2) $N_{c,0}$ shall be calculated by

$$N_{c,0} = 0.85f_{cd}A_c \tag{83}$$

where

A_c is cross-sectional area of concrete.

(3) For cross-sections loaded by an axial compression force N_{Ed}, the minimum moment shall be determined in accordance with **Formula (8.1) of EN 1992-1-1:2023**.

(4) **Clause R.8.1 (3) of EN 1992-1-1:2023** shall be followed.

(5) Design value of axial tensile strength, N_{Ed}, shall not be taken greater than the maximum value $N_{t,max}$, calculated by:

$$N_{t,max} = f_{ftd}A_{fl} \tag{84}$$

where

A_{fl} is cross-sectional area of longitudinal FRP reinforcement.

(6) Slenderness effects shall be permitted to be neglected if (a) or (b) is satisfied:

(a) For compressive members not braced against sidesway

$$l_0/i \leq 17 \tag{85}$$

(b) For compressive members braced against sidesway

$$l_0/i \leq 29 + 12(M_1/M_2) \tag{86}$$

and

$$l_0/i \leq 35 \tag{87}$$

where

l_0 is effective length of column;

M_1/M_2 is negative if the compressive member is bent in single curvature, and positive for double curvature; $|M_1| \leq |M_2|$.

(7) If slenderness effects are not permitted to be neglected, the design moment for eccentrically compressed members considering the slenderness effect shall be calculated as:

$$M = C_m\eta_{ns}M_2 \tag{88}$$

$$C_m = 0.7 + 0.3\frac{M_1}{M_2} \tag{89}$$

$$\eta_{ns} = 1 + \frac{1}{1000\left(\frac{M_2}{N} + e_a\right)/d_f}\left(\frac{l_0}{h}\right)^2 \zeta_c \tag{90}$$

$$\zeta_c = \frac{0.5f_{cd}A}{N} \tag{91}$$

where

M is the design value of moment, kN·m;
C_m is the eccentricity adjustment factor for the end section of the member, which shall be taken as 0.7 when it is calculated to be less than 0.7.
η is the moment amplification factor;
N is the axial force design value corresponding to the moment value M_2, kN;
e_a is the additional load eccentricity, taken as the greater of 20 mm and 1/30 of the maximum sectional dimension in the direction of eccentricity;
ζ_c is the section curvature correction factor, which shall be taken as 1.0 when the calculated value exceeds 1.0;
A is the gross sectional area.

(8) The flexural strength of eccentrically loaded members in flexure-compression shall be checked using **Formula (92) of this document.**

$$\begin{cases} N_{\mathrm{Ed}} = \alpha_1 f_{\mathrm{cd}} b_{\mathrm{w}} x_{\mathrm{sb}} - A_{\mathrm{f}} \sigma_{\mathrm{ftd}} \\ N_{\mathrm{Ed}} e \leq \alpha_1 f_{\mathrm{cd}} b_{\mathrm{w}} x_{\mathrm{sb}} \left(d_{\mathrm{f}} - \dfrac{x_{\mathrm{sb}}}{2} \right) \\ \sigma_{\mathrm{ftd}} = \varepsilon_{\mathrm{cu}} (\dfrac{\beta_1 d_{\mathrm{f}}}{x_{\mathrm{sb}}} - 1) E_{\mathrm{fR}} \leq f_{\mathrm{ftd}} \end{cases} \tag{92}$$

where

e is the eccentricity, calculated as

$$e = e_0 + e_{\mathrm{a}} + \frac{h}{2} - a_{\mathrm{f}} \tag{93}$$

where

e_0 is the eccentricity (mm) of the axial force relative to the centroid of the section, taken as M/N;
a_f is the distance from the resultant force point of the longitudinal tension reinforcement to the tension face of the section.

2.7.2 Shear

(1) **Clauses R.8.2 of EN 1992-1-1:2023 shall be followed.**
(2) A maximum value of 0.005 shall be applied to **Formula (R.7) of EN 1992-1-1:2023**.

2.7.3 Torsion

Clauses R.8.3 (1)-(2) of EN 1992-1-1:2023 shall be followed.

2.7.4 Punching

R.8.4 of EN 1992-1-1:2023 shall be followed.

2.7.5 Design with strut-and-tie models and stress fields

R.8.5 of EN 1992-1-1:2023 shall be followed.

2.8 Serviceability Limit States (SLS) – Special rules for FRP reinforcement

2.8.1 General

(1) **Clauses 2.8.2 and 2.8.3 of this document** covers the common serviceability limit states.
(2) The stresses and strains of the GFRP-reinforced concrete section under service loads can be calculated using elastic analysis based on an uncracked section or a cracked section.
(3) The service stress of GFRP bars under sustained load shall be evaluated with an elastic cracked section analysis, while the value of GFRP service stress shall not exceed $C_{\mathrm{C}} \cdot f_{\mathrm{ftk,100a}}$, where the value of C_{C} shall be taken as 0.3 unless more accurate values are determined.

2.8.2 Crack control

(1) Crack control shall be implemented for GFRP-reinforced slabs and beams by imposing constraints on the distribution of GFRP flexural reinforcement.
(2) The spacing of reinforcement nearest to the tension face shall not exceed the following limit:

$$s \leq \min\left\{\frac{0.677E_{\mathrm{fR}}}{f_{\mathrm{fs}}} - 2.5c; \frac{0.550E_{\mathrm{fR}}}{f_{\mathrm{fs}}}\right\} \tag{94}$$

where

- c is the concrete cover, i.e., the least distance from surface of reinforcement to the tension face, mm;
- f_{fs} is the calculated stress in reinforcement closest to the tension face at service loads, which shall be calculated based on an elastic cracked section analysis using the moment due to service loads M_{Ec}.

(3) The bar stress f_{fs} shall not exceed the following limit:

$$f_{\mathrm{fs}} \leq \frac{0.296E_{\mathrm{fR}}}{d_{\mathrm{c}}\beta_{\mathrm{cr}}} \tag{95}$$

where

- d_{c} is the thickness of concrete cover measured from extreme tension fibre to centre of bar location closest thereto, mm;
- β_{cr} is the ratio of the distance from the elastic cracked section neutral axis to the extreme tension fibre to the distance from the elastic cracked section neutral axis to the centroid of the longitudinal tensile reinforcement.

2.8.3 Deflection control

(1) **Clauses 9.3.1 (1)-(2) of EN 1992-1-1:2023** shall be followed.

(2) The long-term flexural stiffness, B, of a cracked section under the quasi-permanent combination of actions may be calculated using **Formula (96) of this document**:

$$B = \frac{B_s}{\theta} \tag{96}$$

where

B_s is the short-term flexural stiffness of the cracked section;

θ is a coefficient accounting for the increase in deflection due to long-term effects. The value of θ may be taken as 2.0. For T-sections with the flange in tension, θ may be increased by 20%. Where reliable engineering data or test results are available, θ may be determined accordingly.

(3) The short-term flexural stiffness, B_s, of a cracked FRP-reinforced section may be calculated as follows:

$$B_s = \frac{E_{fR} A_f d_f^2}{1.15\psi_f + 0.2 + \frac{6\alpha_{fE}\rho_f}{1 + 3.5\gamma_f'}} \tag{97}$$

where

ψ_f is the coefficient for non-uniformity of strain in tension FRP reinforcement;

α_{fE} is the ratio of the modulus of elasticity, E_{fR}/E_{cm};

ρ_f is the tensile FRP reinforcement ratio, $A_f/(b_w \cdot d_f)$;

γ_f' is the ratio of the flange area in compression to the effective area of the web, calculated by $\gamma_f' = [(b_f' - b_w)h_f']/(b_w \cdot d_f)$.

(4) The coefficient ψ_f shall be calculated from:

$$\psi_f = 1.1 - 0.65 \cdot \frac{f_{ctm}}{\rho_{p,eff} \cdot \sigma_f} \cdot \frac{E_{fR}}{E_s} \tag{98}$$

where

$\rho_{p,eff}$ is the effective reinforcement ratio, calculated by

$$\rho_{p,eff} = \frac{A_f}{A_{c,eff}} \tag{99}$$

σ_f is the tensile stress in the longitudinal FRP reinforcement under the quasi-permanent combination of actions, estimated by,

$$\sigma_f = \frac{M_{Eq}}{0.9 \cdot A_f \cdot d_f} \tag{100}$$

$A_{c,eff}$ is the effective tension area of concrete, which shall be taken as $0.5b_w h+(b_f-b_w)h_f$, where b_f and h_f are the width and height of tension flange, respectively.

E_s is the elastic modulus of the steel reinforcement (MPa). Where test data is unavailable, a value of 200 GPa may be used.

The value of ψ_f shall not be taken as less than 0.2 nor greater than 1.0. For members directly subjected to cyclic loading, ψ_f shall be taken as 1.0.

2.9 Fatigue

This document does not provide rules for fatigue utilising FRP reinforcement.

2.10 Detailing of FRP reinforcement

2.10.1 General

R.11.1(1) of EN 1992-1-1:2023 shall be followed.

2.10.2 Spacing of bars

(1) The clear distance c (horizontal and vertical) between individual parallel bars should be not less than the maximum value of ϕ_{f}, D_{upper}+5 mm, and 25 mm, where D_{upper} is the largest value of the upper sieve size *D*.
(2) For longitudinal reinforcement in columns, pedestals, struts, and boundary elements in walls, clear spacing between bars should be not less than the maximum value of $1.5\phi_{\mathrm{f}}$, D_{upper}+5 mm, 40 mm.

2.10.3 Permissible mandrel diameters for bent bars

R.11.3 of EN 1992-1-1:2023 shall be followed.

2.10.4 Anchorage of FRP reinforcement in tension and compression

(1) **11.4.1 (2) of EN 1992-1-1:2023** shall be followed.
(2) Tension bar anchorage using standard hook shall employ the geometries and dimensions specified in **Table 25.3.1 of ACI 440.11-22**.
(3) Hooks shall not be used for anchorage of compression reinforcements.
(4) Anchorage length l_{bd} for bars in tension/compression shall be the greatest of (a), (b) (c), and (d):

(a)

$$\frac{\phi_{\mathrm{f}}\left(\frac{\sigma_{\mathrm{ftd}}}{0.083\sqrt{f_{\mathrm{ck}}}}-340\right)\psi_{\mathrm{t}}}{13.6+\frac{c_{\mathrm{b}}}{\phi_{\mathrm{f}}}} \qquad (101)$$

where

$c_{\mathrm{b}}/\phi_{\mathrm{f}}$ shall not be taken greater than 3.5;
c_{b} is lesser of: (a) concrete cover of reinforcement (to the surface of the bar),

and (b) one-half the centre-to-centre spacing of bars being developed;

σ_{ftd} is the stress in the bar required to develop the full sectional strength;

ψ_{t} is reinforcement location factor and shall be 1.5 if more than 300 mm of fresh concrete is placed below horizontal reinforcement being developed and 1.0 for all other cases.

(b) $20\phi_{\mathrm{f}}$

(c) 300 mm

(d) $\frac{f_{\mathrm{ftd}}}{8f_{\mathrm{ctd}}}\phi_{\mathrm{f}}$ (102)

where

f_{ftd} is design value of the tensile strength of concrete.

(5) **Clause 25.4.3.1 of ACI 440.11-22** shall be followed. f_{fu} in **Clause 9.7.7.4 of ACI 440.11-22** may be replaced by $f_{\mathrm{ftk,100a}}$.

(6) If the cover normal to the plane of the hook exceeds 64 mm and the cover extension beyond the hook is at least 50 mm, the calculation of $l_{\mathrm{bd,tot}}$ is permitted to be multiplied by 0.7.

(7) **Clause 25.4.10.1 of ACI 440.11-22** shall be followed.

2.10.5 Laps of FRP reinforcement in tension

(1) Lap splices shall not be permitted for bars larger than 32 mm.

(2) If bars of different size are lap spliced, the design value of lap length for GFRP reinforcement shall be based on the design value of anchorage length of the larger bar.

(3) The design value of lap length of GFRP reinforcement shall be the greatest of **Formula (101) and (102) of this document.**

(4) **Clauses 25.5.7.3** and **25.5.7.4 of ACI 440.11-22** shall be followed.

2.10.6 Post-tensioning tendons

NOTE This document does not apply to the use of FRP reinforcement as post-tensioning tendons.

2.10.7 Deviation forces due to curved tensile and compressive chords

Clause 11.7 of EN 1992-1-1:2023 shall be followed.

2.10.8 Transverse reinforcement

The clauses specified in **Sections 25.7.1-25.7.2 of ACI 440.11-22** shall be followed. When using **Clause 25.7.1.3 of ACI 440.11-22**, f_{fu} in **Clause 9.7.7.4 of ACI 440.11-22** may be replaced by $f_{\mathrm{ftk,100a}}$. l_d may be replaced by l_{bd}, f_{fu} may be replaced by σ_{ftd}, f_{ft} is taken as the smaller of design tensile strength of FRP reinforcement with bent portion [Formulae (76), (77)] and $0.005E_{\mathrm{fR}}$.

2.11 Detailing of members and particular rules

2.11.1 General

R.12.1 of EN 1992-1-1:2023 shall be followed.

2.11.2 Minimum reinforcement rules

(1) **Clauses R.12.2 (1)-(3) of EN 1992-1-1:2023** shall be followed.
(2) The minimum ratio of FRP reinforcement area to gross concrete area shall not be less than $140/E_{\mathrm{fR}}$.
(3) The spacing of minimum FRP reinforcement shall not exceed the lesser of $3h$ and 300 mm.

2.11.3 Beams

(1) **R.12.3 of EN 1992-1-1:2023** shall be followed.
(2) **Clauses 9.7.7.3** and **9.7.7.4 of ACI 440.11-22** shall be followed. f_{fu} in **Clause 9.7.7.4 of ACI 440.11-22** is replaced by $f_{\mathrm{ftk,100a}}$.

2.11.4 Slabs

2.11.4.1 General
R.12.4 of EN 1992-1-1:2023 shall be followed.

2.11.4.2 Two-way slabs
Clauses 8.3.6.1 and **8.4.2.2.2 of ACI 440.11-22** shall be followed.

2.11.5 Slabs-column connections and column bases

Clauses 15.4.2.2, 16.3.4.1 and **16.3.5.2 of ACI 440.11-22** shall be followed.

2.11.6 Columns

(1) Longitudinal and hoop reinforcement of columns should be detailed in accordance with the

requirements of **Table 4 of this document**.

Table 4 — Detailing requirements for reinforcement in columns

Description		Symbol	Requirement
1	Maximum amount of longitudinal reinforcement		$0.08A_c$
2	Minimum amount of longitudinal reinforcement		$0.01A_c$
3	Minimum number of longitudinal bars[a]		— Three within triangular ties — Four within rectangular or circular ties — Six enclosed by spirals
4	Maximum spacing of transverse reinforcement (stirrups/hoops) for columns with dimensions h and b:	$s_{\mathrm{max,col}}$	—where $\tau_{\mathrm{Rd,f}} \leq 0.33\sqrt{f_{\mathrm{ck}}}$,$\min\{h/2; 600\ \mathrm{mm}\}$ — where $\tau_{\mathrm{Rd,f}} > 0.33\sqrt{f_{\mathrm{ck}}}$, $\min\{h_c/4; 300\ \mathrm{mm}\}$
[a] For constructability, the diameter of longitudinal bars $\emptyset_{\mathrm{l,max}}$ should be at least 12 mm.			

(2) **Clauses 10.3.1.3**, **10.3.2.1**, **10.7.5.2.1 of ACI 440.11-22** and **12.6(2) of EN 1992-1-1:2023** shall be followed. f_{fu} in **Clause 10.7.5.2.1 of ACI 440.11-22** is replaced by $f_{\mathrm{ftk,100a}}$.

(3) The strength of the column in the direction of the load eccentricity and that of the column in the direction perpendicular to the load eccentricity should both be calculated, and the smaller value of the two should be taken as the design strength of the column.

2.11.7 Walls

(1) For walls subjected predominantly to out-of-plane bending, **Clause 2.11.4 of this document shall be followed**.

(2) Vertical, horizontal and orthogonal-to-the-surface reinforcement in walls, should be detailed in accordance with the requirements of **Table 5 in this document**.

Table 5 — Detailing requirements for reinforcement in walls

Description		Symbol	Requirement
1	Minimum amount of longitudinal reinforcement: — where shear reinforcement is not required for shear resistance — where shear reinforcement is required for shear resistance	$A_{\mathrm{f,min,v}}$	$140A_c/E_{\mathrm{fwR}}{}^{a}$ $260A_c/E_{\mathrm{fwR}}$
2	Minimum amount of transverse reinforcement: — where shear reinforcement is not required for shear resistance — where shear reinforcement is required for shear resistance	$A_{\mathrm{f,min,h}}$	$0.0025A_c{}^{a}$ $0.0025A_c$

3	Maximum spacing of longitudinal reinforcement		$\min\{3h_{\mathrm{w}}{}^{b}; 250\ \mathrm{mm}\}^{c}$
4	Maximum spacing of transverse reinforcement		$\min\{3h_{\mathrm{w}}{}^{b}; 250\ \mathrm{mm}\}^{d}$
[a] These limits need not be satisfied if adequate strength and stability can be demonstrated by structural analysis. [b] h_{w} — thickness of wall. [c] If shear reinforcement is required for in-plane strength, spacing of longitudinal reinforcement shall not exceed one third of wall length. [d] If shear reinforcement is required for in-plane strength, spacing of longitudinal reinforcement shall not exceed one fifth of wall length.			

(3) For axial load and out-of-plane flexure, if the resultant of all loads is located within the middle third of the thickness of a solid wall with a rectangular cross-section, N_{Ed} shall be permitted to be calculated by:

$$N_{\mathrm{Ed}} = 0.45 f_{\mathrm{cd}} A_{\mathrm{c}} [1 - (\frac{k_{\mathrm{el}} l}{32h})^{2}] \tag{103}$$

Effective length factor k_{el} shall be in accordance with **Table 11.5.3.2 of ACI 440.11-22** and l is length of wall, measured centre-to-centre of the joints.

(4) **Clauses 11.2.3.1, 11.7.2.3, 11.7.4.1 and 11.7.5.1 of ACI 440.11-22** shall be followed. f_{fu} in **Clause 11.7.5.1 of ACI 440.11-22** may be replaced by $f_{\mathrm{ftk,100a}}$.

2.11.8 Pile caps

Clauses 13.2.6.6, **13.4.6.2** and **13.4.6.5 of ACI 440.11-22** shall be followed.

2.11.9 Tying systems for robustness of buildings

R.12.9 of EN 1992-1-1:2023 shall be followed.

2.11.10 Supports, bearings and expansion joints

R.12.10 of EN 1992-1-1:2023 shall be followed.

2.12 Additional rules for precast concrete elements and structures

R.13 of EN 1992-1-1:2023 shall be followed.

2.13 Lightly reinforced concrete structures

NOTE There are no additional requirements.

3. MATERIAL SPECIFICATION ON STRENGTHENING PRODUCTS FOR MARINE STRUCTURES

3.1 Specification

3.1.1 Scope

This specification specifies requirements for permitted constituent materials, performance requirements of those constituent materials, and performance requirements for CFRP and GFRP laminae fabricated using the wet layup process.

This specification covers fabric reinforcement and saturating resin that comprise the FRP system. This specification applies to FRP laminae consisting of continuous, unidirectional fibre reinforcement and saturating resin produced via the wet layup process.

3.1.2 Terms and Definitions

For the purposes of this document, specification, the terms and definitions are given in the glossary of terms.

3.1.3 Materials

The constitutive materials of the strengthening system shall meet the requirements herein.

3.1.3.1 Fibres

Fibre reinforcement should be organised in a practical format, either as continuous glass fibre strands or carbon fibre bundles, each with a uniform size and weight. The fabric producer will arrange these strands or bundles into fabric reinforcement, ensuring they are oriented in one consistent direction along the fabric's length. Fibres of any material may be used at an angle to the strands or bundles to maintain the fabric's integrity.

3.1.3.2 Resins

Only epoxy resins shall be used to saturate the resin. The resins should remain undiluted and shall not be mixed with organic solvents like thinner.

3.1.4 Physical Properties

3.1.4.1 Glass Transition Temperature

The mechanical properties of FRP lamina may decrease significantly when the temperature is beyond the glass transition temperature. The glass transition temperature of FRP lamina, as determined in accordance with ASTM E1640-18, shall not be less than 60°C.

3.1.4.1 Areal weight

The areal weight of dry fabric shall be determined by measuring the mass per unit area in accordance with ASTM D3776/D3776M-20 and shall be reported in g/mm^2.

3.1.5 Mechanical Properties

3.1.5.1 Tensile properties of saturating resins

The tensile properties of resin shall be determined in accordance with ASTM D638-14. The tensile strength of resin shall not be less than 41.4 MPa, while the tensile modulus of elasticity and ultimate tensile strain shall not be less than 1.72 GPa and 3%, respectively.

3.1.5.2 Flexural properties of saturating resins

The flexural strength and modulus of resin, as determined in accordance with ASTM D790-17, shall not be less than 68.9 MPa and 1.72GPa, respectively.

3.1.5.3 Tensile properties of FRP lamina

The tensile properties of FRP lamina shall be determined in accordance with ASTM D7565/D7565M-10 and shall be reported on a per-ply basis.

The tensile strength per unit width of FRP lamina shall be divided by the areal weight of the dry fabric as determined in **Section 3.1.4.1 of this document** and shall not be less than 878 $kN/mm/(g/mm^2)$ for carbon and 323 $kN/mm/(g/mm^2)$ for glass.

The chord tensile stiffness per unit width of FRP lamina shall be divided by the areal weight of the dry fabric as determined in **Section 3.1.4.1 of this document** and shall not be less than 103,295 $kN/mm/(g/mm^2)$ for carbon and 21,485 $kN/mm/(g/mm^2)$ for glass. To determine the rupture strain, divide the ultimate tensile force per unit width by the average chord tensile stiffness per unit width.

