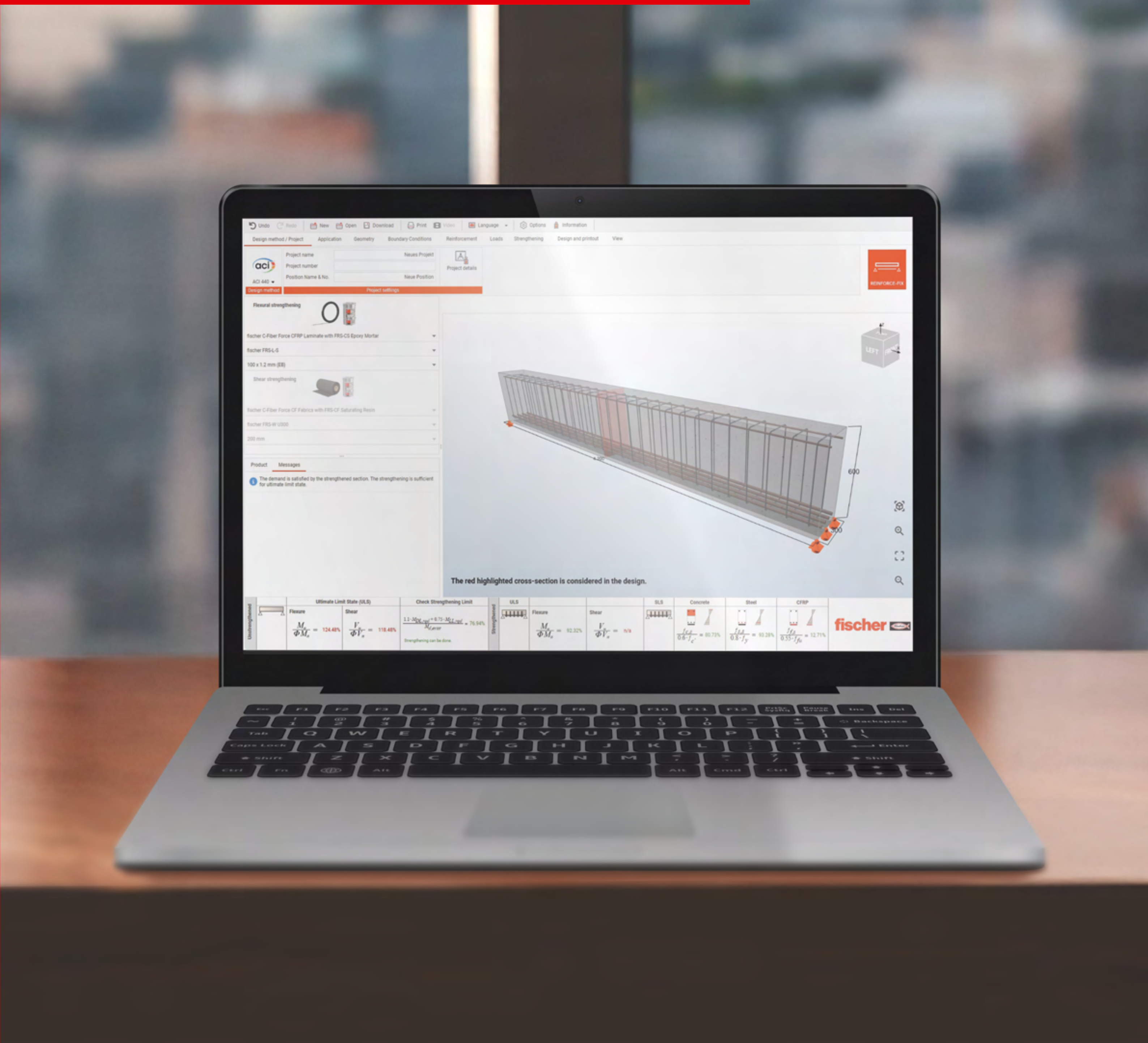
Expert Guide for Structural Strengthening.