The rupture strain shall be calculated as the ultimate tensile force per unit width divided by the mean chord tensile stiffness per unit width.

3.1.6 Durability Properties

3.1.5.1 FRP lamina

The FRP lamina shall conform to the retained tensile force per unit width requirements shown in **Table 8.1 of ACI 440.8-13**.

3.2 Manufactures Inspection and Testing

3.2.1 Sampling

3.2.1.1 Areal weight

A minimum of five specimens shall be tested from each of the five fabric specimens.

3.2.1.2 Glass Transition Temperature

A minimum of five specimens shall be tested from five FRP single- or double-ply lamina panels, which shall be prepared in accordance with ASTM D7565/D7565M-10. Each test should include at least one specimen from each of the five panels.

3.2.2 Mechanical Properties

3.2.2.1 Tensile and flexural properties of saturating resins

Each property should be determined as the average of test results from a minimum of five specimens. Testing shall be conducted at least once for each resin product and whenever there are changes to the constituent materials.

3.2.2.2 Tensile properties of FRP lamina

A minimum of 20 specimens shall be tested from four FRP single- or double-ply lamina, in accordance with ASTM D7565/D7565M-10. A minimum of five specimens shall be taken from each of the four panels. For each specimen, tensile stiffness per unit width and the tensile force per unit width shall be measured, and the results on a per-ply basis shall be reported.

The ultimate tensile force per unit width shall be determined as the average of the test results minus three times the standard deviation. The chord tensile stiffness per unit width should be determined as the average of the test results.

3.2.3 Durability Properties

3.2.3.1 FRP lamina

To evaluate strength retention under specified exposure conditions, specimens shall be taken from five FRP single- or double-ply lamina panels, prepared in accordance with ASTM D7565/D7565M-10. Test five specimens before exposure and five specimens after each exposure condition. Ensure that all specimens have the same number of plies. The average values of the test results for the five unexposed and five exposed specimens shall be used to calculate percent retention.

3.2.4 Property limits and test methods

3.2.4.1 Fibre

The areal weight shall be measured and reported, in accordance with ASTM D3776/D3776M-20.

3.2.4.2 Resin

Tensile strength, tensile modulus of elasticity, ultimate tensile strain, flexural strength, and flexural modulus shall be tested according to the specified methods and must conform to the minimum requirements specified in **Table 7.1.2 of ACI 440.8-13**.

3.2.4.3 FRP lamina

The glass transition temperature, tensile force per unit width, chord tensile stiffness per unit width, retained tensile force per unit width shall be tested according to the specified methods and must conform to the minimum requirements specified in **Table 7.2.1 of ACI 440.8-13**.

3.2.5 Manufacturer record and delivery documentation

The manufacturer shall establish and maintain the records required and their delivery documentation accordingly.

For each delivery, the manufacturer shall supply the following information:

a) Manufacturer's name and place of production;
b) Name of the FRP material (manufacturer's description);
c) Physical properties of the fabric and lamina;
d) Mechanical properties of the lamina;
e) Mechanical properties of the saturating resin;

f) Details of any pretreatments applied when assessing the physical or mechanical properties of the lamina;
g) Information regarding product packaging;
h) Instructions for mixing the resin;
i) Estimated pot life of the saturating resin, considering ambient temperature;
j) Shelf life of all constituent materials;

3.3 Purchasers Testing

3.3.1 Sampling

A minimum of five random specimens shall be obtained from each batch of materials delivered to site to determine each of the property limits.

3.3.2 Property limits and test methods

3.3.2.1 Fibre

The areal weight shall be measured and reported, in accordance with ASTM D3776/D3776M-20.

3.3.2.2 Resin

Tensile strength, tensile modulus of elasticity, and ultimate tensile strain shall be tested according to the specified methods and must conform to the minimum requirements specified in **Table 7.1.2 of ACI 440.8-13.**

3.3.2.3 FRP lamina

The glass transition temperature, tensile force per unit width and chord tensile stiffness per unit width shall be tested according to the specified methods and must conform to the minimum requirements specified in **Table 7.2.1 of ACI 440.8-13**.

3.3.3 Test reports

The test results shall be reported in an endorsed test report bearing a HKAS symbol or its Mutual Recognition Arrangement (MRA) partner.

3.4 Handling, Storage and Installation

3.4.1 Handling

The handle requirements from **Section 5.3 of ACI 440.2-23** shall be followed, including safety data sheet, information sources, general handline hazards, personnel safe handling and clothing and clean-up and disposal.

3.4.2 Storage

The handle requirement from **Section 5.2 of ACI 440.2-23** shall be followed.

3.4.3 Installation

The installation procedures and requirements from **Chapter 6 of ACI 440.2-23** shall be followed, including contractor competency, temperature, humidity and moisture considerations, equipment, substrate repair and surface preparation, mixing of resins, application of FRP systems, alignment of FRP systems, multiple plies and lap splices, curing of reins and temporary protection.

3.4.4 Evaluation of adhesion strength for bond-critical application

For bond-critical applications, tension adhesion testing of cored samples should be conducted in accordance with ASTM D7522/D7522-21, as specified in **Section 3.5.2 of ACI 440.12-22**.

4. MATERIAL SPECIFICATION ON FRP REINFORCING BARS FOR MARINE STRUCTURES

4.1 Specification

4.1.1 Scope

This specification specifies requirements for glass fibre reinforced polymer (GFRP) reinforcing bars provided in cut lengths and bent shapes, with an external surface enhancement for concrete reinforcement. This specification covers GFRP bars delivered in the form of reinforcing bars and contains provisions for the physical, mechanical, and durability properties.

4.1.2 Terms and Definitions

For the purposes of this document, specification, the terms and definitions are given in section of glossary of terms.

4.1.3 Materials and Manufacture

4.1.3.1 Fibres

The reinforcing fibre shall be in the form of continuous unidirectional rovings. Fibre sizing and coupling agents should be compatible with the resin system used.

4.1.3.2 Resins

Vinylester and epoxy resin systems are permitted, provided the finished product meets all requirements of this specification. The base polymer in the resin system should not contain any polyester.

4.1.3.3 Manufacturing Process

The specified manufacturing process in **Section 5.3 of ASTM D7957/D7957M-22** shall be followed.

4.1.4 Physical Properties

4.1.4.1 Fibre Content

The fibre content of GFRP reinforcing bars, as determined in accordance with ASTM D3171-22 (by volume) or ASTM D2584-18 (by mass), shall not be less than 55% by volume or 70% by mass.

4.1.4.2 Glass Transition Temperature

The glass transition temperature of GFRP reinforcing bars, as determined in accordance with ASTM E1356-08 (differential scanning calorimetry method) or ASTM E1640-18 (dynamic mechanical analysis method), shall not be less than 100°C.

4.1.4.3 Degree of Cure

The degree of cure of GFRP reinforcing bars, as determined in accordance with ASTM E2160-04, shall not be less than 95%.

4.1.4.4 Bar Sizes

The range of nominal diameter of GFRP reinforcing bar shall be 6 mm to 32 mm. The values for the nominal cross-sectional area shall follow Table 3 of ASTM D7957/D7957M-22. The mechanical properties reported according to this specification shall be calculated based on the nominal cross-sectional area.

The measured cross-sectional area of the bar shall be determined in accordance with ASTM D7205/D7205M-06 and shall be measured on the final product of the GFRP reinforcing bar with the surface enhancements. The measured cross-sectional area shall be within the range specified in **Table 3 of ASTM D7957/D7957M-22**.

4.1.5 Mechanical Properties

4.1.5.1 Tensile properties

The tensile properties of GFRP reinforcing bar shall be determined in accordance w**ith ASTM D7205/D7205M-06**. The tensile strength of GFRP reinforcing bar shall comply with the specified values given in **Table 6**, while the tensile modulus of elasticity and ultimate tensile strain shall not be less than 44.8 GPa and 1.1%, respectively.

Table 6 –Tensile property limits

Nominal diameter (mm)	Minimum tensile strength (MPa)
6	954.9
10	751.2
13	723.3
16	646.6
19	641.9
22	634.0
25	605.0
29	552.6
32	543.4

4.1.5.2 Transverse shear strength

The transverse shear strength of GFRP reinforcing bars, as determined in accordance with ASTM D7617/D7617M-11, shall not be less than 131 MPa.

4.1.5.3 Bond strength with concrete

The bond strength of GFRP reinforcing bars with concrete, as determined in accordance with ASTM D7913/D7913M-14, shall not be less than 9.6 MPa, at which the slip at the loaded end is limited to 0.5mm. If the slip at ultimate bond stress exceeds 0.5 mm, the bond stress corresponding to a slip of 0.5 mm shall be reported.

4.1.6 Durability Properties

4.1.6.1 Moisture Absorption

The moisture absorption of GFRP reinforcing bars shall be determined in accordance with Section 8.4 of ASTM D570-22 with water temperature of 50°C. The values of moisture absorption shall not exceed 1.0% for saturated conditions and 0.25% for 24-hour immersion.

4.1.6.2 Resistance to Alkaline Environment

The resistance to alkaline environment of GFRP reinforcing bars shall be determined in accordance with ASTM D7705/D7705M-12. The mean retention of the tensile strength shall not be smaller than 80% of the initial mean ultimate tensile strength, after 90-day immersion with a solution temperature of 60°C.

4.1.6.3 Requirements of bent GFRP reinforcing bars

Bends in GFRP reinforcing bars shall be formed only while the resin is in a physical liquid state. The minimum inside bend diameters shall follow **Table 4 of ASTM D7957/D7957M-22**. The tensile strength of the bent portion of GFRP reinforcing bars, as determined by **ASTM D7914/D7914M-21**, shall not be less than 60% of the values in **Table 6 of this document**.

NOTE Requirements of bent GFRP reinforcing bars with nominal diameters of 29 mm and 32 mm are not included in this specification. Where use of bent GFRP reinforcing bars of nominal diameters 29 mm and 32 mm is unavoidable, the Contractor shall submit project-specific qualification data demonstrating that the bent portion retains at least 60 % of the straight-bar tensile force, together with the proposed minimum inside bend radius, for the Engineer’s approval.

4.1.7 Traceability

Each delivered batch of straight GFRP bars shall be identifiable and traceable to the manufacturer and its production data by permanent marking at regular intervals along the length, spaced not more than 2m between intervals. Traceability documentation shall include:

1. Manufacturer/ product name
2. Nominal diameter
3. Unique batch/lot number

Bent GFRP bars shall include the above information by an attached, weather-resistant tag only and shall additionally include the shape description.

4.2 Manufactures Inspection and Testing

4.2.1 Sampling

For the determination of the average and characteristic properties of GFRP reinforcing bars, a minimum of 24 specimens shall be obtained in groups of eight or more from three or more different production lots. The average and characteristic properties shall satisfy the limits as given in **Table 7 of this document**. Tests for qualification shall be repeated if there is a process or constituent material change.

4.2.2 Property limits and test methods

Bars manufactured according to this specification shall be qualified using the specified test methods and shall meet the requirements given by **Table 7 of this document**.

Table 7 – Property limits and test methods for manufactures inspection

Property	Limit	Test Method
Fibre content	Average value ≥70 % (by volume) or ≥55% (by mass)	ASTM D2584-18/ ASTM D3171-22
Glass transition temperature	Average value ≥100 °C	ASTM E1356-08/ ASTM E1640-18
Degree of cure	Average value ≥ 95 %	ASTM E2160-04
Measured cross-sectional area	Average value within the given ranges in **Table 3 of ASTM D7957/D7957M-22**	ASTM D7205/D7205M-06
Tensile strength	Characteristic value≥ values in **Table 6 of this document**	ASTM D7205/D7205M-06

Tensile modulus of elasticity	Average value ≥ 44.8 GPa	ASTM D7205/D7205M-06
Ultimate tensile strain	Average value ≥ 1.1 %	ASTM D7205/D7205M-06
Transverse shear strength	Characteristic value≥131 MPa	ASTM D7617/D7617M-11
Bond strength	Characteristic value≥9.6 MPa at the maximum slip of 0.5 mm	ASTM D7913/D7913M-14
Moisture absorption[a]	Average value ≤ 1.0 % (saturation)	ASTM D570-22
Alkaline resistance[b]	Average value ≥ 80 % of initial average tensile strength	ASTM D7705/D7705M-12
Tensile strength of bent portion of bar	Characteristic value ≥ 60 % of the value in **Table 6 of this document**	ASTM D7914/D7914M-21
[a] Immersion in water at 50±1°C, per ASTM D570-22 [b] Immersion in alkaline solution (pH value of 12.6-13.0) at 60±3°C for 90 days, per ASTM D7705-12.		

4.2.3 Manufacturer record and delivery documentation

The manufacturer shall establish and maintain the records required and shall identify the GFRP reinforcing bars and their delivery documentation accordingly.

For each delivery, the manufacturer shall supply the following information:

a) Manufacturer's name and place for production;
b) Description of the GFRP reinforcing bar;
c) Quantity of each type of bar;
d) Nominal diameter of GFRP reinforcing bar;
e) Cut length of the GFRP reinforcing bar;
f) Test certificates verifying compliance with all requirements in Table 7;
g) For bent bars, the drawing of the shape of the bend, the diameter of the bend, and the length of the legs.
h) Scope of the current ISO 9001 certificate.

The manufacturer shall hold a valid ISO 9001 certificate which is issued by a certification body accredited by Hong Kong Accreditation Service (HKAS) or its Multilateral Recognition Arrangements (MLA) partners.

4.3 Purchasers Testing

4.3.1 Sampling

A minimum of five random specimens shall be obtained from each batch of materials delivered to site to determine each of the property limits.

4.3.2 Property limits and test methods

Each individual specimen from a production lot shall satisfy the property limits as given in **Table 8 of this document**.

Table 8 – Property limits and test methods for purchasers testing

Property	Compliance Criteria	Test Method
Fibre content	≥70 % (by volume) or ≥55% (by mass)	ASTM D2584-18/ ASTM D3171-22
Glass transition temperature	≥100 °C [a]	ASTM E1356-08/ ASTM E1640-18
Degree of cure	≥ 95 %	ASTM E2160-04
Measured cross-sectional area	Within the given ranges in **Table 3 of ASTM D7957/D7957M-22**	ASTM D7205/D7205M-06
Ultimate tensile strength	≥ values in **Table 6 of this document**	
Tensile modulus of elasticity	≥ 44.8 GPa	
Ultimate tensile strain	≥ 1.1 %	
Moisture absorption[b]	<0.25% (24-hour immersion)	ASTM D570-22

[a] Midpoint temperature (T_m), per ASTM E1356-08
[b] Immersion in water at 50±1°C, per ASTM D570-22

4.3.3 Test reports

The test results shall be reported in an endorsed test report bearing a HKAS symbol or its Mutual Recognition Arrangement (MRA) partner.

4.3.4 Non-compliance

If any test specimen fails to conform to any requirement specified in Table 6, the batch shall be deemed non-compliant with this Specification.

4.4 Handling, Storage and Installation of GFRP Reinforcing Bar

4.4.1 Handling

GFRP reinforcement shall retain its manufactured condition during handling. Field bending is prohibited, and bars shall be protected from contact with soil, hydrocarbons, or foreign substances. Workers shall wear appropriate personal protective equipment such as gloves during handling of GFRP reinforcing bars.

Use non-abrasive lifting equipment supporting bundles at minimum three points. Avoid impact, dragging, or surface abrasion of bars.
High-temperature operations (e.g., welding, flame-cutting, etc.) must not be carried out near the exposed GFRP reinforcing bars

4.4.2 Storage

When storing GFRP reinforcing bars in the outdoor environment, the bars shall be covered with opaque plastic sheets to protect them from direct sunlight and other environmental factors. The FRP reinforcing bars shall not be exposed to temperatures above 48°C during storage.

4.4.3 Installation

Use wire reinforcement supports that are either galvanized, coated with dielectric material, or made of dielectric material (such as GFRP) when supporting GFRP reinforcing bars from wood, metal, or other rigid formwork.

Use precast concrete reinforcement supports to keep GFRP reinforcement off the ground. Reinforcement used to support GFRP reinforcement shall be either epoxy-coated bars or made of GFRP.

Do not stand, step, walk, or place equipment directly on the bars. To prevent flotation of bars during concrete placement, provide tie-downs. Bar supports and tie-downs should be made of plastic or other non-corroding materials.

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Annex A

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Worked examples of FRP-strengthened RC members

1. Example of FRP flexural strengthening

A continuous slab in a precast beam-slab of a marine concrete pier is taken as a design example for FRP flexural strengthening. The slab has been taking service in the aerated zone (XS1), and its schematic diagram is shown in **Figure 1.1.** Uniformly distributed dead load and live load are assumed to be applied on the slab, where dead load is increased from 20kN/m to 40kN/m due to heavy marine equipment to be placed on the existing pier. According to the spans of the continuous slab, under the action of uniformly distributed loads, there is no positive bending moment at midspan of the central span. The load-carrying capacity of the slab is controlled by negative bending moments of the slab on both sides of the supports (i.e., beams). Due to the symmetry of the structure, the design of flexural strengthening can be based a cantilever span of the slab, and 1 m-wide section is taken for the strength evaluations (**Figure 1.2**). The dimensions and reinforcement detailing of the strengthened slab are displayed in Table 1.1, the relevant material properties in Table 1.2, the assumed load conditions in Table 1.3, and the design calculations in Table 1.4. In the calculation of the flexural strength of the slab, the contribution of the compression steel reinforcement is not considered.

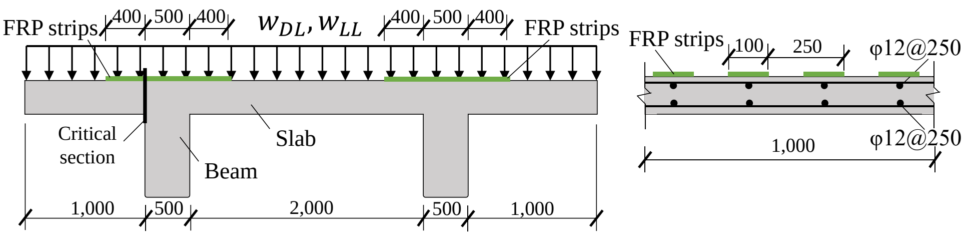

Figure 1.1 — Schematic diagram of the FRP flexural strengthening

Table 1.1 — Values of parameters

Parameter	Value
l (Span of the slab)	1,000 mm
b (Width for calculation of the slab)	1,000 mm
h (Height of the slab section)	250 mm
Concrete cover thickness	60 mm
d (Effective depth of the slab)	172 mm
A_{st} (Cross-sectional area of steel tension reinforcement)	452.4 mm^2
L_p (Length of bonded FRP strip)	400 mm
b_f (Width of FRP strip)	400 mm (100 mm-wide strips with centre-to centre spacing of 250 mm)
t_f (Thickness of FRP strip)	0.5 mm

Table 1.2 — Material properties

Material properties	Value
Concrete	C35/45
f_{ck} (Characteristic concrete cylinder compressive strength)	35 MPa
f_{cm} (Mean concrete cylinder compressive strength)	43 MPa
ε_{cu} (Ultimate compressive strain in the concrete)	0.0035
E_{cm} (Secant modulus of elasticity of concrete)	33,282.3 MPa
f_{ctm} (Mean axial tensile strength of concrete)	3.2 MPa
Steel rebars	Grade 460
f_{yk} (Characteristic value of yield strength of reinforcement)	460 MPa
E_s (Design value of modulus of elasticity of steel reinforcement)	200,000 MPa
FRP	
f_{fuk} (Characteristic short-term tensile strength of FRP)	1,000 MPa
ε_{fuk} (Characteristic ultimate strain of adhesively bonded FRP)	0.02
E_f (Mean modulus of elasticity in longitudinal direction of FRP)	50,000 MPa

Table 1.3 — Load conditions

Loading/moment	Existing loads	Design loads
w_{DL} (Dead load)	20 kN/m	40 kN/m
w_{LL} (Live load)	15 kN/m	15 kN/m
Dead-load moment M_{DL}	10.0 kN·m	—
Design load effects ($w_d = 1.35w_{DL}+1.5w_{LL}$)		
Moment $M_{Ed} = w_d l_0^2/2$	—	38.25 kN·m
Shear $V_{Ed} = w_d(l_0-d)$	—	63.34 kN
Characteristic load effects ($w_c = 1.0w_{DL}+1.0w_{LL}$)		
Moment $M_{Ec} = w_c l_0^2/2$	—	27.50 kN·m
Shear $V_{Ec} = w_c(l_0-d)$	—	45.54 kN
Load effects for the verification of strengthening limits ($w_{SL} = 1.1w_{DL}+0.75w_{LL}$)		
Moment $M_{SL} = w_{SL} l_0^2/2$	—	27.63 kN·m
Shear $V_{SL} = w_{SL}(l_0-d)$	—	45.75 kN