Technical Manual for the design of C-Fiber Force Strengthening System according to ACI PRC-440.2-23



REINFORCE-FIX

Our new planning assistant for structural strengthening

Experience a new generation of structural strengthening design

fischer.de/fixperience

Introduction

Concrete is one of today's most widely used materials in the construction industry worldwide due to its strength in compression, ease of processing and flexibility of design. However, concrete members usually require reinforcement to compensate for their low tensile and shear strength. Steel has traditionally been the material of choice due to its mechanical performance and cost efficiency, resulting in reinforced concrete being a scalable material solution able to withstand a multitude of different load cases when designed correctly. In the event of additional live loads, design or execution flaws, or changes on the structure, structural strengthening may be the preferred solution.

This Expert Guide provides guidance for structural engineers when it comes to the structural strengthening of reinforced concrete members. The design requirements specified in ESR-4774 (ICC-ES Evaluation Report) are based on Acceptance Criteria AC125 from the International Code Council Evaluation Service (ICC-ES). These criteria cover the strengthening of concrete and both reinforced and unreinforced masonry using externally bonded fiber-reinforced polymer (FRP) composite systems, as detailed in Section 7.3 of AC125. This applies to the C-Fiber Force Strengthening System with externally bonded precured CFRP Laminates for applications such as out-of-plane flexural strengthening of masonry walls, concrete walls, slabs, and beams; shear strengthening of concrete beams; and axial strengthening of concrete columns under static or quasi-static loading scenarios.

This document contains the experimental and design, material specifications, design equations, installation instructions, reference to design tools for the licensed professional, and design examples for reference purposes.

fischer offers system solutions with the newly introduced C-Fiber Force Strengthening System using CFRP Laminates and CF Fabric. The design can be supported with the fischer REINFORCE-FIX Design Software, a new design tool in the well-known FiXperience Suite. With REINFORCE-FIX, structural engineers can verify rectangular beams, T-beams and slabs for flexural and shear strengthening with fischer CFRP Laminates, CF Fabrics and steel plates. The design can be performed according to ACI PRC-440.2-23.

We, together with our fischer national subsidiaries, are looking forward to your project inquiries.

Sarah Kleiner, M.Sc.
Engineering, Software & Training
Business Unit Structural Retrofitting

Hisham Keshta, M.Sc.
Engineering, Software & Training
Business Unit Structural Retrofitting

ICC-ES Evaluation Report (ESR-4774) Approval Holder Information

fischerwerke GmbH & Co. KG
Klaus-Fischer-Straße 1
72178 Waldachtal
Germany
www.fischer.group/en/

Content

1. Strengthening Solutions

New requirements arising from increased loads, stricter safety regulations, building repurposing, limited ground space in major cities, the restoration of historical structures, or damage-related causes – such as corrosion of structural elements, design and construction flaws, and seismic loads – can compromise structural stability, making strengthening necessary. Consider the example of reusing a residential apartment as an office space. For the new purpose an escalator is needed, thus a new opening must be added in the existing slab.

Furthermore, the live loads for office use are higher than those originally anticipated for residential use. Consequently, the utilization of the structural elements is exceeded. The solution is to strengthen the structure: the new opening for the added escalator is strengthened with CFRP Laminates, the columns are wrapped with CF Fabrics, the slabs and beams can be flexurally strengthened with CFRP Laminates or CF Fabrics. Concrete, steel reinforcement and CFRP each play distinct roles in load-bearing structures.

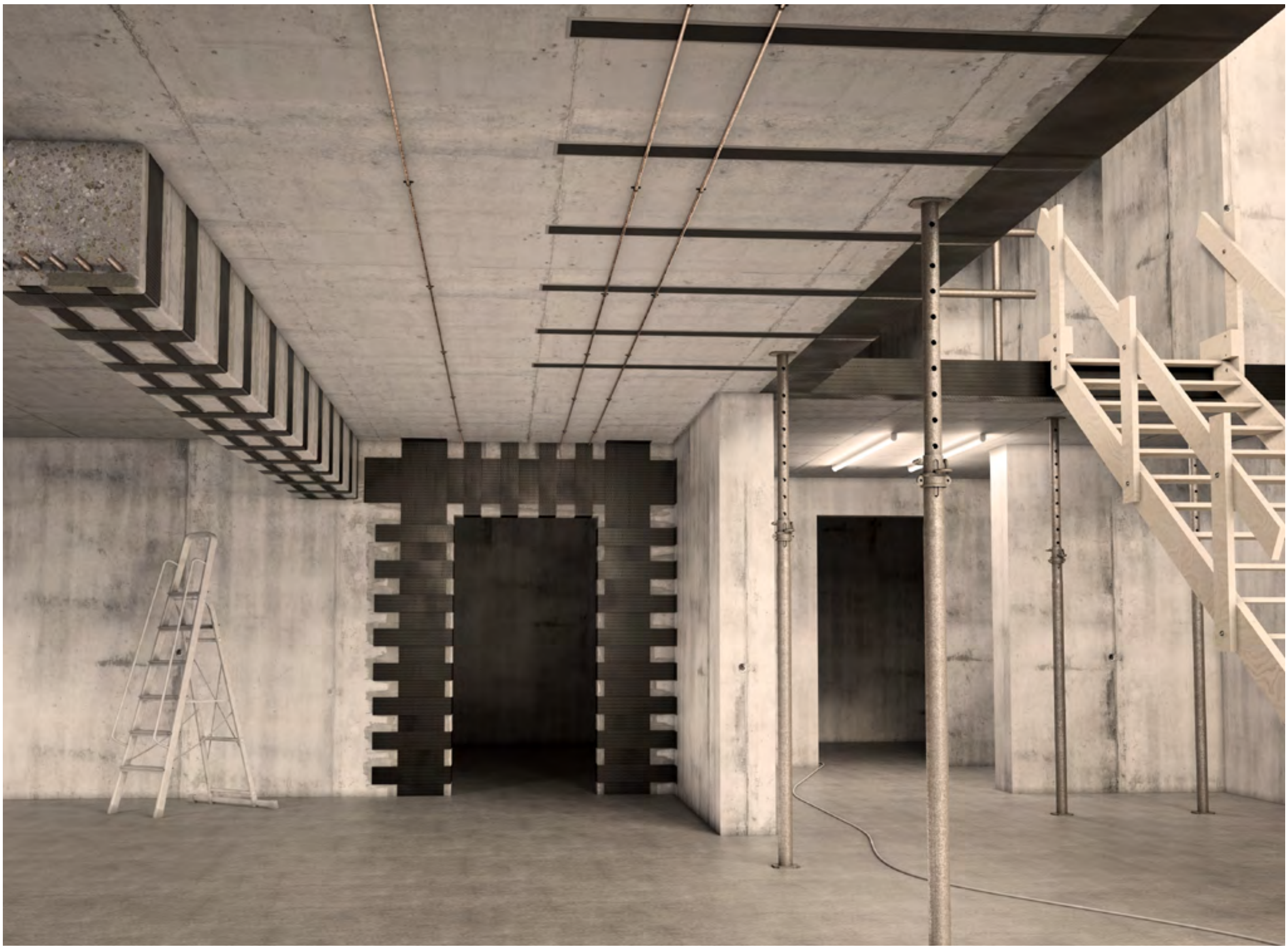


Fig. 1 – Building with flexural and shear strengthening

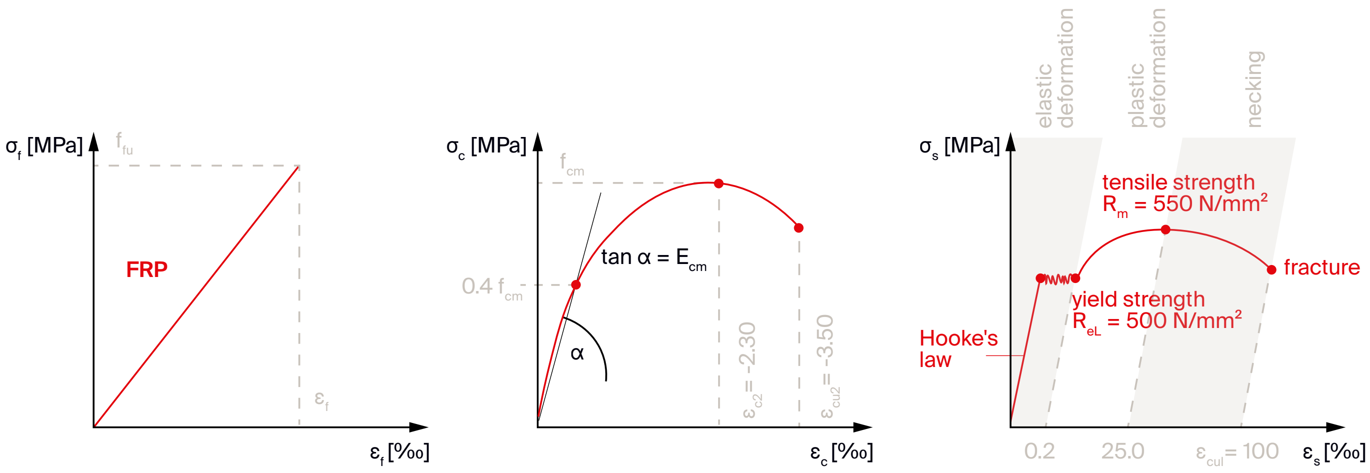


Fig. 2 – Stress-strain diagrams – left: CFRP, center: concrete, right: steel reinforcement

Steel reinforcement exhibits high initial stiffness, followed by significant ductility beyond its yield point, making it effective in absorbing energy and resisting tensile stresses in reinforced concrete. In contrast, concrete shows nonlinear stress-strain behavior; strong in compression, but weak in tension, requiring reinforcement to manage tensile stresses and prevent brittle failure. CFRP demonstrates high stiffness and strength with minimal to no plastic deformation, providing efficient load distribution and enhanced tensile capacity compared to steel. When used together, these materials form a composite system where concrete bears the compressive loads, steel reinforcement and CFRP handle tensile stresses, and the overall structural integrity is significantly enhanced.

CFRP primarily carries the additional load, enhancing the overall load-carrying capacity and stiffness of the structure. By reinforcing the steel with CFRP, the steel maintains its ductile behavior, even under increased loads. The CFRP strengthening, possessing a higher specific elastic modulus and distance from the neutral axis, develops higher strains and stresses under the same loading conditions, allowing it to bear significant loads and relieve stress on the steel reinforcement.

This combined approach leverages the strengths of both materials: CFRP enhances the load-carrying capacity and stiffness, while steel maintains ductility and prevents brittle failure. The integration of CFRP with steel reinforcement yields a balanced and resilient structural system, capable of sustaining higher loads, ensuring safety and enhancing durability.