Table 1.4 — Steps for design checking of FRP flexural strengthening

Steps	Calculations

Step 1- Calculate the design material properties Design strength of existing reinforcing steel [**Formula (5.11) of EN 1992**]: $$f_{\mathrm{yd}}=\frac{f_{\mathrm{yk}}}{\gamma_{\mathrm{s}}}$$ Design compressive strength of concrete [**Formula (5.3) of EN 1992**]: $$f_{\mathrm{cd}}=\eta_{\mathrm{cc}}k_{\mathrm{tc}}\frac{f_{\mathrm{ck}}}{\gamma_{\mathrm{c}}}$$ For the GFRP strips used for strengthening marine concrete structures servicing in aerated zone (XS1), the environmental reduction factor η_{f} is taken to be 0.65, η_{BA} is taken to be 0.92, the partial factor γ_{f} is taken to be 1.30, and the design long-term tensile strength of FRP [**Formula (3)**]: $$f_{\mathrm{fud}}=\frac{\eta_{\mathrm{f}}\cdot f_{\mathrm{fuk}}}{\gamma_{\mathrm{f}}}$$ Cross-sectional area of FRP strips [**Clause 1.4.2(2)**]: $$A_{\mathrm{f}}=b_{\mathrm{f}}t_{\mathrm{f}}$$	$$f_{\mathrm{yd}}=\frac{460\ \mathrm{MPa}}{1.15}=400\ \mathrm{MPa}$$ According to **Clause 5.1.6(1) of EN 1992**, $\eta_{\mathrm{cc}}=1.0$, $k_{\mathrm{tc}}=1.0$; then, $$f_{\mathrm{cd}}=1.0\times1.0\times\frac{35\ \mathrm{MPa}}{1.5}=23.3\ \mathrm{MPa}$$ $$f_{\mathrm{fud}}=\frac{0.65\times1{,}000\ \mathrm{MPa}}{1.30}=500\ \mathrm{MPa}$$ $$A_{\mathrm{f}}=(400\ \mathrm{mm})(0.5\ \mathrm{mm})=200\ \mathrm{mm}^2$$
Step 2 - Check the strength of the un-strengthened slab ***<u>Moment capacity</u>*** Stress distribution within the compression zone (rectangular stress block) (**Clause 8.1.2 of EN 1992**) $$\alpha_1=1.0,\beta_1=0.8$$ Maximum depth of the compression zone assuming a rectangular stress block [**Clause 1.7.1.3.2(5)**]: $$\xi_{\mathrm{b}}d=\frac{\beta_1}{1+f_{\mathrm{yd}}/(E_{\mathrm{s}}\varepsilon_{\mathrm{cu}})}d$$ Equilibrium function (singly-reinforced) $$\alpha_1 f_{\mathrm{cd}}bx_{\mathrm{sb}}-f_{\mathrm{yd}}A_{\mathrm{st}}=0$$ $$M_{\mathrm{u}}=f_{\mathrm{yd}}A_{\mathrm{st}}(d-x_{\mathrm{sb}}/2)$$ ***<u>Shear capacity</u>***	**For the existing slab without strengthening,** ***<u>Moment capacity</u>*** $$\xi_{\mathrm{b}}=\frac{0.8}{1+400/(200000\times0.0035)}=0.509$$ $$x_{\mathrm{sb}}=\frac{400\times452.4}{1.0\times23.3\times1000}=7.77\ \mathrm{mm}<\xi_{\mathrm{b}}d$$ $$=0.509\times172=87.6\ \mathrm{mm}$$ $$M_{\mathrm{u,0}}=400\times452.4\times\left(172-\frac{7.77}{2}\right)=30.4\ \mathrm{kN\cdot m}$$ $$>M_{\mathrm{SL}}=27.63\ \mathrm{kN\cdot m}$$ $$<M_{\mathrm{Ed}}=38.25\ \mathrm{kN\cdot m}$$ <u>Strengthening is needed, and the strengthening limit is satisfied.</u> ***<u>Shear capacity</u>*** For the verification of strengthening limit: $$\tau_{\mathrm{SL}}=\frac{45.75\times10^3\ \mathrm{N}}{1000\ \mathrm{mm}\times0.9\times172\ \mathrm{mm}}=0.296\ \mathrm{MPa}$$

Average shear stress over the cross-section [**Formula (8.18) of EN 1992**] $$\tau_{\mathrm{Ed}}=\frac{V_{\mathrm{Ed}}}{b_{\mathrm{w}}\cdot z}=\frac{V_{\mathrm{Ed}}}{b_{\mathrm{w}}\cdot(0.9d)}$$ The minimum shear stress resistance [**Formula (8.20) of EN 1992**]: $$\tau_{\mathrm{Rdc,min}}=\frac{11}{\gamma_{\mathrm{V}}}\cdot\sqrt{\frac{f_{\mathrm{ck}}}{f_{\mathrm{yd}}}\cdot\frac{d_{\mathrm{dg}}}{d}}$$	Under design loads: $$\tau_{\mathrm{Ed}}=\frac{63.34\times10^{3}\ \mathrm{N}}{1000\ \mathrm{mm}\times0.9\times172\ \mathrm{mm}}=0.409\ \mathrm{MPa}$$ The size parameter $d_{\mathrm{dg}}=40$ mm is assumed; then, $$\tau_{\mathrm{Rdc,min}}=\frac{11}{1.4}\cdot\sqrt{\frac{35}{400}\cdot\frac{40}{172}}=1.121\ \mathrm{MPa}>\tau_{\mathrm{Ed}}$$ <u>No need for shear strengthening.</u>
Step 3 - Determine the existing state of strain prior to strengthening Assume $\alpha_{\mathrm{e}}=E_{\mathrm{s}}/E_{\mathrm{c,eff}}$ to be 15 referring to **Clause 9.1(5) of EN 1992**, where $E_{\mathrm{c,eff}}$ is the effective elastic modulus of concrete considering long-term creep. The existing state of strain is calculated assuming the beam is cracked and the only loads acting on the beam at the time of the FRP installation are dead loads. The existing strain at the tensile face is evaluated based on an analysis of cracked section in the following. Depth of neutral axis for the cracked section: $$x_{\mathrm{c}}=\frac{-A_{\mathrm{st}}\alpha_{\mathrm{e}}+\sqrt{(A_{\mathrm{st}}\alpha_{\mathrm{e}})^{2}-2b(-A_{\mathrm{st}}\alpha_{\mathrm{e}}d)}}{b}$$ Concrete stress at the extreme compression face for the cracked section: $$\sigma_{\mathrm{c}}=\frac{M_{\mathrm{DL}}}{bx(d-x/3)/2}$$ **Existing strain at the tensile face:** $$\varepsilon_{\mathrm{i}}=\frac{\sigma_{\mathrm{c}}}{E_{\mathrm{c,eff}}}\frac{h-x_{\mathrm{c}}}{x_{\mathrm{c}}}$$	$$E_{\mathrm{c,eff}}=\frac{200{,}000}{15}=13{,}333.3\ \mathrm{MPa}$$ $$x_{\mathrm{c}}=\frac{-452.4\times15+\sqrt{(452.4\times15)^{2}-2\times1000\times(-452.4\times15\times172)}}{1000}$$ $$=42.0\ \mathrm{mm}$$ $$\sigma_{\mathrm{c}}=\frac{10.0\times10^{6}}{1000\times42.0\times(172-42.0/3)/2}=3.01\ \mathrm{MPa}$$ $$\varepsilon_{\mathrm{i}}=\frac{3.01}{13{,}333.3}\times\frac{250-42.0}{42.0}=0.00112$$
Step 4 - Determine the bond strength of the FRP system Intermediate crack (IC) debonding	$$L_{\mathrm{ee}}=0.228\times\sqrt{50000\times0.5}=36.1\ \mathrm{mm}$$ $$\alpha=3.41\times\frac{36.1}{400}=0.307$$

strength (**Clause 1.10.1.2**) $f_{\mathrm{dbic}} = 0.114(4.41-\alpha)\tau_{\max}\sqrt{\frac{E_{\mathrm{f}}}{t_{\mathrm{f}}}} + f_{\mathrm{anch}}$ $\tau_{\max} = 1.5\beta_{\mathrm{w}} f_{\mathrm{ctm}}$ $\beta_{\mathrm{w}} = \sqrt{\frac{2-b_{\mathrm{f}}/b_{\mathrm{t}}}{1+b_{\mathrm{f}}/b_{\mathrm{t}}}}$ $\alpha = 3.41 L_{\mathrm{ee}}/L_{\mathrm{d}}$ where L_d is the distance from the maximum moment section to the end of EBR, and L_d = 400 mm $L_{\mathrm{ee}} = 0.228\sqrt{E_{\mathrm{f}} t_{\mathrm{f}}}$ (mm) $f_{\mathrm{bfRd}} = \frac{\eta_{\mathrm{BA}} f_{\mathrm{dbic}}}{\gamma_{\mathrm{BA}}}$	$\beta_{\mathrm{w}} = \sqrt{\frac{2-400/1000}{1+400/1000}} = 1.07$ $\tau_{\max} = 1.5 \times 1.07 \times 3.2 = 5.13\ \mathrm{MPa}$ No additional anchorage of mechanical fasteners is designed for EBR; then, $f_{\mathrm{anch}} = 0$ $\Rightarrow f_{\mathrm{dbic}} = 0.114 \times (4.41-0.307) \times 5.13 \times \sqrt{\frac{50000}{0.5}} + 0$ $= 758.9\ \mathrm{MPa}$ Intermediate crack (IC) debonding strength: $f_{\mathrm{bfRd}} = \frac{0.92 \times 758.9}{1.5} = 465.5\ \mathrm{MPa}$
Step 5 – Determine the failure mode and effective stress of FRP at ULS Firstly, assuming the failure mode as concrete crushing, $\sigma_{\mathrm{fe,m1}}$ is calculated by **Clause 1.7.1.3.2(3)** $\begin{cases} f_{\mathrm{cd}} b_{\mathrm{w}} x_{\mathrm{sb}} = f_{\mathrm{yd}} A_{\mathrm{st}} - \sigma_{\mathrm{fe,m1}} A_{\mathrm{f}} \\ x_{\mathrm{sb}} = \frac{\beta_1 \varepsilon_{\mathrm{cu}}}{\varepsilon_{\mathrm{cu}} + \frac{\sigma_{\mathrm{fe,m1}}}{E_{\mathrm{f}}} + \varepsilon_{\mathrm{i}}} h \end{cases}$ Then, determine the failure mode and the maximum tensile stress in the FRP reinforcement [**Clause 1.7.1.3.2(2)**] $\sigma_{\mathrm{f,md}} = \min\{f_{\mathrm{fud}}, f_{\mathrm{bfRd}}, \sigma_{\mathrm{fe,m1}}\}$	<u>Solve the system of equations on the left column to calculate x_{sb}:</u> $A = \alpha_1 f_{\mathrm{cd}} b = 1 \times 23.3 \times 1000 = 23333.3$ $B = E_{\mathrm{f}} A_{\mathrm{f}}(\varepsilon_{\mathrm{cu}} + \varepsilon_{\mathrm{i}}) - f_{\mathrm{y}} A_{\mathrm{s}} = 50000 \times 200 \times (0.0035 + 0.0011) - 400 \times 452.4 = -134763$ $C = -E_{\mathrm{f}} A_{\mathrm{f}} \varepsilon_{\mathrm{cu}} \beta_1 h = -50000 \times 200 \times 0.0035 \times 0.8 \times 250 = -7.0 \times 10^6$ $x_{\mathrm{sb}} = \frac{-B+\sqrt{B^2-4AC}}{2A} = \frac{134763 + \sqrt{(-134763)^2 + 4 \times 23333.3 \times 7.0 \times 10^6}}{2 \times 23333.3} = 20.45\ \mathrm{mm}$ $\sigma_{\mathrm{fe,m1}} = E_{\mathrm{f}}\left(\frac{\beta_1 \varepsilon_{\mathrm{cu}}}{x_{\mathrm{sb}}} h - \varepsilon_{\mathrm{cu}} - \varepsilon_{\mathrm{i}}\right) = 50000 \times \left(\frac{0.8 \times 0.0035}{20.4} \times 250 - 0.0035 - 0.0011\right)$ $= 1480.8\ \mathrm{MPa}$ $\sigma_{\mathrm{f,md}} = \min\{500.0, 465.5, 1480.8\} = 465.5\ \mathrm{MPa}$ The cause of flexural failure of the FRP-strengthened slab is expected to be <u>*FRP debonding*</u>.
Step 6 - Calculate and check the ultimate strength of FRP-strengthened flexural member Factors of equivalent rectangular stress block (**Clause 1.7.1.3.1**) $\begin{cases} \alpha_1 = 0.5 + 0.5\sigma_{\mathrm{f,md}}/\sigma_{\mathrm{fe,m1}} \\ \beta_1 = 0.8 \end{cases}$ Check of the flexural strength of FRP-strengthened slab [**Clause**	$\begin{cases} \alpha_1 = 0.5 + 0.5 \times 465.5/1480.8 = 0.657 \\ \beta_1 = 0.8 \end{cases}$ $x_{\mathrm{sb}} = \frac{400 \times 452.4 + 465.5 \times 200}{0.657 \times 23.3 \times 1000} = 17.9\ \mathrm{mm}$ $< \xi_{\mathrm{b}} d = 0.509 \times 172 = 87.6\ \mathrm{mm}$ The steel tension bars can reach yielding at the ULS, **satisfying Clause 1.7.1.1(5)**.

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1.7.1.3.2(1)]

$$\alpha_1 f_{\mathrm{cd}} b_{\mathrm{w}} x_{\mathrm{sb}} = f_{\mathrm{yd}} A_{\mathrm{st}} + \sigma_{\mathrm{f,md}} A_{\mathrm{f}}$$

$$M_{\mathrm{Rd}} = \alpha_1 f_{\mathrm{cd}} b_{\mathrm{w}} x_{\mathrm{sb}} \left(d - \frac{x_{\mathrm{sb}}}{2}\right) + \sigma_{\mathrm{f,md}} A_{\mathrm{f}} (d_{\mathrm{f}} - d)$$

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$$M_{\mathrm{Rd}} = 0.657 \times 23.3 \times 1000 \times 17.9 \times \left(172 - \frac{17.9}{2}\right) + 465.5 \times 200 \times (250 - 172) = 51.9 \times 10^6\ \mathrm{N \cdot mm} = 51.9\ \mathrm{kN \cdot m} > M_{\mathrm{Ed}} = 38.25\ \mathrm{kN \cdot m}$$

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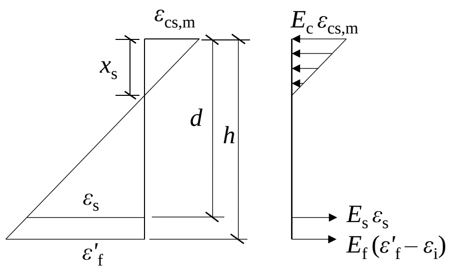
Step 7 - Check the stresses in the concrete, steel reinforcement and FRP under service loads

Analysis of cracked section

Still assuming $\alpha_{\mathrm{e}} = E_{\mathrm{s}}/E_{\mathrm{c,eff}} = 15$, and the stress and strain distributions of the cracked section are displayed in the following.



Strain distribution Stress distribution

Service stresses

Concrete (Max):

$$\sigma_{\mathrm{cs,m}} = E_{\mathrm{c,eff}} \varepsilon_{\mathrm{cs,m}} < 0.6 f_{\mathrm{ck}}$$

Steel reinforcement:

$$\sigma_{\mathrm{s}} = \frac{E_{\mathrm{s}} \varepsilon_{\mathrm{cs,m}}}{x_{\mathrm{s}}} (d - x_{\mathrm{s}}) < 0.8 f_{\mathrm{yk}}$$

FRP strip:

$$\sigma_{\mathrm{f}} = E_{\mathrm{f}} \left(\frac{h - x_{\mathrm{s}}}{x_{\mathrm{s}}} \varepsilon_{\mathrm{cs,m}} - \varepsilon_{\mathrm{i}}\right) < 0.2 \eta_{\mathrm{f}} f_{\mathrm{fuk}}$$

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<u>Results of cracked section analysis under characteristic load</u>

By solving the following system of equations,

$$\begin{cases} \frac{1}{2} E_{\mathrm{c,eff}} \varepsilon_{\mathrm{cs,m}} b_{\mathrm{w}} x_{\mathrm{s}} = E_{\mathrm{s}} \varepsilon_{\mathrm{s}} A_{\mathrm{st}} + E_{\mathrm{f}} (\varepsilon'_{\mathrm{f}} - \varepsilon_{\mathrm{i}}) A_{\mathrm{f}} \\ M_{\mathrm{Ec}} = \frac{1}{2} E_{\mathrm{c,eff}} \varepsilon_{\mathrm{cs,m}} b_{\mathrm{w}} x_{\mathrm{s}} \left(d - \frac{x_{\mathrm{s}}}{3}\right) + E_{\mathrm{f}} (\varepsilon'_{\mathrm{f}} - \varepsilon_{\mathrm{i}}) A_{\mathrm{f}} (h - d) \\ \frac{\varepsilon_{\mathrm{cs,m}}}{x_{\mathrm{s}}} = \frac{\varepsilon_{\mathrm{s}}}{d - x_{\mathrm{s}}} \\ \frac{\varepsilon_{\mathrm{cs,m}}}{x_{\mathrm{s}}} = \frac{\varepsilon'_{\mathrm{f}}}{h - x_{\mathrm{s}}} \end{cases}$$

Strain of concrete at the extreme compression face:

$$\varepsilon_{\mathrm{cs,m}} = 0.00057$$

Depth of neutral axis:

$$x_{\mathrm{s}} = 43.8\ \mathrm{mm}$$

<u>Service stresses</u> [**Clause 1.8.2(1)**]

Concrete (Max):

$$\sigma_{\mathrm{cs,m}} = 13333.3 \times 0.00057 = 7.62\mathrm{MPa} < 0.6 \times 35 = 21\ \mathrm{MPa}$$

Steel reinforcement:

$$\sigma_{\mathrm{s}} = \frac{200000 \times 0.00057}{43.8} \times (172 - 43.8) = 334.3\ \mathrm{MPa} < 0.8 \times 460 = 368\ \mathrm{MPa}$$

FRP strip:

$$\sigma_{\mathrm{f}} = 50000 \times \left(\frac{250 - 43.8}{43.8} \times 0.00057 - 0.00112\right) = 78.5\ \mathrm{MPa} < 0.2 \times 0.65 \times 1000 = 130\ \mathrm{MPa}$$

<u>Satisfy the requirements for SLS.</u>

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Step 8 - Check the deflection

<u>Deflection control</u>

Referring to **Table 9.3 of EN 1992** and **Formula (63)**:

$$LL/TL = w_{\mathrm{LL}}/(w_{\mathrm{DL}} + w_{\mathrm{LL}})$$

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$$\frac{LL}{TL} = \frac{15}{(40 + 15)} = 27.2\%$$

$$\omega_{\mathrm{r}} = \frac{452.4 + (50000 \times 200)/200000}{1000 \times 172} \times \frac{400}{23.3} = 0.05$$

$\Longrightarrow$ Conservatively, take the minimum limit of l/d =8, corresponding to a cantilever slab (**Table 9.3 of EN 1992**)

In this example, *l/d* = 1000/172 = 5.8<8, <u>satisfy the requirement of deflection control.</u>

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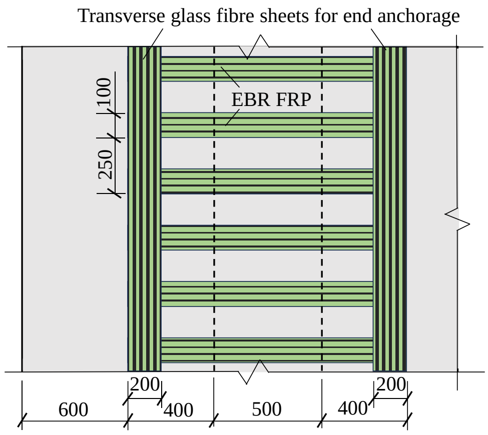
$\omega_{\mathrm{r}} = \dfrac{A_{\mathrm{s}} + (E_{\mathrm{f}} A_{\mathrm{f}})/E_{\mathrm{s}}}{b_{\mathrm{w}} d} \cdot \dfrac{f_{\mathrm{yd}}}{f_{\mathrm{cd}}}$	
Step 9 - Prevention of unintended debonding or anchorage failure Anchorage measures for preventing end debonding (**Clause 1.10.1.3**) ***<u>Strength check for shear capacity</u>*** Based on **Clause 1.10.1.4** The shear demand corresponding to the ultimate flexural strength of the strengthened slab: $V_{\mathrm{Ed,m}} = \dfrac{M_{\mathrm{Rd}}}{M_{\mathrm{Ed}}} V_{\mathrm{Ed}}$ $\tau_{\mathrm{Ed,m}} = \dfrac{V_{\mathrm{Ed,m}}}{b_{\mathrm{w}} z} = \dfrac{V_{\mathrm{Ed,m}}}{b_{\mathrm{w}}(0.9d)}$	200 mm-wide and 0.25 mm-thick transverse glass fibre sheets are designed as the end anchorage of the GFRP strips for flexural strengthening, as displayed in **Figure 1.2**.  **Figure 1.2** Arrangement of EBR for flexural strengthening ***<u>Strength check for shear capacity</u>*** $V_{\mathrm{Ed,m}} = \dfrac{51.9}{38.25} \times 63.34 = 86.0\ \mathrm{kN}$ $\tau_{\mathrm{Ed,m}} = \dfrac{86.0 \times 10^3}{1000 \times 0.9 \times 172} = 0.556\ \mathrm{MPa}$ $< \tau_{\mathrm{Rdc,min}} = 1.121\ \mathrm{MPa}$ <u>Satisfy the requirement for shear capacity.</u>
Step 10 - Check the reinforcement detailing ***<u>The centre-to-centre spacing of FRP strips</u>*** **Clause 1.11.1(1):** $s_{\mathrm{f,max}} \leq$ 3 times slab thickness $s_{\mathrm{f,max}} \leq$ 0.4 times cantilever length $s_{\mathrm{f,max}} \leq$ 400 mm The clear spacing of FRP strips [**Clause 1.11.1(2)**] ***<u>The length of EBR</u>*** **Clause 1.11.1(3):** L_{f} + 200 mm According to the parabolic distribution of moment,	$s_{\mathrm{f,max}} = 250\ \mathrm{mm} < 3 \times 250 = 750\ \mathrm{mm}$ $< 0.4 \times 1000 = 400\ \mathrm{mm}$ $< 400\ \mathrm{mm}$ <u>Satisfy the requirement.</u> $s_{\mathrm{f}} - b_{\mathrm{f}} = 250 - 100 = 150\ \mathrm{mm} < \min(250, 200)\ (\mathrm{mm})$ <u>Satisfy the requirement.</u> $L_{\mathrm{f}} = 1\ \mathrm{m} - \sqrt{\dfrac{2 \times 30.4\ \mathrm{kN \cdot m}}{76.5\ \mathrm{kN/m}}} = 108.2\ \mathrm{mm}$ $L_{\mathrm{f}} + 200\ \mathrm{mm} = 308.2\ \mathrm{mm} < 400\ \mathrm{mm}$ <u>Satisfy the requirement.</u>

$L_{\mathrm{f}} = l - \sqrt{\frac{2M_{\mathrm{u,0}}}{w_{\mathrm{d}}}}$	

2. Example of FRP shear strengthening

A simply supported beam in a precast beam-slab of a marine concrete pier is taken as a design example for FRP shear strengthening. The beam has been taking service in the splash zone (XS3) according to PWDM, and its schematic diagram is shown in **Figure 2.1**. Uniformly distributed dead load and live load are assumed to be applied on the beam, where dead load is increased from 30 kN/m to 40 kN/m and live load is increased from 30 kN/m to 45 kN/m due to new roof cover to be constructed on the existing pier. Since the beam and slab are prefabricated as a whole, the flexural strength of the beam can be evaluated according to a T-shaped section formed by the beam web and a portion of the slab on it. The dimensions and reinforcement detailing of the analysed beam are displayed in **Table 2.1**, the relevant material properties in **Table 2.2**, the assumed load conditions in **Table 2.3**, and the design calculations in **Table 2.4**. In the calculation of the flexural strength of the beam, the contribution of the compression reinforcement is not considered.

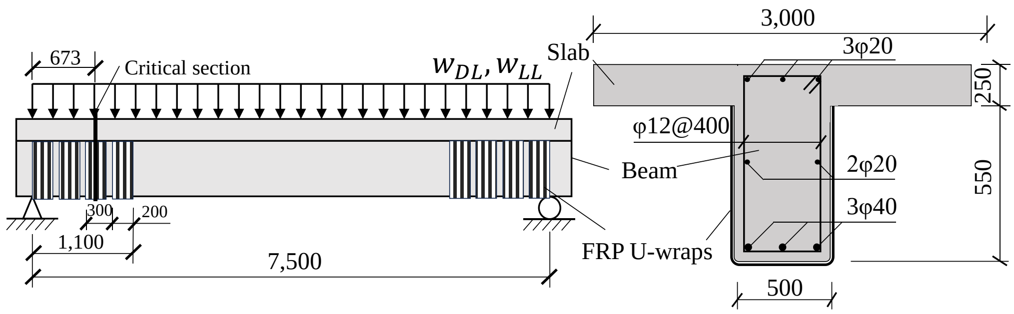


Figure 2.1 — Schematic diagram of the FRP shear strengthening

Table 2.1 — Values of parameters

Parameter	**Value**
l_0 (Effective span of the beam)	7,500 mm
b_w (Width of the beam web)	500 mm
b_f' (Flange width of T-section)	3,000 mm
h (Height of the beam)	800 mm
h_f' (Flange thickness of T-section)	250 mm
Concrete cover thickness	75 mm
d (Effective depth of the section)	693 mm
A_{st} (Cross-sectional area of steel tension reinforcement)	3770 mm^2
ϕ_{trans} (Diameter of transverse bars)	12 mm
s_v (Spacing of transverse steel bars)	400 mm
h_f (Height of a FRP strip bonded on the web)	550 mm
b_f (Width of a FRP strip)	200 mm
s_f (Spacing of FRP strips)	300 mm
t_f (Thickness of a FRP strip)	0.5 mm
α_f (Angle of inclination of fibres in the FRP)	90°

Table 2.2 — Material properties

Material properties	**Value**
Concrete	C35/45
f_{ck} (Characteristic concrete cylinder compressive strength)	35 MPa
f_{cm} (Mean concrete cylinder compressive strength)	43 MPa
ε_{cu} (Ultimate compressive strain in the concrete)	0.0035
E_{cm} (Secant modulus of elasticity of concrete)	33,282.3 MPa
f_{ctm} (Mean axial tensile strength of concrete)	3.2 MPa
Steel rebars	Grade 460
f_{yk} (Characteristic value of yield strength of reinforcement)	460 MPa
E_s (Design value of modulus of elasticity of steel reinforcement)	200,000 MPa
FRP	
f_{fuk} (Characteristic short-term tensile strength of FRP)	2,500 MPa
ε_{fuk} (Characteristic ultimate strain of FRP)	0.0156
E_f (Mean modulus of elasticity in longitudinal direction of FRP)	160,000 MPa

Table 2.3 — Load conditions

Loading/moment	**Existing loads**	**Design loads**
w_{DL} (Dead load)	30 kN/m	40 kN/m
w_{LL} (Live load)	30 kN/m	45 kN/m
Design load effects ($w_d = 1.35w_{DL}+1.5w_{LL}$)		
Moment $M_{Ed} = w_d l_0^2/8$	—	854.30 kN·m
Shear $V_{Ed} = w_d(l_0\text{-}2d)/2$	—	371.43 kN
Load effects for the verification of strengthening limits ($w_{SL} = 1.1w_{DL}+0.75w_{LL}$)		
Moment $M_{SL} = w_{SL} l_0^2/8$	—	546.68 kN·m
Shear $V_{SL} = w_{SL}(l_0\text{-}2d)/2$	—	237.68 kN

Table 2.4 — Steps for design checking of FRP shear strengthening

Steps	**Calculations**
Step 1- Calculate the design material properties Design strength of existing reinforcing steel [**Formula (5.11) of EN 1992**]: $$f_{yd} = \frac{f_{yk}}{\gamma_s}$$ Design compressive strength of concrete	$$f_{yd} = \frac{460\ \text{MPa}}{1.15} = 400\ \text{MPa}$$ According to **Clause 5.1.6(1) of EN 1992**, $\eta_{cc} = 1.0$, $k_{tc} = 1.0$; then, $$f_{cd} = 1.0 \times 1.0 \times \frac{35\ \text{MPa}}{1.5} = 23.3\ \text{MPa}$$

[**Formula (5.3) of EN 1992**]: $$f_{\text{cd}} = \eta_{\text{cc}} k_{\text{tc}} \frac{f_{\text{ck}}}{\gamma_c}$$ For the CFRP strips used for strengthening marine concrete structures servicing in aerated zone (XS1), the environmental reduction factor η_{f} is taken to be 0.83, η_{BA} is taken to be 0.85, the partial factor γ_{f} is taken to be 1.30, and the design long-term tensile strength of FRP [**Formula (3)**]: $$f_{\text{fud}} = \frac{\eta_{\text{f}} \cdot f_{\text{fuk}}}{\gamma_{\text{f}}}$$ Cross-sectional area of FRP strips [**Clause 1.4.2(2)**]: $$A_{\text{f}} = b_{\text{f}} t_{\text{f}}$$ Cross-sectional area of shear reinforcement: $$A_{\text{sw}} = n_{\text{s}} \pi \left(\frac{\phi_{\text{trans}}}{2} \right)^2$$	$$f_{\text{fud}} = \frac{0.83 \times 2{,}500 \text{ MPa}}{1.30} = 1596.2 \text{ MPa}$$ $$A_{\text{sw}} = 2 \times \pi \times \left(\frac{12}{2} \right)^2 = 226.2 \text{ mm}^2$$
Step 2 - Check the strength of the un-strengthened beam ***<u>Moment capacity</u>*** Stress distribution within the compression zone (rectangular stress block) (**Clause 8.1.2 of EN 1992**) $$\alpha_1 = 1.0, \beta_1 = 0.8$$ Maximum depth of the compression zone assuming a rectangular stress block [**Clause 1.7.1.3.2(5)**]: $$\xi_{\text{b}} d = \frac{\beta_1}{1 + f_{\text{yd}}/(E_{\text{s}} \varepsilon_{\text{cu}})} d$$ Equilibrium function (singly-reinforced) $$\alpha_1 f_{\text{cd}} b x_{\text{sb}} - f_{\text{yd}} A_{\text{st}} = 0$$ $$M_{\text{u}} = f_{\text{yd}} A_{\text{st}} (d - x_{\text{sb}}/2)$$ ***<u>Shear capacity</u>*** Average shear stress over the cross-section [**Formula (8.18) of EN 1992**] $$\tau_{\text{Ed}} = \frac{V_{\text{Ed}}}{b_{\text{w}} \cdot z} = \frac{V_{\text{Ed}}}{b_{\text{w}} \cdot (0.9d)}$$ The minimum shear stress resistance [**Formula (8.20) of EN 1992**]: $$\tau_{\text{Rdc,min}} = \frac{11}{\gamma_{\text{V}}} \cdot \sqrt{\frac{f_{\text{ck}}}{f_{\text{yd}}} \cdot \frac{d_{\text{dg}}}{d}}$$ Design shear stress resistance without shear	**For the existing beam without strengthening,** ***<u>Moment capacity</u>*** $$\xi_{\text{b}} = \frac{0.8}{1 + 400/(200000 \times 0.0035)} = 0.509$$ $$x_{\text{sb}} = \frac{400 \times 3770}{1.0 \times 23.3 \times 3000} = 21.5 \text{ mm}$$ $$< h'_{\text{f}} = 250 \text{ mm}$$ $$< \xi_{\text{b}} d = 0.509 \times 693 = 353 \text{ mm}$$ $$M_{\text{u,0}} = 400 \times 3770 \times \left(693 - \frac{21.5}{2} \right)$$ $$= 1028.8 \text{ kN} \cdot \text{m}$$ $$> M_{\text{Ed}} = 854.30 \text{ kN} \cdot \text{m}$$ <u>No need for flexural strengthening.</u> ***<u>Shear capacity</u>*** For the verification of strengthening limit: $$\tau_{\text{SL}} = \frac{237.68 \times 10^3 \text{ N}}{500 \text{ mm} \times 0.9 \times 693 \text{ mm}} = 0.762 \text{ MPa}$$ Under design loads: $$\tau_{\text{Ed}} = \frac{371.43 \times 10^3 \text{ N}}{500 \text{ mm} \times 0.9 \times 693 \text{ mm}} = 1.19 \text{ MPa}$$ $$\tau_{\text{Rdc,min}} = \frac{11}{1.4} \cdot \sqrt{\frac{35}{400} \cdot \frac{40}{693}} = 0.558 \text{ MPa}$$

reinforcement [**Formula (8.27) of EN 1992**]: $\tau_{\mathrm{Rd,c}} = \frac{0.66}{\gamma_{\mathrm{V}}}\cdot\left(100\rho_{\mathrm{l}} f_{\mathrm{ck}}\frac{d_{\mathrm{dg}}}{d}\right)^{1/3} \geq \tau_{\mathrm{Rdc,min}}$ $\rho_{\mathrm{l}} = \frac{A_{\mathrm{sl}}}{b_{\mathrm{w}} d}$ Design shear stress resistance in case of yielding of the shear reinforcement [**Clause 8.2.3(5) of EN 1992**]: $\tau_{\mathrm{Rd,sy}} = \rho_{\mathrm{w}}\cdot f_{\mathrm{ywd}}\cdot\cot\theta \leq \frac{\nu\cdot f_{\mathrm{cd}}}{2}$ $\rho_{\mathrm{w}}=\frac{A_{\mathrm{sw}}}{b_{\mathrm{w}} s}$ $\cot\theta_{\mathrm{min}} \geq \cot\theta = \sqrt{\frac{\nu\cdot f_{\mathrm{cd}}}{\rho_{\mathrm{w}}\cdot f_{\mathrm{ywd}}}-1}\geq 1$ According to **Clause 8.2.3(4) of EN 1992**, $\cot\theta_{\mathrm{min}}=2.5$ According to **Clause 8.2.3(6) of EN 1992**, $\nu=0.5$	$\rho_{\mathrm{l}}=\frac{3770}{500\times693}=0.0109$ $\tau_{\mathrm{Rd,c}}=\frac{0.66}{1.4}\cdot\left(100\times0.0109\times35\times\frac{40}{693}\right)^{\frac{1}{3}}$ $=0.613\ \mathrm{MPa}$ $\rho_{\mathrm{w}} = \frac{226.2}{500\times400}=0.00113$ $\cot\theta=\sqrt{\frac{0.5\times23.3}{0.00113\times400}-1}=4.98>2.5$ $\cot\theta=2.5$ is used for calculation. $\tau_{\mathrm{Rd,sy}}=0.00113\times400\times2.5=1.13\ \mathrm{MPa}$ $< \frac{0.5\times23.3}{2}=5.83\ \mathrm{MPa}$ $\tau_{\mathrm{Rd}}=\tau_{\mathrm{Rd,sy}}=1.13\ \mathrm{MPa}$ $> \tau_{\mathrm{SL}}=0.762\ \mathrm{MPa}$ $< \tau_{\mathrm{Ed}}=1.19\ \mathrm{MPa}$ Satisfy the strengthening limit, and need shear strengthening.
Step 3 - Determine the bond strength of FRP shear strengthening system Based on **Clause 1.10.2** ($\gamma_{\mathrm{BA}}=1.50$) $f_{\mathrm{bfRd}}=\frac{\eta_{\mathrm{BA}}\alpha\beta_{\mathrm{w}}\beta_{\mathrm{L}}}{\gamma_{\mathrm{BA}}}\sqrt{\frac{E_{\mathrm{f}}}{t_{\mathrm{f}}}\sqrt{f_{\mathrm{ck}}}}$ $\beta_{\mathrm{w}}=\sqrt{\frac{2-\frac{b_{\mathrm{f}}}{s_{\mathrm{f}}\sin\alpha_{\mathrm{f}}}}{1+\frac{b_{\mathrm{f}}}{s_{\mathrm{f}}\sin\alpha_{\mathrm{f}}}}}\geq\frac{\sqrt{2}}{2}$ $\beta_{\mathrm{L}}=\begin{cases}1 & \text{if }\lambda\geq1\\ \sin\left(\frac{\pi\lambda}{2}\right) & \text{if }\lambda<1\end{cases}$ $\lambda=L/L_{\mathrm{e}}$ Bond length [**Formula (58)**]: $L=\frac{h_{\mathrm{fe}}}{\sin\alpha_{\mathrm{f}}}$, U − wraps Effective height of the FRP [**Formula (49)**]: $h_{\mathrm{fe}}=h_{\mathrm{f}}-(h-0.9d)$	$\beta_{\mathrm{w}}=\sqrt{\frac{2-\frac{200}{300\times1}}{1+\frac{200}{300\times1}}}=0.894>\frac{\sqrt{2}}{2}$ $h_{\mathrm{fe}}=550-(800-0.9\times693)=373.7\ \mathrm{mm}$ $L=\frac{373.7}{\sin(90°)}=373.7\ \mathrm{mm}$ $L_{\mathrm{e}}=\sqrt{\frac{160000\times0.5}{\sqrt{35}}}=116.3\ \mathrm{mm}$ $\lambda=\frac{373.7}{116.3}=3.2>1 \Rightarrow \beta_L=1$ $f_{\mathrm{bfRd}}=\frac{0.85\times0.315\times0.894\times1}{1.5}$ $\times\sqrt{\frac{160000\ \mathrm{MPa}}{0.5\ \mathrm{mm}}\sqrt{35\ \mathrm{MPa}}}$ $=219.7\ \mathrm{MPa}$

$L_{\mathrm{e}} = \sqrt{\frac{E_{\mathrm{f}} t_{\mathrm{f}}}{\sqrt{f_{\mathrm{ck}}}}}$	
Step 4 - Determine the effective stress in FRP of FRP-strengthened member _Effective stress in FRP_ (**Clause 1.7.2.4**) If the shear failure is controlled due to the tensile rupture of FRP [**Formulae (53), (54)**], $D_{\mathrm{f1}} = 0.5\left(1+\frac{d_{\mathrm{ft}}}{h_{\mathrm{fe}}+d_{\mathrm{ft}}}\right)$ $\sigma_{\mathrm{f,max1}} = \min\left(0.8\times f_{\mathrm{fud}}, 0.8 \times \frac{0.015\cdot\eta_{\mathrm{f}}\cdot E_{\mathrm{f}}}{\gamma_{\mathrm{f}}}\right)$ $f_{\mathrm{fwd1}} = D_{\mathrm{f1}}\sigma_{\mathrm{f,max1}}$ If the shear failure is controlled due to FRP debonding [**Formulae (55), (56)**], $D_{\mathrm{f2}} = 1-\frac{\pi-2}{\pi\lambda},\ \ \lambda>1$ $\sigma_{\mathrm{f,max2}} = \min(0.8\times f_{\mathrm{fud}}, f_{\mathrm{bfRd}})$ $f_{\mathrm{fwd2}} = D_{\mathrm{f2}}\sigma_{\mathrm{f,max2}}$ $f_{\mathrm{fwd}} = \min(f_{\mathrm{fwd1}}, f_{\mathrm{fwd2}})$	$D_{\mathrm{f1}} = 0.5\times\left(1+\frac{250}{373.7+250}\right) = 0.7$ $\sigma_{\mathrm{f,max1}} = \min\left(0.8\times 1596.2, 0.8 \times \frac{0.015\times 0.83\times 160000}{1.3}\right)$ $= 1225.9\ \mathrm{MPa}$ $\Longrightarrow f_{\mathrm{fwd1}} = 0.7\times 1225.9 = 858.6\ \mathrm{MPa}$ $D_{\mathrm{f2}} = 1-\frac{\pi-2}{\pi\times 3.2} = 0.887$ $\sigma_{\mathrm{f,max2}} = \min(0.8\times 1596.2, 219.7)$ $= 219.7\ \mathrm{MPa}$ $\Longrightarrow f_{\mathrm{fwd2}} = 0.887\times 219.7 = 194.8\ \mathrm{MPa}$ **Effective stress in FRP:** $f_{\mathrm{fwd}} = \min(858.6, 194.8) = 194.8\ \mathrm{MPa}$
Step 5 - Determine the shear capacity of FRP-strengthened member According to **Clause 1.7.2.3**, $\tau_{\mathrm{Rd,FRP}} = \tau_{\mathrm{Rd}} + \frac{V_{\mathrm{Rd,f}}}{(b_{\mathrm{w}}\cdot z)} \leq 0.5\cdot\nu\cdot f_{\mathrm{cd}}$ $V_{\mathrm{Rd,f}} = 2\psi_{\mathrm{f}} f_{\mathrm{fwd}} t_{\mathrm{f}} b_{\mathrm{f}} \frac{h_{\mathrm{fe}}}{s_{\mathrm{f}}}(\cot\theta+\cot\alpha_{\mathrm{f}}) \cdot \sin\alpha_{\mathrm{f}}$ **Clause 1.7.2.5(1)**: For U-wraps, $\psi_{\mathrm{f}} = 1.0$ **Clause 1.7.2.3(5)**: $\cot\theta = 1$	$V_{\mathrm{Rd,f}} = 2\times 1\times 194.8\times 0.5\times 200\times\frac{373.7}{300} \times(1+0)\cdot 1 = 48539.4\ \mathrm{N}$ $\tau_{\mathrm{Rd,FRP}} = 1.13+\frac{48539.4}{500\times(0.9\times 693)}$ $= 1.29\ \mathrm{MPa}$ $< 0.5\cdot 0.5\cdot 23.3 = 5.83\ \mathrm{MPa}$ $> \tau_{\mathrm{Ed}} = 1.19\ \mathrm{MPa}$ <u>Satisfy the shear demand.</u>
Step 6 – Check the reinforcement detailing According to **Clause 1.11.2,** $\left(s_{\mathrm{f}}-\frac{b_{\mathrm{f}}}{\sin\alpha_{\mathrm{f}}}\right) \leq \min\left(h_{\mathrm{fe}}\frac{1+\cot\alpha_{\mathrm{f}}}{2}, 300\ \mathrm{mm}\right)$ Length in the longitudinal direction at one end of RC member that requires shear strengthening:	**Formula (74)**: $\left(300-\frac{200}{\sin(90°)}\right) = 100\ \mathrm{mm}$ $< \min\left(373.7\times\frac{1+0}{2}, 300\ \mathrm{mm}\right) = 186.9\ \mathrm{mm}$ <u>Satisfy the requirement for reinforcement detailing.</u>

$$L_{\mathrm{str}} = \frac{l_0}{2} - \frac{\tau_{\mathrm{Rd}}}{\tau_{\mathrm{Ed}}}\left(\frac{l_0}{2} - d\right)$$	$$L_{\mathrm{str}} = \frac{7500}{2} - \frac{1.13}{1.19}\left(\frac{7500}{2} - 693\right) = 847.2\ \mathrm{mm} < 1100\ \mathrm{mm}$$ The arrangement of shear strengthening shown in **Figure 2.1** satisfies the requirement.

3. Example of FRP compressive strengthening

A concrete column servicing as a pile of a marine structure is taken as a design example for FRP compressive strengthening. The column has been taking service in the tidal zone (XS3), and its schematic diagram is shown in **Figure 3.1.** Axial and lateral loads are assumed to be applied on the column (**Table 3.3**), where dead axial load is increased from 1,200 kN to 1,500 kN due to a new large-area roof with solar panels. The FRP jacket composed of 6 plies of continuous carbon fibre sheets along is set within a height of 200 mm at the top of the column for compressive strengthening. The fibres are oriented in the hoop direction (i.e., perpendicular to the longitudinal axis) of the strengthened member. The dimensions and reinforcement detailing of the analysed column are displayed in **Table 3.1**, the relevant material properties in **Table 3.2**, the assumed load conditions in **Table 3.3**, and the design calculations in **Table 3.4**.