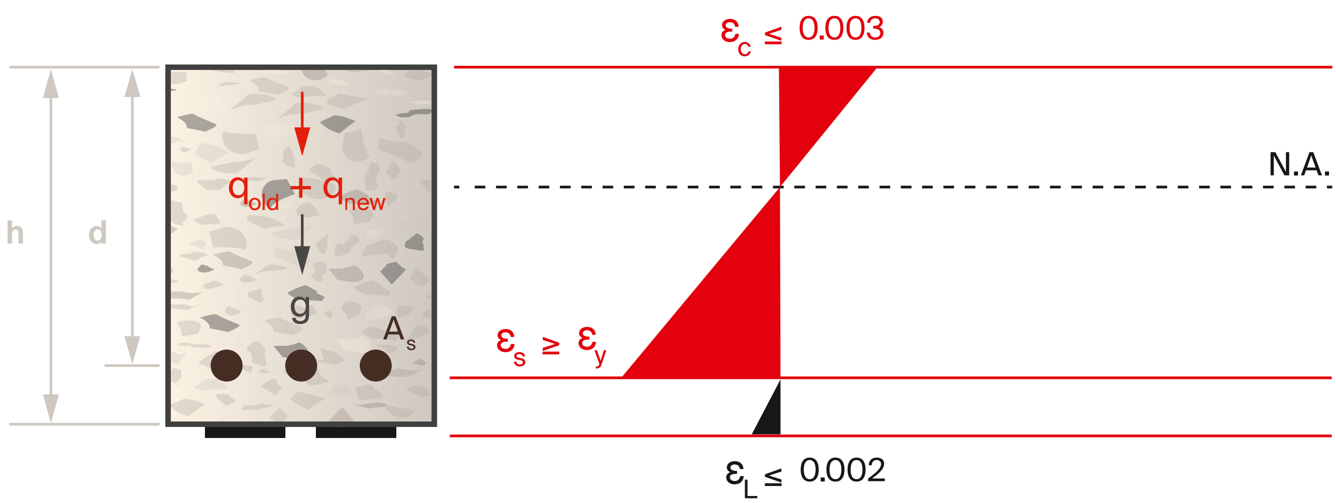


Fig. 3 - Distribution of strains for concrete, steel reinforcement and CFRP

1.1. Flexural strengthening

Rectangular and T-Beams, slabs and walls are structural elements typically subjected to flexural loads. These loads induce bending, which generates tensile stresses in the bottom and top area of the cross-section.

Fig. 4 – Left: unstrengthened beam, Right: flexural strengthened beam

In Fig. 4 on the left side, a simple beam is subjected to a uniformly distributed load. Such loading generates a parabolic bending moment, with compression resisted by concrete and tension resisted by steel reinforcement. If the acting load exceeds the capacity, these elements can be strengthened by carbon fiber laminates or fabrics, enhancing their tensile strength and overall performance.