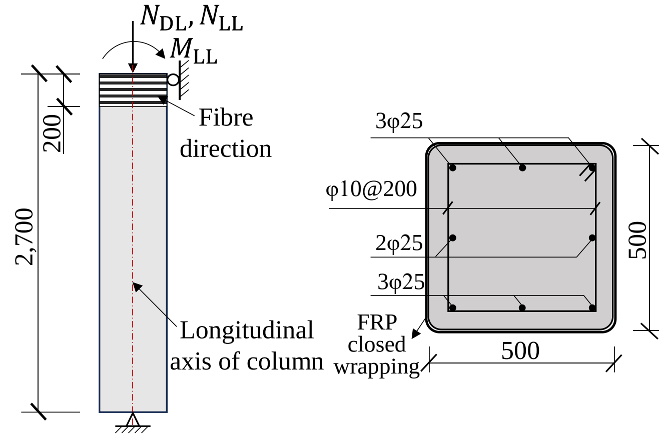


Figure 3.1 — Schematic diagram of the FRP compressive strengthening

Table 3.1 — Values of parameters

Parameter	Value
l_0 (Effective length of the column)	2,700 mm
h (Height of the section)	500 mm
b (Width of the section)	500 mm
d (Effective depth of the section)	402.5 mm
r_c (Radius of the rounded corner)	50 mm
Concrete cover thickness	75 mm
Number of longitudinal bars	8
ϕ (Diameter of longitudinal bars)	25 mm
ϕ_{trans} (Diameter of transverse bars)	10 mm
s_v (Spacing of transverse steel bars)	200 mm
Number of layers of EBR fibre sheets	6
A_f/b_f or t_{f1} (Thickness of a single layer of fibre sheet)	0.167 mm
a_{s1} (Distance from the centre of top rebar layer to the top surface of member)	97.5 mm
a_{s1} (Distance from the centre of medium rebar layer to the top surface of member)	250.0 mm
a_{s1} (Distance from the centre of bottom rebar layer to the top surface of member)	402.5 mm

Table 3.2 — Material properties

Material properties	**Value**
Concrete	C35/45
f_{ck} (Characteristic concrete cylinder compressive strength)	35 MPa
f_{cm} (Mean concrete cylinder compressive strength)	43 MPa
ε_{cu} (Ultimate compressive strain in of plain concrete)	0.0035
Steel rebars	Grade 460
f_{yk} (Characteristic value of yield strength of reinforcement)	460 MPa
E_s (Design value of modulus of elasticity of steel reinforcement)	200,000 MPa
FRP	
f_{fuk} (Characteristic short-term tensile strength of FRP)	2,900 MPa
ε_{fuk} (Characteristic ultimate strain of FRP)	0.0074
E_f (Mean modulus of elasticity in hoop direction of FRP)	390,000 MPa

Table 3.3 — Load conditions

Loading/moment	**Existing loads**		**Design loads**	
	Dead load	Live load	Dead load	Live load
Unfactored axial load N	1,200 kN	600 kN	1,500 kN	600 kN
Unfactored moment M_2	0	320 kN·m	0	320 kN·m
Unfactored moment M_1	0	0	0	0
Unfactored shear force V	0	118.5 kN	0	118.5 kN
Design load effects ($1.35L_{DL}$+$1.5L_{LL}$)				
Axial load N_{Ed}	—		2925.0 kN	
Moment M_{Ed2}	—		480.0 kN·m	
Moment M_{Ed1}	—		0	
Shear V_{Ed}	—		177.8 kN	
Load effects for the verification of strengthening limits ($1.1L_{DL}$+$0.75L_{LL}$)				
Axial load N_{SL}	—		2100.0 kN·m	
Moment M_{SL2}	—		240.0 kN·m	
Moment M_{SL1}	—		0	
Shear V_{SL}	—		88.9 kN	

Characteristic load effects (1.0L_{DL}+1.0L_{LL})		
Axial load N_{Ec}	—	2100.0 kN·m
Moment M_{Ec2}	—	320.0 kN·m
Moment M_{Ec1}	—	0
Shear V_{Ec}	—	118.5 kN

Table 3.4 — Steps for design checking of FRP compressive strengthening

Steps	**Calculations**
Step 1- Calculate the design material properties Design strength of existing reinforcing steel [**Formula (5.11) of EN 1992**]: $$f_{\mathrm{yd}} = \frac{f_{\mathrm{yk}}}{\gamma_{\mathrm{s}}}$$ Design compressive strength of concrete [**Formula (5.3) of EN 1992**]: $$f_{\mathrm{cd}} = \eta_{\mathrm{cc}} k_{\mathrm{tc}} \frac{f_{\mathrm{ck}}}{\gamma_c}$$ Characteristic ultimate strain of the FRP jacket in the hoop direction [**Clause 1.7.1.2(5)**]: $$\varepsilon_{\mathrm{ruk}} = 0.7\varepsilon_{\mathrm{fuk}}$$ For the carbon fibre sheets used for strengthening marine concrete structures servicing in splash zone (XS3), the environmental reduction factor η_{f} is taken to be 0.83, the partial factor γ_{f} is taken to be 1.40, and the design ultimate strain of the FRP jacket in the hoop direction [**Formula (11)**]: $$\varepsilon_{\mathrm{ru}} = \frac{\eta_{\mathrm{f}} \cdot \varepsilon_{\mathrm{ruk}}}{\gamma_{\mathrm{f}}}$$ Effective thickness of fibre sheets [**Formula (2)**]: $$t_{\mathrm{f}} = n_{\mathrm{f}}^{k_{\mathrm{f}}} \cdot \frac{A_{\mathrm{f}}}{b_{\mathrm{f}}}$$ Cross-sectional area of longitudinal reinforcement:	$$f_{\mathrm{yd}} = \frac{460\ \mathrm{MPa}}{1.15} = 400\ \mathrm{MPa}$$ According to **Clause 5.1.6(1) of EN 1992**, $\eta_{\mathrm{cc}} = 1.0$, $k_{\mathrm{tc}} = 1.0$; then, $$f_{\mathrm{cd}} = 1.0 \times 1.0 \times \frac{35\ \mathrm{MPa}}{1.5} = 23.3\ \mathrm{MPa}$$ $$\varepsilon_{\mathrm{ruk}} = 0.7 \times 0.0074 = 0.00521$$ $$\varepsilon_{\mathrm{ru}} = \frac{0.83 \times 0.00521}{1.4} = 0.00309$$ $$t_{\mathrm{f}} = 6^{0.85} \cdot 0.167 = 0.766\ \mathrm{mm}$$ Top layer of longitudinal bars: $$A_{\mathrm{s1}} = 3 \times \pi \times \left(\frac{25}{2}\right)^2 = 1472.6\ \mathrm{mm}^2$$ Intermediate layer of longitudinal bars: $$A_{\mathrm{s2}} = 2 \times \pi \times \left(\frac{25}{2}\right)^2 = 981.7\ \mathrm{mm}^2$$ Bottom layer of longitudinal bars: $$A_{\mathrm{s3}} = 3 \times \pi \times \left(\frac{25}{2}\right)^2 = 1472.6\ \mathrm{mm}^2$$ Shear reinforcement: $$A_{\mathrm{sw}} = 2 \times \pi \times \left(\frac{10}{2}\right)^2 = 157.1\ \mathrm{mm}^2$$

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$$A_{si} = n_{si}\pi\left(\frac{\phi}{2}\right)^2$$

Cross-sectional area of shear reinforcement:

$$A_{sw} = n_s\pi\left(\frac{\phi_{trans}}{2}\right)^2$$

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Step 2- Calculate the properties of FRP-confined concrete

Confinement stiffness ratio β_j [**Clause 1.7.1.2(3)**]:

$$\beta_j = \frac{E_f t_f}{\eta_{cc} f_{ck} R}$$

Clause 1.7.1.2(4):

$$R = 0.5\sqrt{b^2 + h^2}$$

Clause 1.7.1.2(8):

$$f_{cd,c} = f_{cd} + 3.5\frac{E_f t_f}{R}\left(k_{s\sigma} - \frac{6.5}{\beta_j}\right)\varepsilon_{ru}$$

$$k_{s\sigma} = \left(1.25\frac{r_c}{b} + \frac{r_c}{h} + 0.33\right)\left(\frac{b}{h}\right)^{0.4}$$

Clause 1.7.1.2(6):

$$\frac{f_{cd,c}}{f_{cd}} \le 1.75$$

Clause 1.7.1.2(9):

$$\varepsilon_{cu,c} = 0.0035 + 0.6 k_{s\varepsilon}{\beta_j}^{0.8}{\varepsilon_{ru}}^{1.45}$$

$$k_{s\varepsilon} = \left(1.25\frac{r_c}{b} + \frac{r_c}{h} + 0.33\right)\left(\frac{h}{b}\right)^{0.5}$$

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$$R = 0.5 \times \sqrt{500^2 + 500^2} = 353.6 \text{ mm}$$

$$\beta_j = \frac{390000 \times 0.766}{1 \times 35 \times 353.6} = 24.1$$

$$k_{s\sigma} = \left(1.25 \times \frac{50}{500} + \frac{50}{500} + 0.33\right) \times \left(\frac{500}{500}\right)^{0.4} = 0.555$$

$$f_{cd,c} = 23.3 + 3.5 \times \frac{390000 \times 0.766}{353.6} \times \left(0.555 - \frac{6.5}{24.1}\right) \times 0.00309 = 25.94 \text{ MPa}$$

$$\frac{25.94}{23.3} = 1.11 < 1.75$$

$$k_{s\varepsilon} = \left(1.25 \times \frac{50}{500} + \frac{50}{500} + 0.33\right) \times \left(\frac{500}{500}\right)^{0.5} = 0.555$$

$$\varepsilon_{cu,c} = 0.0035 + 0.6 \times 0.555 \times 24.1^{0.8} \times 0.00309^{1.45} = 0.00447$$

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Step 3- Check the strength of the un-strengthened and FRP-strengthened RC member

<u>Additional eccentricity</u>

Based on **Clause 1.7.1.3.3(1)**

$$e_a = \max\left(20\text{mm}, \frac{h}{30}\right)$$

<u>Slender effect consideration</u>

Based on **Clause 1.7.1.3.3(2)**, the slenderness effect of FRP-strengthened columns can be omitted, when the slenderness ratio satisfies the following equation:

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$$e_a = \max\left(20\text{mm}, \frac{500 \text{ mm}}{30}\right) = 20 \text{ mm}$$

$$e_2 = \frac{480 \times 10^6}{2925 \times 10^3} = 164.1 \text{ mm} \;;\quad e_1 = 0$$

$$\frac{l_0}{h} = \frac{2700}{500} = 5.4 < \frac{15 \times \frac{164.1 - 0}{500} + 5}{\frac{25.94}{23.3} \times (1 + 30 \times 0.00521)} = 7.7$$

<u>The slenderness effect can be omitted.</u>

Calculation of design moment,

$$e_{max} = 164.1 + 20 = 184.1 \text{ mm}$$

$$N_{Ed}e_{max} = 2925.0 \times 184.1 \times 10^{-3} = 538.5 \text{ kN} \cdot \text{m}$$

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$$\frac{l_0}{h} \le \frac{15\frac{e_2 - e_1}{h} + 5}{\frac{f_{\mathrm{cd,c}}}{f_{\mathrm{cd}}}(1 + 30\varepsilon_{\mathrm{ruk}})}$$

where $e_2 = \frac{M_{\mathrm{Ed2}}}{N_{\mathrm{Ed}}}$ and $e_1 = \frac{M_{\mathrm{Ed1}}}{N_{\mathrm{Ed}}}$

<u>*Calculation of the strength of the FRP-strengthened and un-strengthened member*</u>

Based on **Clauses 1.7.1.3.3(5),(6)**

$$N_{\mathrm{Ed}} \le \alpha_1 f_{\mathrm{cd,c}} b x_{\mathrm{sb}} + \sum_{i=1}^{m} \sigma_{si} A_{si}$$

$$N_{\mathrm{Ed}} e_{\mathrm{max}} \le \alpha_1 f_{\mathrm{cd,c}} b x_{\mathrm{sb}} \left(\frac{h}{2} - \frac{x_{\mathrm{sb}}}{2}\right) + \sum_{i=1}^{m} \sigma_{si} A_{si} \left(\frac{h}{2} - a_{si}\right)$$

$$-f_{\mathrm{yd}} \le \sigma_{si} = \varepsilon_{\mathrm{cu,c}} \left(1 - \frac{\beta_1 a_{si}}{x_{\mathrm{sb}}}\right) E_{\mathrm{s}} \le f_{\mathrm{yd}}$$

$$e_{\mathrm{max}} = \max(\eta e_{\mathrm{i}}, e_2 + e_{\mathrm{a}})$$

The *N*-*M* curves for the FRP-strengthened and un-strengthened members can be generated respectively by solving the following systems of equations while increasing the value N_{Rd} from 0 to the ultimate axial strength of the analysed member.

The system of equations for FRP-strengthened member,

$$\begin{cases} N_{\mathrm{Rd}} = \alpha_1 f_{\mathrm{cd,c}} b x_{\mathrm{sb}} + \sum_{i=1}^{m} \sigma_{si} A_{si} \\ M_{\mathrm{Rd}} = \alpha_1 f_{\mathrm{cd,c}} b x_{\mathrm{sb}} \left(\frac{h}{2} - \frac{x_{\mathrm{sb}}}{2}\right) + \sum_{i=1}^{m} \sigma_{si} A_{si} \left(\frac{h}{2} - a_{si}\right) \\ -f_{\mathrm{yd}} \le \sigma_{si} = \varepsilon_{\mathrm{cu,c}} \left(1 - \frac{\beta_1 a_{si}}{x_{\mathrm{sb}}}\right) E_{\mathrm{s}} \le f_{\mathrm{yd}} \end{cases}$$

where $\alpha_1 = 1.17 - 0.2 f_{\mathrm{cd,c}}/f_{\mathrm{cd}} = 0.95$, $\beta_1 = 0.9$ [**Formula (22)**]

The system of equations for un-strengthened member,

$$\begin{cases} N_{\mathrm{Rd}} = \alpha_1 f_{\mathrm{cd}} b x_{\mathrm{sb}} + \sum_{i=1}^{m} \sigma_{si} A_{si} \\ M_{\mathrm{Rd}} = \alpha_1 f_{\mathrm{cd}} b x_{\mathrm{sb}} \left(\frac{h}{2} - \frac{x_{\mathrm{sb}}}{2}\right) + \sum_{i=1}^{m} \sigma_{si} A_{si} \left(\frac{h}{2} - a_{si}\right) \\ -f_{\mathrm{yd}} \le \sigma_{si} = \varepsilon_{\mathrm{cu}} \left(1 - \frac{\beta_1 a_{si}}{x_{\mathrm{sb}}}\right) E_{\mathrm{s}} \le f_{\mathrm{yd}} \end{cases}$$

For checking the strength of the un-strengthened RC member against strengthening limits,

$$e_{2,\mathrm{SL}} = \frac{240 \times 10^6}{2100 \times 10^3} = 114.3\ \mathrm{mm}$$

$$e_{\mathrm{max,SL}} = 114.3 + 20 = 134.3\ \mathrm{mm}$$

$$N_{\mathrm{SL}} e_{\mathrm{max,SL}} = 2100.0 \times 134.3 \times 10^{-3} = 282.0\ \mathrm{kN \cdot m}$$

<u>*Calculation of the strength of the FRP-strengthened and un-strengthened member*</u>

The results of *N*-*M* curves are shown in **Figure 3.2**.

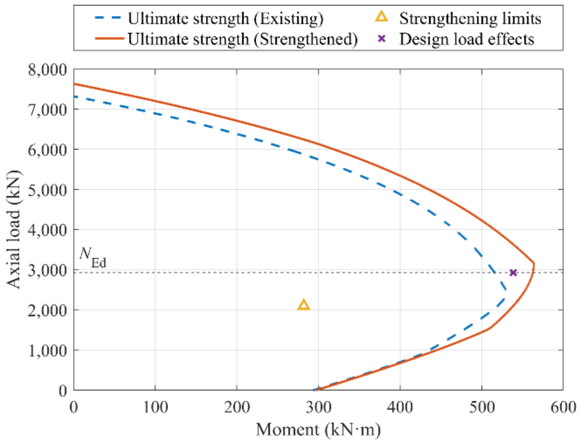

Figure 3.2 — *N*-*M* relationships of the un-strengthened and FRP-strengthened RC member

<u>The un-strengthened member satisfies the requirement of strengthening limit, and the FRP-strengthened member satisfies the strength demand of the design load effects.</u>

Due to the symmetry of structural member and loadings, the check of strength in the direction perpendicular to load eccentricity is not necessary.

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where $\alpha_1 = 1.0$, $\beta_1 = 0.8$ [**Clause 8.1.2(2) of EN 1992**]

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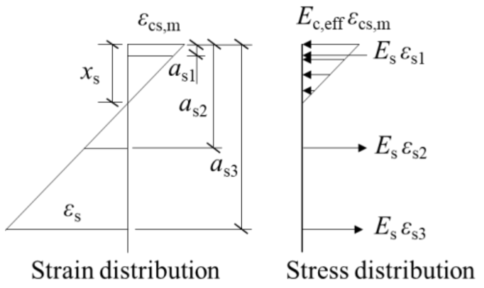
Step 4 - Check the stresses in the concrete and steel reinforcement under service loads

Analysis of cracked section

Assuming $\alpha_e = E_s/E_{c,eff} = 15$ [according to **Clause 9.1(5) of EN 1992**], the stress and strain distributions of the cracked section are displayed in the following.



Service stresses

Concrete (Max) (**Table 9.2 of EN 1992**)

$$\sigma_{cs,m} \leq 0.66 f_{ck}$$

Note: since the FRP-strengthened member is confined by FRP jacket, the coefficient is taken as 0.66 according to the note c in Table 9.2 of EN 1992.

Steel reinforcement [**Clause 1.8.2(1)**]

$$\sigma_s \leq 0.6 f_{yk}$$

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Results of cracked section analysis under characteristic load

By solving the following system of equations,

$$\begin{cases} N_{Ec} = \frac{1}{2}E_{c,eff}\varepsilon_{cs,m}b_w x_s + \sum_{i=1}^{m}\sigma_{si}A_{si} \\ M_{Ec} = \frac{1}{2}E_{c,eff}\varepsilon_{cs,m}b_w x_s\left(\frac{h}{2} - \frac{x_s}{3}\right) + \sum_{i=1}^{m}\sigma_{si}A_{si}\left(\frac{h}{2} - a_{si}\right) \\ -f_{yd} \leq \sigma_{si} = \varepsilon_{cs,m}\left(1 - \frac{a_{si}}{x_s}\right)E_s \leq f_{yd} \end{cases}$$

Strain of concrete at the extreme compression face:

$$\varepsilon_{cs,m} = 0.001636$$

Depth of neutral axis:

$$x_s = 328.6 \text{ mm}$$

Service stress of concrete:

$$\sigma_{cs,m} = \frac{200{,}000}{15} \times 0.001636 = 21.8 \text{ MPa}$$
$$< 0.66 \times 35 = 23.1 \text{ MPa}$$

Service stress of steel compression reinforcement:

$$\sigma_{s,c} = 200{,}000 \times \frac{0.001636}{328.6} \times (328.6 - 97.5)$$
$$= 230.2 \text{ MPa} < 0.6 f_{yk}$$
$$= 276 \text{ MPa}$$

Service stress of steel tension reinforcement:

$$\sigma_{s,t} = 200{,}000 \times \frac{0.001636}{328.6} \times (402.5 - 328.6)$$
$$= 73.6 \text{ MPa} < 0.6 f_{yk} = 276 \text{ MPa}$$

<u>Satisfy all the requirements.</u>

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Step 5 – Check the shear strength of the column

Average shear stress over the cross-section [**Formula (8.18) of EN 1992**]

$$\tau_{Ed} = \frac{V_{Ed}}{b_w \cdot z} = \frac{V_{Ed}}{b_w \cdot (0.9d)}$$

The minimum shear stress resistance [**Formula (8.20) of EN 1992**]:

$$\tau_{Rdc,min} = \frac{11}{\gamma_V} \cdot \sqrt{\frac{f_{ck}}{f_{yd}} \cdot \frac{d_{dg}}{d}}$$

Design shear stress resistance without shear

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Under design loads:

$$\tau_{Ed} = \frac{177.8 \times 10^3 \text{ N}}{500 \text{ mm} \times 0.9 \times 402.5 \text{ mm}} = 0.98 \text{ MPa}$$

$$\tau_{Rdc,min} = \frac{11}{1.4} \cdot \sqrt{\frac{35}{400} \cdot \frac{40}{402.5}} = 0.733 \text{ MPa}$$

$$\rho_l = \frac{1472.6}{500 \times 402.5} = 0.00732$$

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reinforcement [**Formula (8.27) of EN 1992**]: $$\tau_{\rm Rd,c}=\frac{0.66}{\gamma_{\rm V}}\cdot\left(100\rho_{\rm l}f_{\rm ck}\frac{d_{\rm dg}}{d}\right)^{1/3}\geq\tau_{\rm Rdc,min}$$ $$\rho_{\rm l}=\frac{A_{\rm sl}}{b_{\rm w}d}$$ Design shear stress resistance in case of yielding of the shear reinforcement [**Clause 8.2.3(5) of EN 1992**]: $$\tau_{\rm Rd,sy}=\rho_{\rm w}\cdot f_{\rm ywd}\cdot\cot\theta\leq\frac{\nu\cdot f_{\rm cd}}{2}$$ $$\rho_{\rm w}=\frac{A_{\rm sw}}{b_{\rm w}s}$$ $$\cot\theta_{\min}\geq\cot\theta=\sqrt{\frac{\nu\cdot f_{\rm cd}}{\rho_{\rm w}\cdot f_{\rm ywd}}-1}\geq 1$$ According to **Clause 8.2.3(4) of EN 1992**, $$\cot\theta_{\min}=2.5$$ According to **Clause 8.2.3(6) of EN 1992**, $$\nu=0.5$$	$$\tau_{\rm Rd,c}=\frac{0.66}{1.4}\cdot\left(100\times0.00732\times35\times\frac{40}{402.5}\right)^{\frac{1}{3}}$$ $$=0.644\ \text{MPa}$$ $$\cot\theta=\sqrt{\frac{0.5\times23.3}{0.00157\cdot400}-1}=4.2>\cot\theta_{\min}$$ $$=2.5$$ $$\rho_{\rm w}=\frac{157.1}{500\times200}=0.00157$$ $$\tau_{\rm Rd,sy}=0.00157\times400\times2.5=1.57\ \text{MPa}$$ $$\leq\frac{0.5\times23.3}{2}=5.83\ \text{MPa}$$ $$\tau_{\rm Rd}=\tau_{\rm Rd,sy}=1.57\ \text{MPa}>\tau_{\rm Ed}=0.98\ \text{MPa}$$ <u>No need for shear strengthening.</u>
Step 6 – Check the reinforcement detailing The minimum overlap of a lap joint in the FRP jacket is determined by, $$L_{\rm l}\leq\max\left(150\ \text{mm},\frac{f_{\rm fuk}t_{\rm f1}}{\tau_{\rm ave}}\right)$$	According to **Clause 1.11.3(1)**, $\tau_{\rm ave}=4$ MPa, $$L_{\rm l}\leq\max\left(150\ \text{mm},\frac{2900\ \text{MPa}\times0.167\ \text{mm}}{4\ \text{MPa}}\right)$$ $$=121.1\ \text{mm}$$
Step 7 – Determination of the length of the portion requiring compressive strengthening For the analysed column, the shear force varies along the column length from the maximum value at the top to zero at the bottom. Thereby, the length of the portion requiring compressive strengthening ($L_{\rm s,min}$) is evaluated by, $$L_{\rm s,min}=l_0\left(1-\frac{M_{\rm Rd\_un}}{M_{\rm Ed}}\right)$$ where $M_{\rm Rd\_un}$ represents the moment capacity of the un-strengthened column corresponding to the design axial load.	According to **Figure 3.2**, the minimum moment capacity of the un-strengthened column corresponding to the design axial load is 515.8 kN·m. $$L_{\rm s,min}=2700\times\left(1-\frac{515.8}{538.5}\right)=113.7\ \text{mm}$$ $$<200\text{mm}$$ <u>The designed strengthening length in Figure 3.1 satisfies the requirement.</u>

Annex B

Worked examples of GFRP-reinforced marine concrete structures

1. Example of GFRP-reinforced one-way slab

The continuous slab in a precast beam-slab of a marine concrete pier is taken as a design example for GFRP-reinforced one-way slab, and its schematic diagram is shown in **Figure 1.1**. Uniformly distributed dead load and live load are assumed to be applied on the slab. According to the spans of the continuous slab, under the action of uniformly distributed loads, there is no positive bending moment at midspan of the central span. The load-carrying capacity of the slab is controlled by negative bending moments of the slab on both sides of the supports (i.e., beams). Due to the symmetry of the structure, the design of GFRP-reinforced one-way slab can be based a cantilever span of the slab, and 1 m-wide section is taken for the strength evaluations (**Figure 1.1**). The dimensions and reinforcement detailing of the GFRP-reinforced slab are displayed in **Table 1.1**, the relevant material properties in **Table 1.2**, the assumed loading conditions in **Table 1.3**, and the design calculations in **Table 1.4**. In the calculation of the flexural strength of the slab, the contribution of the compression reinforcement is not considered according to Clause 8.1(3).

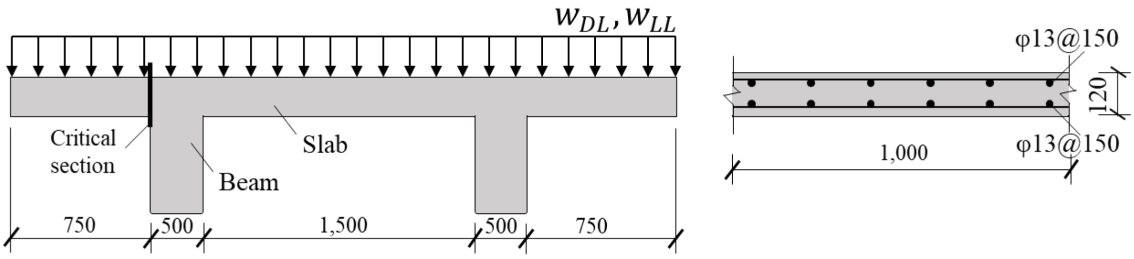

Figure 1.1 — Schematic diagram of the GFRP-reinforced one-way slab

Table 1.1 — Values of parameters

Parameter	**Value**
l (Span of the slab)	750 mm
b (Width for calculation of the slab)	1,000 mm
h (Height of the slab section)	120 mm
c (Concrete cover thickness)	30 mm
d (Effective depth of the slab)	83.5 mm
ϕ_{f} (Diameter of GFRP longitudinal bars)	13 mm
$s_{\mathrm{slab,l}}$ (Spacing of bars for concrete in tension in longitudinal direction)	150 mm
A_{fl} (Cross-sectional area of GFRP tension reinforcement in longitudinal direction)	884.9 mm^2

Table 1.2 — Material properties

Material property	**Value**
Concrete	C35/45
f_{ck} (Characteristic concrete cylinder compressive strength)	35 MPa
f_{cm} (Mean concrete cylinder compressive strength)	43 MPa
$\varepsilon_{\mathrm{cu}}$ (Ultimate compressive strain in the concrete)	0.0035
E_{cm} (Secant modulus of elasticity of concrete)	33,282.3 MPa
f_{ctm} (Mean axial tensile strength of concrete)	3.2 MPa
GFRP bars	
f_{ftk0} (Characteristic value of tensile strength of GFRP reinforcement)	1,000 MPa
$\varepsilon_{\mathrm{ftk0}}$ (Characteristic ultimate strain of GFRP reinforcement)	0.02
E_{fR} (Mean modulus of elasticity of GFRP reinforcement)	50,000 MPa

Table 1.3 — Loading conditions

Loading/moment	**Design loads**
w_{DL} (Dead Load+ Service Dead Load)	12.5 kN/m
$w_{\mathrm{LL,leading}}$ (Leading Live Load)	10 kN/m
$w_{\mathrm{LL,accompanying}}$ (Accompanying Live Load)	2.5 kN/m
ULS	
Design load effects ($w_{\mathrm{d}} = 1.35 w_{\mathrm{DL}} + 1.5 w_{\mathrm{LL,leading}} + 1.5 \times 0.5 w_{\mathrm{LL,accompanying}} = 33.75\ \mathrm{kN/m}$)	
Moment M_{Ed} (at support) $= w_d {l_0}^2/2$	9.5 kN·m (Hogging)
Shear V_{Ed} (at support) $= w_d(l_0 - d)$	22.5 kN
SLS	
Characteristic load effects ($w_{\mathrm{c}} = 1.0 w_{\mathrm{DL}} + 1.0 w_{\mathrm{LL,leading}} + 1.0 \times 0.5 w_{\mathrm{LL,accompanying}} = 23.75\ \mathrm{kN/m}$)	
Moment M_{Ec} (at support) $= w_c {l_0}^2/2$	6.7 kN·m
Quasi-permanent load effects ($w_{\mathrm{q}} = 1.0 w_{\mathrm{DL}} + 0.5 w_{\mathrm{LL,leading}} = 17.5\ \mathrm{kN/m}$)	
Moment M_{Eq} (at support) $= w_{\mathrm{q}} {l_0}^2/2$	4.9 kN·m

Table 1.4 — Steps for design checking of GFRP-reinforced one-way slab

Steps	**Calculations**
Step 1- Calculate the design material properties Characteristic long term tensile strength of GFRP reinforcement [**Formula (77)**]: $$f_{\mathrm{ftk,100a}} = C_{\mathrm{t}} \cdot C_{\mathrm{e}} \cdot f_{\mathrm{ftk0}}$$ Design tensile strength of GFRP reinforcement [**Formula (76)**]: $$f_{\mathrm{ftd}} = \frac{f_{\mathrm{ftk,100a}}}{\gamma_{\mathrm{FRP}}}$$	$$f_{\mathrm{ftd}} = 0.80 \times 0.80 \times 1{,}000 = 640.0\ \mathrm{MPa}$$ $$f_{\mathrm{ftd}} = \frac{640.0}{1.50} = 426.7\ \mathrm{MPa}$$ According to **Clause 5.1.6(1) of EN 1992**, $\eta_{\mathrm{cc}} = 1.0$, $k_{\mathrm{tc}} = 1.0$; then, $$f_{\mathrm{cd}} = 1.0 \times 1.0 \times \frac{35}{1.5} = 23.3\ \mathrm{MPa}$$

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For the GFRP reinforcement used for marine concrete pier, the factor considering temperature effects C_{t} is taken to be 0.80, the environmental reduction factor C_{e} is taken to be 0.80, the partial factor γ_{f} is taken to be 1.50.

Design compressive strength of concrete [**Formula (5.3) of EN 1992**]:

$$f_{\mathrm{cd}} = \eta_{\mathrm{cc}} k_{\mathrm{tc}} \frac{f_{\mathrm{ck}}}{\gamma_c}$$

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Step 2- Check the strength of the GFRP bar reinforced slab

Moment capacity

Stress distribution within the compression zone (rectangular stress block) (**Clause 8.1.2 of EN 1992**)

$$\alpha_1 = 1.0, \beta_1 = 0.8$$

Critical reinforcement ratio and depth of the compression zone for different failure modes, assuming a rectangular stress block [**Formulae (78) and (79)**]:

$$\xi_{\mathrm{b}} = \frac{\beta_1}{1 + f_{\mathrm{ftd}}/(E_{\mathrm{fR}}\varepsilon_{\mathrm{cu}})}$$

$$\rho_{\mathrm{fb}} = \frac{\alpha_1 f_{\mathrm{cd}}}{f_{\mathrm{ftd}}} \cdot \xi_{\mathrm{b}}$$

Equilibrium function (singly-reinforced) for flexural strength is controlled by FRP rupture [**Formulae (80) to (82)**]:

$$\sigma_{\mathrm{ftd}} = f_{\mathrm{ftd}}$$

$$x_{\mathrm{sb}} = \left[\frac{0.14}{1 + 400(\sigma_{\mathrm{ftd}}/E_{\mathrm{fR}})} + \frac{\rho_{\mathrm{lf}}\sigma_{\mathrm{ftd}}}{f_{\mathrm{cd}}}\right] d$$

$$M_{\mathrm{Rd}} = \varphi A_{\mathrm{f}} \sigma_{\mathrm{ftd}} \left(d - \frac{x_{\mathrm{sb}}}{2}\right)$$

$$\varphi = 0.85$$

Shear capacity

Average shear stress over the cross-section [**Formula (8.18) of EN 1992**]

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<u>*Moment capacity*</u>

$$\xi_{\mathrm{b}} = \frac{0.8}{1 + 426.7/(50{,}000 \times 0.0035)} = 0.233$$

$$1.5\rho_{\mathrm{fb}} = 1.5 \times \frac{1.0 \times 23.3}{426.7} \times 0.233 = 1.91\%$$

$$> \frac{884.9}{83.5 \times 1{,}000} = 1.06\%$$

→The flexural strength is controlled by FRP rupture.

$$x_{\mathrm{sb}} = \left[\frac{0.14}{1 + 400 \times (426.7/50000)} + \frac{1.06\% \times 426.7}{23.3}\right] \times 83.5 = 18.83\ \mathrm{mm}$$

$$M_{\mathrm{Rd}} = 0.85 \times 884.9 \times 426.7 \times \left(83.5 - \frac{18.83}{2}\right) \times 10^{-6} = 23.8\ \mathrm{kN \cdot m} > M_{\mathrm{Ed}} = 9.5\ \mathrm{kN \cdot m}$$

<u>Satisfy the flexural strength.</u>

<u>*Shear capacity*</u>

$$\tau_{\mathrm{Ed}} = \frac{22.5 \times 1{,}000}{1{,}000 \times (0.9 \times 83.5)} = 0.30\ \mathrm{MPa}$$

$$\tau_{\mathrm{Rd,f}} = \tau_{\mathrm{Rd,c}} = \frac{0.66}{1.4} \times \left(100 \times 1.06\% \times \frac{50{,}000}{200{,}000} \times 35 \times \frac{40}{83.5}\right)^{\frac{1}{3}} = 0.77\ \mathrm{MPa}$$

$$\tau_{\mathrm{Rdc,min}} = \frac{11}{1.4} \times \sqrt{\frac{35}{1{,}000} \times \frac{50{,}000}{200{,}000} \times \frac{40}{83.5}} = 0.51\ \mathrm{MPa}$$

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$$\tau_{\mathrm{Ed}}=\frac{V_{\mathrm{Ed}}}{b_{\mathrm{w}}\cdot z}=\frac{V_{\mathrm{Ed}}}{b_{\mathrm{w}}\cdot(0.9d)}$$ Shear resistance for members [**Clause R.8.2 of EN 1992**] $$\tau_{Rd,f}=\tau_{Rd,c}+\rho_w\cdot f_{fwRd}\cdot cot\,\theta \le 0.17\cdot f_{cd}$$ Since there is no shear reinforcement, ρ_w is taken as 0. The design value of the shear stress resistance for member without shear reinforcement: $$\tau_{Rd,c}=\frac{0.66}{\gamma_V}\cdot(100\cdot\rho_l\cdot\frac{E_{\mathrm{fR}}}{E_{\mathrm{s}}}\cdot f_{\mathrm{ck}}\cdot\frac{d_{dg}}{d})^{\frac{1}{3}}$$ The minimum shear stress resistance: $$\tau_{Rdc,min}=\frac{11}{\gamma_V}\sqrt{\frac{f_{ck}}{f_{ftk0}}\cdot\frac{E_{fR}}{E_s}\cdot\frac{d_{dg}}{d}}$$ where $\gamma_{\mathrm{V}}=1.40$ [**Table 4.3 of EN 1992**]	$$\tau_{\mathrm{Rd,f}}=0.77\text{ MPa}>\tau_{\mathrm{Rdc,min}}=0.51\text{ MPa}$$ $$\tau_{\mathrm{Rd,f}}=0.77\text{ MPa}>\tau_{\mathrm{Ed}}=0.30\text{ MPa}$$ Satisfy the shear strength (No need for shear reinforcement).
Step 3 – Stress limitation and crack control under service loads Assume $\alpha_{\mathrm{e}}=E_{\mathrm{s}}/E_{\mathrm{c,eff}}$ to be 15 referring to **Clause 9.1(5) of EN 1992**, where $E_{\mathrm{c,eff}}$ is the effective elastic modulus of concrete considering long-term creep, service stresses are evaluated based on the analysis of cracked section: $$A=\frac{1}{2}E_{\mathrm{c,eff}}b$$ $$B=E_{\mathrm{f}}A_{\mathrm{f}}$$ $$C=-E_{\mathrm{f}}A_{\mathrm{f}}d_{\mathrm{f}}$$ $$x_{\mathrm{cr}}=\frac{-B+\sqrt{B^2-4AC}}{2A}$$ $$I_{\mathrm{cr}}=\frac{1}{3}bx_{\mathrm{cr}}^3+\frac{E_{\mathrm{f}}}{E_{\mathrm{c,eff}}}A_{\mathrm{f}}(d_{\mathrm{f}}-x_{\mathrm{cr}})^2$$ *Service stresses of FRP reinforcement* [**Clause 2.8.1(3)**] Stress check under quasi-permanent load combination (sustained load)	$$E_{\mathrm{c,eff}}=\frac{200000}{15}=13333.3\text{ MPa}$$ Analysis of cracked section $$A=\frac{1}{2}\times13333.3\times1000=6666666.7$$ $$B=50000\times884.9=44244096.5$$ $$C=-50000\times884.9\times83.5=-3694382061$$ $$x_{\mathrm{cr}}=\frac{-44244096.5+\sqrt{44244096.5^2+4\times6666666.7\times3694382061}}{2\times6666666.7}=20.45\text{ mm}$$ $$I_{\mathrm{cr}}=\frac{1}{3}\times1000\times20.45^3+\frac{50000}{13333.3}\times884.9\times(83.5-20.45)^2=1.60\times10^7\text{ mm}^4$$ *Service stress of FRP reinforcement under quasi-permanent load combination (sustained load)* $$\sigma_{\mathrm{f,Eq}}=\frac{50000}{13333.3}\times\frac{4.9\times10^6}{1.60\times10^7}\times(83.5-20.45)=95.3\text{ MPa}<0.3\times640.0=192.0\text{ MPa}$$

$$\sigma_{\mathrm{f,Eq}} = \frac{E_{\mathrm{f}}}{E_{\mathrm{c,eff}}}\frac{M_{\mathrm{Eq}}}{I_{\mathrm{cr}}}(d_{\mathrm{f}} - x_{\mathrm{cr}}) \le C_{\mathrm{C}} \cdot f_{\mathrm{ftk,100a}}$$ <u>*Crack control*</u> (**Clause 2.8.2**) FRP stress under characteristic load combination $$f_{\mathrm{fs}} = \frac{E_{\mathrm{f}}}{E_{\mathrm{c,eff}}}\frac{M_{\mathrm{Ec}}}{I_{\mathrm{cr}}}(d_{\mathrm{f}} - x_{\mathrm{cr}}) \le \frac{0.296E_{\mathrm{fR}}}{d_{\mathrm{c}}\beta_{\mathrm{cr}}}$$ The spacing of reinforcement nearest to the tension face (unit: mm) shall not exceed the following limit: $$s \le \min\left\{\frac{0.677E_{\mathrm{fR}}}{f_{\mathrm{fs}}} - 2.5c;\ \frac{0.550E_{\mathrm{fR}}}{f_{\mathrm{fs}}}\right\}$$	<u>Satisfy the stress limit for preventing creep rupture.</u> <u>*Service stress of FRP reinforcement under characteristic load combination*</u> $$f_{\mathrm{fs}} = \frac{50000}{13333.3} \times \frac{6.7\times 10^{6}}{1.60\times 10^{7}} \times (83.5 - 20.45) = 98.7\ \mathrm{MPa}$$ $$< \frac{0.296\times 50000}{(120 - 83.5)\times\frac{120 - 20.45}{83.5 - 20.45}}$$ $$= 256.8\ \mathrm{MPa}$$ $$s = 150\ \mathrm{mm}$$ $$< \min\left\{\frac{0.677\times 50000}{98.7} - 2.5\times 30;\ \frac{0.550\times 50000}{98.7}\right\}$$ $$= 267.8\ \mathrm{mm}$$ <u>Satisfy the requirement for crack control.</u>
Step 4 – Deflection control The short-term flexural stiffness of a cracked FRP-reinforced section may be calculated as follows (**Clause 2.8.3**): $$B_{\mathrm{s}} = \frac{E_{\mathrm{fR}}A_{\mathrm{f}}d^{2}}{1.15\psi_{\mathrm{f}} + 0.2 + \frac{6\alpha_{\mathrm{fE}}\rho_{\mathrm{f}}}{1 + 3.5\gamma_{\mathrm{f}}}},$$ $$\psi_{\mathrm{f}} = 1.1 - 0.65\cdot\frac{f_{\mathrm{ctm}}}{\rho_{\mathrm{p,eff}}\cdot\sigma_{\mathrm{f}}}\cdot\frac{E_{\mathrm{fR}}}{E_{\mathrm{s}}}$$ $$\rho_{\mathrm{p,eff}} = \frac{A_{\mathrm{f}}}{A_{\mathrm{c,eff}}}$$ $$A_{\mathrm{c,eff}} = 0.5b_{\mathrm{w}}h + (b_{\mathrm{f}} - b_{\mathrm{w}})h_{\mathrm{f}}$$ $$\sigma_{\mathrm{f}} = \frac{M_{\mathrm{Eq}}}{0.9\cdot A_{\mathrm{f}}\cdot d_{\mathrm{f}}}$$ $$\rho_{\mathrm{f}} = A_{\mathrm{f}}/(b_{\mathrm{w}}\cdot d_{\mathrm{f}})$$ $$\alpha_{\mathrm{fE}} = E_{\mathrm{fR}}/E_{\mathrm{cm}}$$ $$\gamma_{\mathrm{f}}' = [(b_{\mathrm{f}}' - b_{\mathrm{w}})h_{\mathrm{f}}']/(b_{\mathrm{w}}\cdot d_{\mathrm{f}})$$ The long-term flexural stiffness of a cracked section under the quasi-permanent combination of actions $$B = \frac{B_{\mathrm{s}}}{\theta}$$ For the cantilever slab,	$$\sigma_{\mathrm{f}} = \frac{4.9\times 10^{6}}{0.9\times 884.9\times 83.5} = 73.7\ \mathrm{MPa}$$ $$A_{\mathrm{c,eff}} = 0.5\times 1000\times 120 = 60000\ \mathrm{mm}^{2}$$ $$\rho_{\mathrm{p,eff}} = \frac{884.9}{60000} = 0.0147$$ $$\psi_{\mathrm{f}} = 1.1 - 0.65\times\frac{3.2}{0.0147\times 73.7}\times\frac{50000}{200000} = 0.621$$ $$\alpha_{\mathrm{fE}} = \frac{50000}{33282.3} = 1.50$$ $$\gamma_{\mathrm{f}}' = 0$$ $$B_{\mathrm{s}} = \frac{50000\times 884.9\times 83.5^{2}}{1.15\times 0.621 + 0.2 + \frac{6\times 1.50\times 1.06\%}{1 + 3.5\times 0}}$$ $$= 3.05\times 10^{11}\ \mathrm{mm}^{4}$$ $$B = \frac{2.50\times 10^{11}}{2} = 1.53\times 10^{11}\ \mathrm{N\cdot mm}^{2}$$ $$\delta_{\mathrm{Eq}} = \frac{17.5\times 750^{4}}{8\times 1.53\times 10^{11}} = 4.53\ \mathrm{mm}$$ $$< 750\times 2/250 = 6\ \mathrm{mm}$$ <u>Satisfy the requirement of deflection control.</u>

$$\delta_{\mathrm{Eq}} = \frac{q_{\mathrm{q}} \cdot l^4}{8B} < 2l_0/250$$	
Step 5 – Check the reinforcement detailing ***<u>Minimum reinforcement ratio</u>*** [**Clauses 2.11.2(1),(2)**] $$M_{\mathrm{R,min}} \geq M_{\mathrm{cr}}$$ $$A_{\mathrm{f,min}} \geq 140/E_{\mathrm{fR}} \cdot A_{\mathrm{c}}$$ ***<u>Maximum reinforcement ratio</u>*** [**Clause R.5.1(3) of EN 1992**]: $\rho_{\mathrm{lf}} \leq 0.05$ ***<u>Maximum spacing of bars</u>*** [**Clause 2.11.2(3)**] $$s_{\mathrm{slab,max}} \leq \max\{3h, 250\ \mathrm{mm}\}$$	Longitudinal direction & Transverse direction: *<u>Reinforcement ratio</u>* $$I_{\mathrm{g}} = \frac{1000 \times 120^3}{12} = 1.44 \times 10^8\ \mathrm{mm}^4$$ $$M_{\mathrm{cr}} = \frac{3.2}{120/2} \times 1.44 \times 10^8 \times 10^{-6} = 7.68\ \mathrm{kN \cdot m}$$ $$M_{\mathrm{Rd}} = 23.8\ \mathrm{kN \cdot m} > M_{\mathrm{cr}} = 7.68\ \mathrm{kN \cdot m}$$ $$A_{\mathrm{fl}} = A_{\mathrm{ft}} = 884.9\ \mathrm{mm}^2 \geq A_{\mathrm{f,min}}$$ $$= \frac{140}{50{,}000} \times 1{,}000 \times 120 = 336\ \mathrm{mm}^2$$ $$\rho_{\mathrm{lf}} = 1.06\% \leq 5\%$$ <u>Satisfy the requirement.</u> *<u>Bar spacing</u>* $$s_{\mathrm{slab,l}} = 150\ \mathrm{mm} \leq \max\{3 \times 120\ \mathrm{mm}, 250\ \mathrm{mm}\}$$ $$= 250\ \mathrm{mm}$$ <u>Satisfy the requirement.</u>

2. Example of GFRP-reinforced beam

A simply supported beam in a precast beam-slab of a marine concrete pier is taken as a design example for GFRP-reinforced beam. The schematic diagram of the analysed beam is shown in **Figure 2.1**, and uniformly distributed dead loads and live loads are assumed to be applied on the beam. Since the beam and slab are prefabricated as a whole, the flexural strength of the beams can be evaluated according to a T-shaped section formed by the beam web and a portion of the slab on it. The dimensions and reinforcement detailing of the analysed beam are displayed in **Table 2.1**, the relevant material properties in **Table 2.2**, the assumed loading conditions in **Table 2.3**, and the design calculations in **Table 2.4**. In the calculation of the flexural strength of the beam, the contribution of the compression reinforcement is not considered according to Clause 8.1(3).