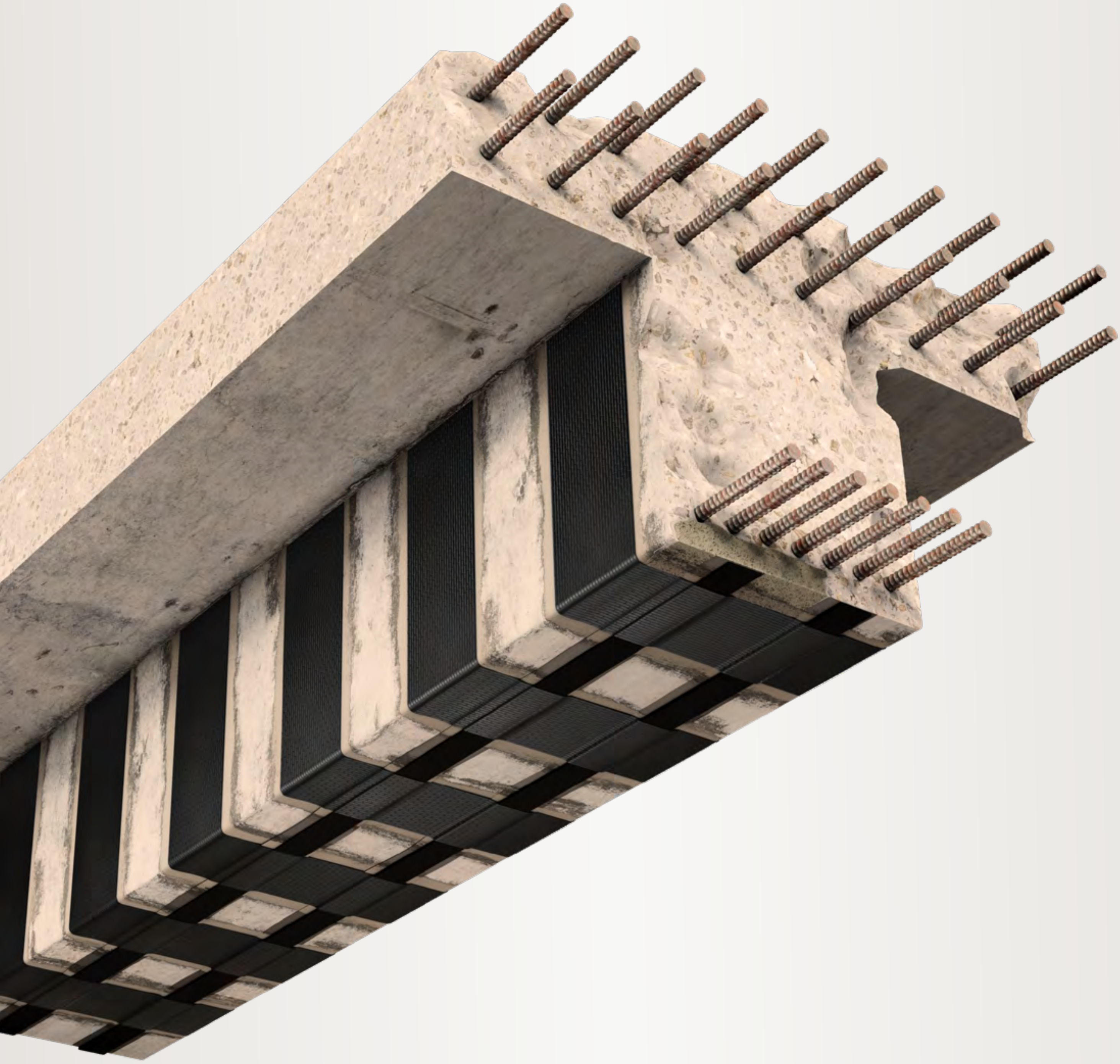
With a solid understanding of the behaviors of concrete, steel reinforcement and CFRP, we can appreciate how these materials work together to enhance structural performance. To address the needs of modern construction and infrastructure reinforcement, we offer a range of advanced products specifically designed for flexural strengthening. Our CFRP Laminates and carbon fiber fabrics are engineered to provide superior load-carrying capacity, increased stiffness, and enhanced durability. These solutions seamlessly integrate with existing structures, leveraging the high strength and stiffness of CFRP to complement the ductility of steel reinforcement, ensuring robust and resilient outcomes for various applications.

Our solutions for flexural strengthening with CFRP Laminate and Carbon Fiber Fabric

Carbon Fiber Reinforced Polymer (CFRP) Laminates
Externally bonded FRS-L-S & FRS-L-H CFRP Laminates
- 50 × 1.2 mm
- 75 × 1.2 mm
- 100 × 1.2 mm
- 50 × 1.4 mm
- 75 × 1.4 mm
- 100 × 1.4 mm

Carbon Fiber (CF) Fabric
CF Fabric FRS-W U300 & FRS-W U600
- 500 mm
- 200 mm

Further product information:
Catalogue Structural Strengthening



1.2. Shear strengthening

Rectangular beams, T-beams and walls are structural elements typically subjected to shear loads. These loads induce internal tensile forces that can cause diagonal cracking and potential failure along the plane of the cross-section. When a beam is uniformly loaded, shear forces peak at critical sections, typically near the supports.

In comparison, shear resistance in concrete can be understood as its ability to resist diagonal tension under load. Similar to axial jacketing, the strengthening is used for confining the critical regions, in this case near the supports, to enhance the shear capacity of the system, as seen in the different variations of applying CF Fabrics around a beam.

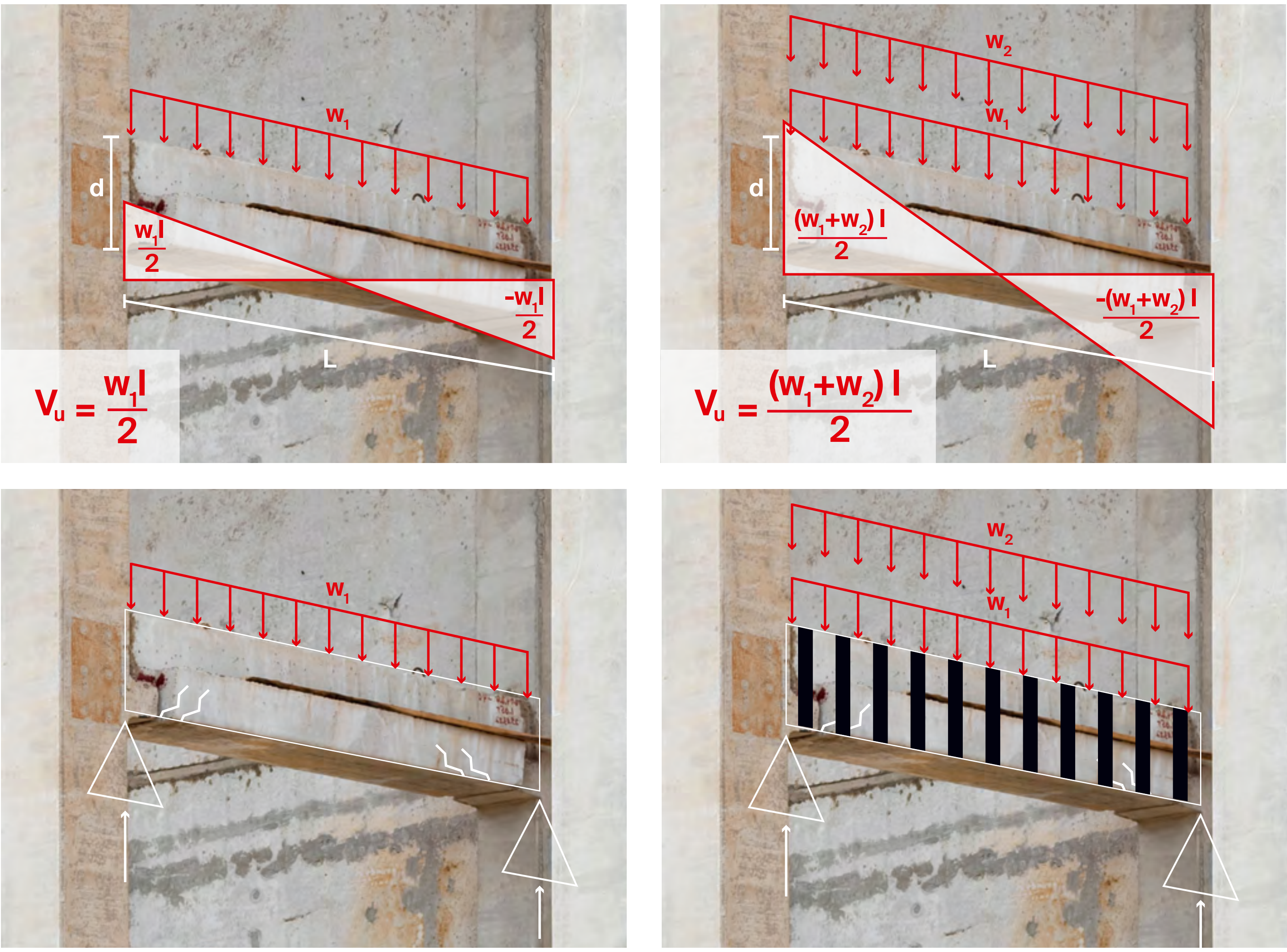


Fig. 5 – left: unstrengthened beam, right: shear strengthened beam

ACI offers different approaches for the application of CF Fabrics, such as completely wrapped, 3-sided U-wrap and 2-sided U-wrap. Furthermore, it can be installed in a declined orientation to effectively resist shear forces.

This alignment is particularly helpful in countering diagonal tensile forces within the structure. According to ACI, fischer offers the following CF Fabrics as shear strengthening options:

Our solution for shear strengthening

Carbon Fiber (CF) Fabric

CF Fabric FRS-W U300 & FRS-W U600
- 500 mm
- 200 mm

Further product information:
Catalogue Structural Strengthening

1.3. Axial strengthening

Axial loading refers to the application of forces along the longitudinal axis of a structural member, such as a column or a wall, causing compression or tension. Under significant axial compression, critical sections form with maximum compressive stresses at the center, gradually decreasing toward the edges.

These critical sections are often the points of failure, if the member is inadequately designed or constructed. Common reasons for failure under axial loading include insufficient reinforcement, leading to buckling or cracking and poor concrete quality, which reduces the member's capacity to withstand axial forces.