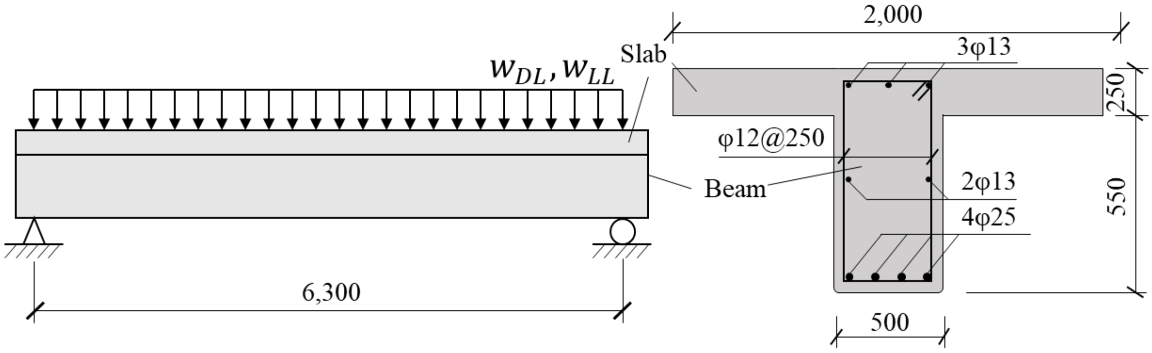


Figure 2.1 — Schematic diagram of the GFRP-reinforced beam

Table 2.1 — Values of parameters

Parameter	**Value**
l_0 (Effective span of the beam)	6,300 mm
b_w (Width of the beam web)	500 mm
b_f' (Flange width of T-section)	2,000 mm
h (Height of the beam)	800 mm
h_f' (Flange thickness of T-section)	250 mm
c (Concrete cover thickness)	50 mm
d (Effective depth of the section)	737.5 mm
ϕ_fl (Diameter of GFRP bottom longitudinal bars)	25 mm
Number of longitudinal bars in the bottom	4
A_fl (Cross-sectional area of GFRP bottom reinforcement)	1963.5 mm^2
ϕ_ft (Diameter of GFRP transverse bars)	12 mm
s (Spacing of GFRP transverse bars)	250 mm
r_b (Bending radius at corners of GFRP transverse bars)	120 mm

Table 2.2 — Material properties

Material property	**Value**
Concrete	C35/45
f_{ck} (Characteristic concrete cylinder compressive strength)	35 MPa
f_{cm} (Mean concrete cylinder compressive strength)	43 MPa
$\varepsilon_{\mathrm{cu}}$ (Ultimate compressive strain in the concrete)	0.0035
E_{cm} (Secant modulus of elasticity of concrete)	33,282.3 MPa
f_{ctm} (Mean axial tensile strength of concrete)	3.2 MPa
GFRP bars	
f_{ftk0} (Characteristic value of tensile strength of GFRP reinforcement)	1000 MPa
$\varepsilon_{\mathrm{ftk0}}$ (Characteristic ultimate strain of GFRP reinforcement)	0.02
E_{fR}, E_{fwR} (Mean modulus of elasticity of GFRP reinforcement)	50,000 MPa

Table 2.3 — Loading conditions

Load case	**Design loads**
w_{DL} (Dead Load+ Service Dead Load)	20 kN/m
$w_{\mathrm{LL,leading}}$ (Leading Live Load)	20 kN/m
$w_{\mathrm{LL,accompanying}}$ (Accompanying Live load)	2.5 kN/m
ULS	
Design load effects ($w_{\mathrm{d}} = 1.35w_{\mathrm{DL}} + 1.5w_{\mathrm{LL,leading}} + 1.5 \times 0.5w_{\mathrm{LL,accompanying}} = 58.88\ \mathrm{kN/m}$)	
Moment $M_{\mathrm{Ed}} = w_d {l_0}^2/8$	292.1 kN·m
Shear $V_{\mathrm{Ed}} = w_d(l_0 - 2d)/2$	142.0 kN
SLS	
Characteristic load effects ($w_{\mathrm{c}} = 1.0w_{\mathrm{DL}} + 1.0w_{\mathrm{LL,leading}} + 1.0 \times 0.5w_{\mathrm{LL,accompanying}} = 41.25\ \mathrm{kN/m}$)	
Moment $M_{\mathrm{Ec}} = w_c {l_0}^2/8$	204.7 kN·m
Quasi-permanent load effects for Load case 1 ($w_{\mathrm{q}} = 1.0w_{\mathrm{DL}} + 0.5w_{\mathrm{LL,leading}} = 30.00\ \mathrm{kN/m}$)	
Moment $M_{\mathrm{Eq}} = w_{\mathrm{q}} {l_0}^2/8$	148.8 kN·m

Table 2.4 — Steps for design checking of GFRP-reinforced beam

Steps	**Calculations**
Step 1- Calculate the design material properties Characteristic long term tensile strength of GFRP reinforcement [**Formula (77)**]: $f_{\mathrm{ftk,100a}} = C_{\mathrm{t}} \cdot C_{\mathrm{e}} \cdot f_{\mathrm{ftk0}}$ Design tensile strength of GFRP	$f_{\mathrm{ftd}} = 0.80 \times 0.80 \times 1{,}000 = 640.0\ \mathrm{MPa}$ $f_{\mathrm{ftd}} = \dfrac{640.0}{1.50} = 426.7\ \mathrm{MPa}$ According to **Clause 5.1.6(1) of EN 1992**, $\eta_{\mathrm{cc}} = 1.0$, $k_{\mathrm{tc}} = 1.0$; then,

reinforcement [**Formula (76)**]: $$f_{\mathrm{ftd}} = \frac{f_{\mathrm{ftk,100a}}}{\gamma_{\mathrm{FRP}}}$$ For the GFRP reinforcement used for marine concrete pier, the factor considering temperature effects C_{t} is taken to be 0.80, the environmental reduction factor C_{e} is taken to be 0.80, the partial factor γ_{f} is taken to be 1.50. Design compressive strength of concrete [**Formula (5.3) of EN 1992**]: $$f_{\mathrm{cd}} = \eta_{\mathrm{cc}} k_{\mathrm{tc}} \frac{f_{\mathrm{ck}}}{\gamma_c}$$	$$f_{\mathrm{cd}} = 1.0 \times 1.0 \times \frac{35}{1.5} = 23.3\ \mathrm{MPa}$$
Step 2- Check the strength of the GFRP bar reinforced beam ***Moment capacity*** Stress distribution within the compression zone (rectangular stress block) (**Clause 8.1.2 of EN 1992**) $$\alpha_1 = 1.0, \beta_1 = 0.8$$ Critical reinforcement ratio and depth of the compression zone for different failure modes, assuming a rectangular stress block [**Formulae (78) and (79)**]: $$\xi_{\mathrm{b}} = \frac{\beta_1}{1 + f_{\mathrm{ftd}}/(E_{\mathrm{fR}}\varepsilon_{\mathrm{cu}})}$$ $$\rho_{\mathrm{fb}} = \frac{\alpha_1 f_{\mathrm{cd}}}{f_{\mathrm{ftd}}} \cdot \xi_{\mathrm{b}}$$ Equilibrium function (singly-reinforced) for flexural strength is controlled by FRP rupture [**Formulae (80) to (82)**]: $$\sigma_{\mathrm{ftd}} = f_{\mathrm{ftd}}$$ $$x_{\mathrm{sb}} = \left[\frac{0.14}{1 + 400(\sigma_{\mathrm{ftd}}/E_{\mathrm{fR}})} + \frac{\rho_{\mathrm{lf}}\sigma_{\mathrm{ftd}}}{f_{\mathrm{cd}}}\right] d$$ $$M_{\mathrm{Rd}} = \varphi A_{\mathrm{f}} \sigma_{\mathrm{ftd}} (d - \frac{x_{\mathrm{sb}}}{2})$$ $$\varphi = 0.85$$ ***Shear capacity***	*Moment capacity* $$\xi_{\mathrm{b}} = \frac{0.8}{1 + 426.7/(50{,}000 \times 0.0035)} = 0.233$$ $$\xi_{\mathrm{b}} d = 0.233 \times 737.5 = 171.6\ \mathrm{mm} < h_{\mathrm{f}}' = 250\ \mathrm{mm}$$ For critical state of simultaneous failure of FRP rupture and concrete crushing, the neutral axis locates within the flange, the reinforcement ratio for determining flexural strength should be calculated based on the width of flange in compression: $$\rho_{\mathrm{lf}} = \frac{1963.5}{737.5 \times 2{,}000} = 0.13\%$$ $$1.5 \times \rho_{\mathrm{fb}} = 1.5 \times \frac{1.0 \times 23.3}{426.7} \times 0.233 = 1.91\% > 0.13\%$$ →The flexural strength is controlled by FRP rupture. $$x_{\mathrm{sb}} = \left[\frac{0.14}{1 + 400 \times (426.7/50000)} + \frac{0.13\% \times 426.7}{23.3}\right] \times 737.5 = 41.35\ \mathrm{mm} < 250\ \mathrm{mm}$$ →The neutral axis locates within the flange $$M_{\mathrm{Rd}} = 0.85 \times 1963.5 \times 453.3 \times \left(737.5 - \frac{41.35}{2}\right) \times 10^{-6} = 510.4\ \mathrm{kN \cdot m} > M_{\mathrm{Ed}} = 292.1\ \mathrm{kN \cdot m}$$ Satisfy the flexural strength. *Shear capacity* $$\tau_{\mathrm{Ed}} = \frac{142.0 \times 1{,}000}{500 \times (0.9 \times 737.5)} = 0.428\ \mathrm{MPa}$$

Average shear stress over the cross-section [**Formula (8.18) of EN 1992**] $$\tau_{\mathrm{Ed}}=\frac{V_{\mathrm{Ed}}}{b_{\mathrm{w}}\cdot z}=\frac{V_{\mathrm{Ed}}}{b_{\mathrm{w}}\cdot(0.9d)}$$	For calculating the shear capacity, the FRP reinforcement ratio is calculated by, $$\rho_{\mathrm{l}}=\frac{1963.5}{737.5\times 500}=0.53\%$$
Shear resistance (**Clause 2.7.2**): $$\tau_{Rd,c}=\frac{0.66}{\gamma_V}\cdot(100\cdot\rho_l\cdot\frac{E_{\mathrm{fR}}}{E_{\mathrm{s}}}\cdot f_{\mathrm{ck}}\cdot\frac{d_{dg}}{d})^{\frac{1}{3}}$$	$$\tau_{\mathrm{Rd,c}}=\frac{0.66}{1.4}\times\left(100\times 0.53\%\times\frac{50{,}000}{200{,}000}\times 35\times\frac{40}{737.5}\right)^{\frac{1}{3}}=0.298\ \mathrm{MPa}$$ $$\tau_{\mathrm{Rdc,min}}=\frac{11}{1.4}\times\sqrt{\frac{35}{1{,}000}\times\frac{50{,}000}{200{,}000}\times\frac{40}{737.5}}=0.17\ \mathrm{MPa}$$ $$\tau_{\mathrm{Rd,c}}=0.298\ \mathrm{MPa}>\tau_{\mathrm{Rdc,min}}=0.17\ \mathrm{MPa}$$
Strain of FRP shear reinforcement at design tensile strength $$\varepsilon_{fwRd}=0.0023+1/15\cdot E_{fR}\cdot A_{\mathrm{fl}}\cdot(0.8\cdot d)^2\cdot 10^{-15}\le 0.005$$	$$\varepsilon_{\mathrm{fwRd}}=0.0023+\frac{1}{15}\times 50{,}000\times 1963.5\times(0.8\times 737.5)^2\cdot 10^{-15}=0.00457<0.005$$
Design tensile strength of FRP shear reinforcement $$f_{fwRd}\le\varepsilon_{fwRd}\cdot E_{fwR}$$	$$f_{\mathrm{fwRd}}=50{,}000\times 0.00457=228.9\ \mathrm{MPa}$$ $$f_{fwRd}=228.9\ \mathrm{MPa}$$
Shear resistance for members $$\tau_{Rd,f}=\tau_{Rd,c}+\rho_w\cdot f_{fwRd}\cdot cot\,\theta\le 0.17\cdot f_{cd}$$ where $cot\,\theta=0.8$. The minimum shear stress resistance: $$\tau_{Rdc,min}=\frac{11}{\gamma_V}\sqrt{\frac{f_{ck}}{f_{ftk0}}\cdot\frac{E_{fR}}{E_s}\cdot\frac{d_{dg}}{d}}$$ where $\gamma_{\mathrm{V}}=1.40$ [**Table 4.3 of EN 1992**]	$$\tau_{Rd,f}=\tau_{Rd,c}+\rho_w\cdot f_{fwRd}\cdot cot\,\theta=0.298+\frac{2\times 12^2\times\pi/4}{500\times 250}\times 228.9\times 0.8=0.63\ \mathrm{MPa}<0.17\times 23.3=3.97\ \mathrm{MPa}$$ $$\tau_{\mathrm{Rd,f}}=0.63\ \mathrm{MPa}>\tau_{\mathrm{Ed}}=0.428\ \mathrm{MPa}$$ <u>Satisfy the shear strength.</u>
Step 3 – Stress limitation and crack control under service loads Assume $\alpha_{\mathrm{e}}=E_{\mathrm{s}}/E_{\mathrm{c,eff}}$ to be 15 referring to **Clause 9.1(5) of EN 1992**, where $E_{\mathrm{c,eff}}$ is the effective elastic modulus of concrete considering long-term creep, service stresses are evaluated based on the analysis of cracked section: $$A=\frac{1}{2}E_{\mathrm{c,eff}}b_{\mathrm{f}}'$$	$$E_{\mathrm{c,eff}}=\frac{200000}{15}=13333.3\ \mathrm{MPa}$$ <u>Analysis of cracked section</u> $$A=\frac{1}{2}\times 13333.3\times 2000=13333333.3$$ $$B=50000\times 1963.5=98174770.4$$ $$C=-50000\times 1963.5\times 737.5=-72403893188$$ $$x_{\mathrm{cr}}=\frac{-98174770.4+\sqrt{98174770.4^2+4\times 13333333.3\times 72403893188}}{2\times 13333333.3}=70.10\ \mathrm{mm}$$

$B = E_f A_f$ $C = -E_f A_f d_f$ $x_{cr} = \frac{-B + \sqrt{B^2 - 4AC}}{2A}$ $I_{cr} = \frac{1}{3} b_f' x_{cr}^3 + \frac{E_f}{E_{c,eff}} A_f (d_f - x_{cr})^2$ *Service stresses of FRP reinforcement* [**Clause 2.8.1(3)**] Stress check under quasi-permanent load combination (sustained load) $\sigma_{f,Eq} = \frac{E_f}{E_{c,eff}} \frac{M_{Eq}}{I_{cr}} (d_f - x_{cr}) \leq C_C \cdot f_{ftk,100a}$ *Crack control* (**Clause 2.8.2**) FRP stress under characteristic load combination $f_{fs} = \frac{E_f}{E_{c,eff}} \frac{M_{Ec}}{I_{cr}} (d_f - x_{cr}) \leq \frac{0.296 E_{fR}}{d_c \beta_{cr}}$ The spacing of reinforcement nearest to the tension face (unit: mm) shall not exceed the following limit: $s \leq \min\left\{\frac{0.677 E_{fR}}{f_{fs}} - 2.5c; \frac{0.550 E_{fR}}{f_{fs}}\right\}$	$I_{cr} = \frac{1}{3} \times 2000 \times 70.10^3$ $+ \frac{50000}{13333.3} \times 1963.5 \times (737.5 - 70.10)^2$ $= 3.51 \times 10^9\ \text{mm}^4$ *Service stress of FRP reinforcement under quasi-permanent load combination (sustained load)* $\sigma_{f,Eq} = \frac{50000}{13333.3} \times \frac{148.8 \times 10^6}{3.51 \times 10^9} \times (737.5 - 70.10)$ $= 106.1\ \text{MPa} < 0.3 \times 640.0 = 192.0\ \text{MPa}$ Satisfy the stress limit for preventing creep rupture. *Service stress of FRP reinforcement under characteristic load combination* $f_{fs} = \frac{50000}{13333.3} \times \frac{204.7 \times 10^6}{3.51 \times 10^9} \times (737.5 - 70.10) = 145.9\ \text{MPa}$ $< \frac{0.296 \times 50000}{(800 - 737.5) \times \frac{800 - 70.10}{737.5 - 70.10}} = 216.5\ \text{MPa}$ $s = \frac{500 - 50 \times 2 - 25}{3} - 25 = 100\ \text{mm}$ $< \min\left\{\frac{0.677 \times 50000}{145.9} - 2.5 \times 50; \frac{0.550 \times 50000}{145.9}\right\}$ $= 106.9\ \text{mm}$ Satisfy the requirement for crack control.
Step 4 – Deflection control The short-term flexural stiffness of a cracked FRP-reinforced section may be calculated as follows (**Clause 2.8.3**): $B_s = \frac{E_{fR} A_f d^2}{1.15\psi_f + 0.2 + \frac{6\alpha_{fE}\rho_f}{1 + 3.5\gamma_f'}}$ $\psi_f = 1.1 - 0.65 \cdot \frac{f_{ctm}}{\rho_{p,eff} \cdot \sigma_f} \cdot \frac{E_{fR}}{E_s}$ $\rho_{p,eff} = \frac{A_f}{A_{c,eff}}$ $A_{c,eff} = 0.5 b_w h + (b_f - b_w) h_f$ $\sigma_f = \frac{M_{Eq}}{0.9 \cdot A_f \cdot d_f}$ $\rho_f = A_f / (b_w \cdot d_f)$	$\sigma_f = \frac{148.8 \times 10^6}{0.9 \times 1963.5 \times 737.5} = 114.2\ \text{MPa}$ $A_{c,eff} = 0.5 \times 500 \times 800 = 200000\ \text{mm}^2$ $\rho_{p,eff} = \frac{1963.5}{200000} = 0.0098$ $\psi_f = 1.1 - 0.65 \times \frac{3.2}{0.0098 \times 114.2} \times \frac{50000}{200000} = 0.636$ $\alpha_{fE} = \frac{50000}{33282.3} = 1.50$ $\gamma_f' = \frac{(2000 - 500) \times 250}{(500 \times 737.5)} = 1.02$ $\rho_f = \frac{1963.5}{737.5 \times 500} = 0.53\%$

$$\alpha_{\mathrm{fE}} = E_{\mathrm{fR}}/E_{\mathrm{cm}}$$ $$\gamma_{\mathrm{f}}' = \left[\left(b_{\mathrm{f}}' - b_{\mathrm{w}}\right) h_{\mathrm{f}}'\right]/(b_{\mathrm{w}} \cdot d_{\mathrm{f}})$$ The long-term flexural stiffness of a cracked section under the quasi-permanent combination of actions $$B = \frac{B_{\mathrm{s}}}{\theta}$$ For the cantilever slab, $$\delta_{\mathrm{Eq}} = \frac{5 q_{\mathrm{q}} l^4}{384 B} < l_0/250$$	$$B_{\mathrm{s}} = \frac{50000 \times 1963.5 \times 737.5^2}{1.15 \times 0.636 + 0.2 + \frac{6 \times 1.50 \times 0.53\%}{1 + 3.5 \times 1.02}}$$ $$= 5.67 \times 10^{13}\ \mathrm{mm}^4$$ $$B = \frac{5.67 \times 10^{13}}{2} = 2.83 \times 10^{13}\ \mathrm{N \cdot mm^2}$$ $$\delta_{\mathrm{Eq}} = \frac{5 \times 30 \times 6300^4}{384 \times 2.83 \times 10^{13}} = 21.7\ \mathrm{mm}$$ $$< 6300/250 = 25.2\ \mathrm{mm}$$ <u>Satisfy the requirement of deflection control.</u>
Step 5 – Check the reinforcement detailing ***<u>Minimum longitudinal reinforcement ratio</u>*** [**Clauses 2.11.2(1),(2)**] $$M_{\mathrm{R,min}} \geq M_{\mathrm{cr}}$$ $$A_{\mathrm{f,min}} \geq 150/E_{\mathrm{fR}} \cdot A_{\mathrm{c}}$$ ***<u>Maximum reinforcement ratio</u>*** [**Clause R.5.1(3) of EN 1992**]: $\rho_{\mathrm{lf}} \leq 0.05$ ***<u>Maximum spacing of longitudinal bars</u>*** [**Clause 2.11.2(3)**] $s_{\max} \leq \min\{3h, 300\ \mathrm{mm}\}$ ***<u>Minimum shear reinforcement ratio</u>*** [**Clause 12.2(4) of EN 1992**] $$\rho_{\mathrm{w}} \geq 0.08 \frac{\sqrt{f_{\mathrm{ck}}}}{f_{\mathrm{ftd}}}$$ ***<u>Maximum spacing of shear reinforcement</u>*** (**Table 12.1 of EN 1992**) $$s_{\mathrm{l,max}} \leq \min\{0.75d(1 + \cot\alpha), 300\ \mathrm{mm}\}$$	Bottom longitudinal reinforcement & Top longitudinal reinforcement: *<u>Reinforcement ratio</u>* Analysis of gross section Central axis: $$h_b = \frac{2000 \times 250 \times \frac{250}{2} + 500 \times 550 \times (250 + \frac{550}{2})}{2000 \times 250 + 500 \times 550}$$ $$= 266.9\ \mathrm{mm}$$ $$I = \frac{2{,}000 \times 250^3}{12} + 2{,}000 \times 250 \times \left(266.9 - \frac{250}{2}\right)^2 + \frac{500 \times 550^3}{12}$$ $$+ 500 \times 550 \times (250 + 550/2 - 266.9)^2$$ $$= 3.79 \times 10^{10}\ \mathrm{mm}^4$$ $$M_{\mathrm{cr}} = \frac{3.2 \times 3.79 \times 10^{10}}{800 - 266.9} \times 10^{-6} = 227.5\ \mathrm{kN \cdot m}$$ $$M_{\mathrm{Rd}} = 510.4\ \mathrm{kN \cdot m} > M_{\mathrm{cr}} = 227.5\ \mathrm{kN \cdot m}$$ $$A_{\mathrm{fl}} = 1963.5\ \mathrm{mm}^2 \geq A_{\mathrm{f,min}} = \frac{150}{50{,}000} \times 500 \times 800$$ $$= 1{,}200\ \mathrm{mm}^2$$ $$\rho_{\mathrm{lf}} = 0.53\% \leq 5\%$$ <u>Satisfy the requirement.</u> *<u>Bar spacing</u>* $$s = 100\ \mathrm{mm} \leq \min\{3 \times 800\ \mathrm{mm}, 300\ \mathrm{mm}\} = 300\ \mathrm{mm}$$ <u>Satisfy the requirement.</u> Transverse reinforcement: *<u>Reinforcement ratio</u>*

	$$\rho_{\mathrm{w}} = \frac{2 \times 12^2 \times \pi/4}{500 \times 250} = 0.18\% \geq 0.08 \times \frac{\sqrt{35}}{426.7} = 0.11\%$$ Satisfy the requirement. *Bar spacing* $s = 250\ \mathrm{mm} \leq \min\{0.75 \times 737.5 \times (1+0), 250\} = 250\ \mathrm{mm}$ Satisfy the requirement.

3. Example of GFRP-reinforced column

A concrete column used as the bracing of a marine concrete pier is taken as a design example for GFRP-reinforced column, and its schematic diagram is shown in **Figure 3.1**. The column is assumed to be subjected to axial load and bending moment induced by lateral wave forces. The dimensions and reinforcement detailing of the analysed column are displayed in **Table 3.1**, the relevant material properties in **Table 3.2**, the assumed loading conditions in **Table 3.3**, and the design calculations in **Table 3.4**.

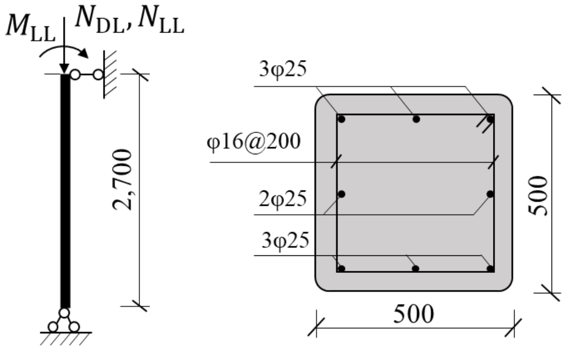


Figure 3.1 — Schematic diagram of the GFRP-reinforced column

Table 3.1 — Values of parameters

Parameter	Value
l_0 (Effective length of the column)	2,700 mm
h (Height of the section)	500 mm
b (Width of the section)	500 mm
c (Concrete cover thickness)	50 mm
d (Effective depth of the section)	437.5 mm
Number of longitudinal bars	8
ϕ_f (Diameter of GFRP longitudinal bars)	25 mm
ϕ_{ft} (Diameter of GFRP transverse bars)	16 mm
s_{col} (Spacing of GFRP transverse bars)	200 mm

Table 3.2 — Material properties

Material property	Value
Concrete	C35/45
f_{ck} (Characteristic concrete cylinder compressive strength)	35 MPa
f_{cm} (Mean concrete cylinder compressive strength)	43 MPa
ε_{cu} (Ultimate compressive strain in the concrete)	0.0035
E_{cm} (Secant modulus of elasticity of concrete)	33,282.3 MPa
f_{ctm} (Mean axial tensile strength of concrete)	3.2 MPa
GFRP bars	
f_{ftk0} (Characteristic value of tensile strength of GFRP reinforcement)	1,000 MPa
ε_{ftk0} (Characteristic ultimate strain of GFRP reinforcement)	0.02

E_{fR} (Mean modulus of elasticity of GFRP reinforcement)	50,000 MPa

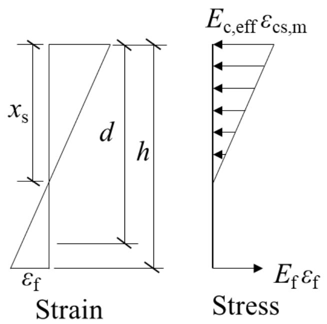
Table 3.3 — Loadings conditions

Loading/moment	**Design loads**
N_{DL} (Dead Load+ Service Dead Load)	100 kN
N_{LL} (Live Load)	200 kN
$M_{\mathrm{LL,2}}$ (Bending Moment due to Lateral Load)	100 kN·m
$M_{\mathrm{LL,1}}$ (Bending Moment due to Lateral Load)	0 kN·m
ULS	
N_{Ed} ($N_{\mathrm{Ed}} = 1.35N_{\mathrm{DL}} + 1.5N_{\mathrm{LL}}$)	$1.35 \times 100 + 1.5 \times 200 = 435$ kN
$M_{\mathrm{Ed,2}}$ ($M_{\mathrm{Ed}} = 1.5M_{\mathrm{LL,2}}$)	$1.5 \times 100 = 150$ kN·m
$M_{\mathrm{Ed,1}}$ ($M_{\mathrm{Ed}} = 1.5M_{\mathrm{LL,1}}$)	0 kN·m
SLS	
Characteristic load combination	
N_{Ec} ($N_{\mathrm{Ec}} = 1.0N_{\mathrm{DL}} + 1.0N_{\mathrm{LL}}$)	300 kN
M_{Ec} ($M_{\mathrm{Ec}} = 1.0M_{\mathrm{LL}}$)	100 kN·m
Quasi-permanent load combination	
N_{Eq} ($N_{\mathrm{Eq}} = 1.0N_{\mathrm{DL}} + 0.5N_{\mathrm{LL}}$)	200 kN
M_{Eq} ($M_{\mathrm{Eq}} = 0.5M_{\mathrm{LL}}$)	50 kN·m

Table 3.4 — Steps for design checking of GFRP-reinforced column

Steps	**Calculations**
Step 1 - Calculate the design material properties Characteristic long term tensile strength of GFRP reinforcement [**Formula (77)**]: $f_{\mathrm{ftk,100a}} = C_{\mathrm{t}} \cdot C_{\mathrm{e}} \cdot f_{\mathrm{ftk0}}$ Design tensile strength of GFRP reinforcement [**Formula (76)**]: $f_{\mathrm{ftd}} = \dfrac{f_{\mathrm{ftk,100a}}}{\gamma_{\mathrm{FRP}}}$ For the GFRP reinforcement used for marine concrete pier, the factor considering temperature effects C_{t} is taken to be 0.80, the environmental reduction factor C_{e} is taken to be 0.80, the partial factor γ_{f} is taken to be 1.50. Design compressive strength of concrete [**Formula (5.3) of EN 1992**]:	$f_{\mathrm{ftd}} = 0.80 \times 0.80 \times 1{,}000 = 640.0$ MPa $f_{\mathrm{ftd}} = \dfrac{640.0}{1.50} = 426.7$ MPa According to **Clause 5.1.6(1) of EN 1992**, $\eta_{\mathrm{cc}} = 1.0$, $k_{\mathrm{tc}} = 1.0$; then, $f_{\mathrm{cd}} = 1.0 \times 1.0 \times \dfrac{35}{1.5} = 23.3$ MPa Top layer of longitudinal bars: $A_{\mathrm{f1}} = 1472.7$ mm^2 Intermediate layer of longitudinal bars: $A_{\mathrm{f2}} = 981.7$ mm^2 Bottom layer of longitudinal bars: $A_{\mathrm{f3}} = 1472.7$ mm^2 $A_{\mathrm{f}} = A_{\mathrm{f1}} + A_{\mathrm{f2}} + A_{\mathrm{f3}} = 3927.0$ mm^2

$$f_{\mathrm{cd}} = \eta_{\mathrm{cc}} k_{\mathrm{tc}} \frac{f_{\mathrm{ck}}}{\gamma_c}$$	
Step 2 - Check the strength of the GFRP bar reinforced column ***<u>Slender effect consideration</u>* [Clause 2.7.1.3(7)]** Additional eccentricity $$e_{\mathrm{a}} = \max(20\mathrm{mm}, h/30)$$ Slenderness ratio $$\lambda = \frac{l_0}{i} = \frac{l_0}{\sqrt{I_c/A_c}}$$ When considering the slenderness effect, $$M = C_{\mathrm{m}} \eta_{\mathrm{ns}} M_2$$ $$C_{\mathrm{m}} = 0.7 + 0.3 \frac{M_1}{M_2}$$ $$\eta_{\mathrm{ns}} = 1 + \frac{1}{1000\left(\frac{M_2}{N} + e_{\mathrm{a}}\right)/d_{\mathrm{f}}} \left(\frac{l_0}{h}\right)^2 \zeta_{\mathrm{c}}$$ $$\zeta_{\mathrm{c}} = \frac{0.5 f_{\mathrm{c}} A}{N}$$ ***<u>Calculation of the strength of the GFRP-reinforced column under bending moment and compression load</u>*** Stress distribution within the compression zone (rectangular stress block) (**Clause 8.1.2 of EN 1992**) $$\alpha_1 = 1.0, \beta_1 = 0.8$$ Equilibrium function for flexural strength **[Clause 2.7.1.3(8)]**: $$\begin{cases} N_{\mathrm{Ed}} = \alpha_1 f_{\mathrm{cd}} b x_{\mathrm{sb}} - A_{\mathrm{f}} \sigma_{\mathrm{ftd}} \\ N_{\mathrm{Ed}} e \le \alpha_1 f_{\mathrm{cd}} b x_{\mathrm{sb}} \left(d_{\mathrm{c}} - \frac{x_{\mathrm{sb}}}{2}\right) \\ \sigma_{\mathrm{ftd}} = \varepsilon_{\mathrm{cu}} (\frac{\beta_1 d}{x_{\mathrm{sb}}} - 1) E_{\mathrm{fR}} \le f_{\mathrm{ftd}} \end{cases}$$ $$e = e_0 + e_{\mathrm{a}} + \frac{h}{2} - a_{\mathrm{f}}$$ $$e_0 = \frac{M}{N}$$	*<u>Eccentricity</u>* $$e_{\mathrm{a}} = \max\left(20\mathrm{mm}, \frac{500\ \mathrm{mm}}{30}\right) = 20\ \mathrm{mm}$$ $$e_2 = \frac{M_{\mathrm{Ed,2}}}{N_{\mathrm{Ed}}} = \frac{150 \times 10^6}{435 \times 10^3} = 344.8\ \mathrm{mm} > e_{\mathrm{d,min}}\ ;\ e_1 = 0$$ *<u>Slender effect consideration</u>* For the designed column not braced against sidesway, $$\lambda = \frac{2{,}700}{500/\sqrt{12}} = 18.7 > 17$$ <u>The slenderness effect needs to be considered.</u> $$C_{\mathrm{m}} = 0.7 + 0.3 \times \frac{0}{150} = 0.7$$ $$\zeta_{\mathrm{c}} = \frac{0.5 \times 23.3 \times 500 \times 500}{435 \times 10^3} = 6.7 > 1 \quad \Rightarrow \zeta_{\mathrm{c}} = 1$$ $$\eta_{\mathrm{ns}} = 1 + \frac{1}{1000 \times (344.8 + 20)/437.5} \times \left(\frac{2700}{500}\right)^2 \times 1$$ $$= 1.035$$ $$M = 0.7 \times 1.035 \times 150 = 108.7\ \mathrm{kN \cdot m}$$ *<u>Load capacity</u>* $$e_0 = \frac{108.7 \times 10^6}{435 \times 10^3} = 249.8\ \mathrm{mm}$$ $$e = 250.0 + 20 + \frac{500}{2} - (500 - 437.5) = 457.3\ \mathrm{mm}$$ $$\begin{cases} N_{\mathrm{Ed}} = 1.0 \times 23.3 \times 500 \times x_{\mathrm{sb}} - 1472.7 \times \sigma_{\mathrm{ftd,3}} \\ \sigma_{\mathrm{ftd,3}} = 0.0035 \times (\frac{0.8 \times 437.5}{x_{\mathrm{sb}}} - 1) \times 50000 \end{cases}$$ Depth of the compression zone assuming a stress block: $$x_{\mathrm{sb}} = 95.9\ \mathrm{mm} < 500/2 = 250\ \mathrm{mm}$$ →The assumption about the location of neutral axis holds true. Stress of FRP reinforcement: $$\sigma_{\mathrm{ftd,2}} = 162.7\ \mathrm{MPa} < f_{\mathrm{ftd}} = 426.7\ \mathrm{MPa}$$ $$\sigma_{\mathrm{ftd,3}} = 416.0\ \mathrm{MPa} < f_{\mathrm{ftd}} = 426.7\ \mathrm{MPa}$$ $$N_{\mathrm{Ed}} e = 435 \times 457.3 \times 10^{-3} = 198.9\ \mathrm{kN \cdot m}$$ $$< 1.0 \times 23.3 \times 500 \times 95.9 \times \left(437.5 - \frac{95.9}{2}\right)$$ $$\times 10^{-6} = 435.3\ \mathrm{kN \cdot m}$$

Calculation of the strength of the GFRP-reinforced column in the direction perpendicular to load eccentricity **[Clause 2.7.1.3(1)-(5)]** $N_{\mathrm{c,0}} = 0.85 f_{\mathrm{cd}} A_{\mathrm{c}}$ For transverse reinforcement in ties, $N_{\mathrm{c,max}} = 0.80 N_{c,0}$ $\mathrm{N_{Ed}} \leq N_{\mathrm{t,max}} = f_{\mathrm{ftd}} A_{\mathrm{fl}}$	Satisfy the strength under bending moment and compression load. *For checking the strength in the direction perpendicular to load eccentricity* $N_{\mathrm{Ed}} = 435\ \mathrm{kN} < 0.10 f_{\mathrm{cd}} A_{\mathrm{c}} = 0.1 \times 23.3 \times 500 \times 500 = 583.3\ \mathrm{kN}$ → No additional limit [**Clause 2.11.6(2)**]. $N_{\mathrm{c,0}} = 0.85 \times 23.3 \times 500 \times 500 = 4{,}958\ \mathrm{kN}$ $N_{\mathrm{c,max}} = 0.80 \times 4{,}958 = 3{,}967\ \mathrm{kN} > N_{\mathrm{Ed}} = 435\ \mathrm{kN}$ $N_{\mathrm{Ed}} = 435\ \mathrm{kN} < 426.7 \times 3927.0 = 1675.52\ \mathrm{kN}$ Satisfy the strength in the direction perpendicular to load eccentricity.
Step 3 – Stress limitation Assume $\alpha_{\mathrm{e}} = E_{\mathrm{s}}/E_{\mathrm{c,eff}}$ to be 15 referring to **Clause 9.1(5) of EN 1992**, where $E_{\mathrm{c,eff}}$ is the effective elastic modulus of concrete considering long-term creep, service stresses are evaluated based on the analysis of cracked section. The distributions of strain and stress over the section are displayed as follows,  *Service stresses of FRP reinforcement* [**Clause 2.8.1(3)**] Stress check under quasi-permanent load combination (sustained load) $\sigma_{\mathrm{f,Eq}} \leq C_{\mathrm{C}} \cdot f_{\mathrm{ftk,100a}}$	$E_{\mathrm{c,eff}} = \frac{200000}{15} = 13333.3\ \mathrm{MPa}$ Analysis of cracked section Solving the following system of equations under quasi-permanent load combination $\begin{cases} N_{\mathrm{Eq}} = \frac{1}{2} \times 13333.3 \times \varepsilon_{\mathrm{cs,m}} \times 500 \times x_{\mathrm{s}} - 1472.7 \times 50000 \times \varepsilon_{\mathrm{f}} \\ M_{\mathrm{Eq}} = \frac{1}{2} \times 13333.3 \times \varepsilon_{\mathrm{cs,m}} \times 500 \times x_{\mathrm{s}} \times \left(\frac{500}{2} - \frac{x_{\mathrm{s}}}{3}\right) + 1472.7 \times 50000 \times \varepsilon_{\mathrm{f}} \times \left(437.5 - \frac{500}{2}\right) \\ \varepsilon_{\mathrm{f}} = \frac{\varepsilon_{\mathrm{cs,m}}}{x_{\mathrm{s}}} \times (437.5 - x_{\mathrm{s}}) \end{cases}$ $\Longrightarrow x_{\mathrm{s}} = 269.2\ \mathrm{mm}$ Service stress of FRP reinforcement under quasi-permanent load combination (sustained load) $\sigma_{\mathrm{f,Eq}} = 35.0\ \mathrm{MPa} < 0.3 \times 640.0 = 192.0\ \mathrm{MPa}$ Satisfy the stress limit for preventing creep rupture.
Step 4 – Check the reinforcement detailing ***Minimum longitudinal reinforcement ratio*** [**Clauses 2.11.2(1),(2); Table 4**]	*Reinforcement ratio* $M_{\mathrm{R,min}}(N_{\mathrm{Ed,min}}) = 435.7 - 435 \times \left(437.5 - \frac{500}{2}\right) \times 10^{-3} = 354.9\ \mathrm{kN \cdot m}$

$M_{\mathrm{R,min}}(N_{\mathrm{Ed,min}}) \geq M_{\mathrm{cr}}(N_{\mathrm{Ed,min}})$ $A_{\mathrm{f,min}} \geq 150/E_{\mathrm{fR}} \cdot A_{\mathrm{c}}$ $1\% \leq \rho_{\mathrm{lf}} \leq 8\%$ ***<u>Maximum spacing of transverse reinforcement</u>*** **(Table 4)** $s_{\mathrm{max,col}}$: — $\tau_{\mathrm{Rd,f}} \leq 0.33\sqrt{f_{\mathrm{ck}}}$: $\min\{h/2; 600\ \mathrm{mm}\}$ — $\tau_{\mathrm{Rd,f}} > 0.33\sqrt{f_{\mathrm{ck}}}$: $\min\{h/2; d/2; 300\ \mathrm{mm}\}$	$I_{\mathrm{g}} = \dfrac{500^4}{12} = 5.21 \times 10^9$ $M_{\mathrm{cr}}(N_{\mathrm{Ed,min}}) = \dfrac{\dfrac{435 \times 10^3}{500 \times 500} + 3.2}{500/2} \times 5.21 \times 10^9 \times 10^{-6}$ $= 102.9\ \mathrm{kN \cdot m}$ $M_{\mathrm{R,min}}(N_{\mathrm{Ed,min}}) > M_{\mathrm{cr}}(N_{\mathrm{Ed,min}})$ $A_{\mathrm{fl}} = 5{,}654.9\ \mathrm{mm}^2 \geq \dfrac{150}{50{,}000} \times 500 \times 500 = 750\ \mathrm{mm}^2$ $\rho_{\mathrm{lf}} = \dfrac{5{,}654.9}{500 \times 500} = 2.26\%$ $1\% \leq \rho_{\mathrm{lf}} \leq 8\%$ <u>Satisfy the requirement.</u> *<u>Spacing of transverse reinforcement</u>* $s_{\mathrm{col}} = 200\ \mathrm{mm} \leq \min\{500/2; 500/2; 600\ \mathrm{mm}\} = 250\ \mathrm{mm}$ <u>Satisfy the requirement.</u>