To mitigate these issues, effective axial strengthening methods are resembled in increasing confinement, such as FRP wraps for enhancing the structural capacity and preventing failure under axial loads.

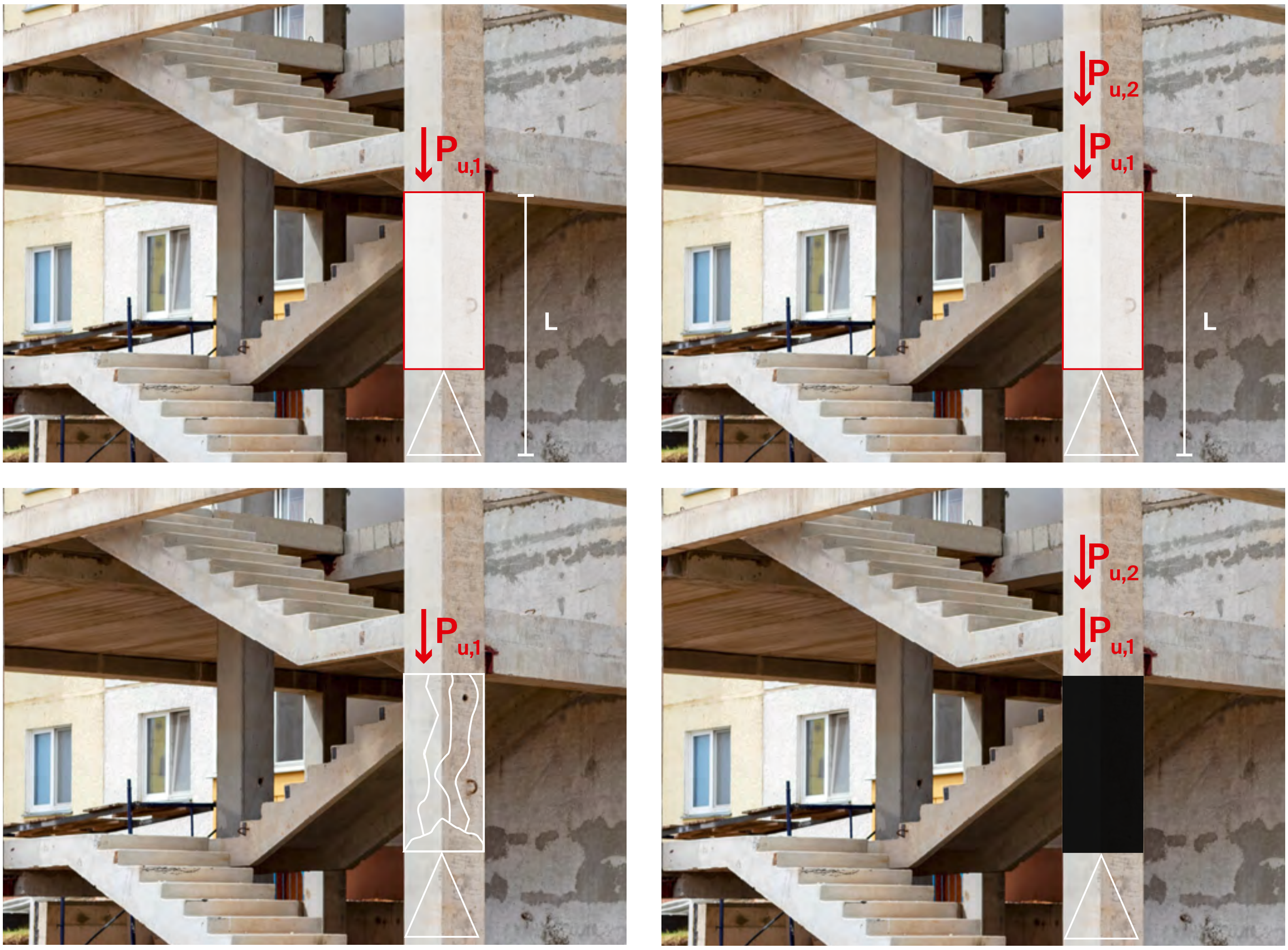


Fig. 6 – left: unstrengthened column, right: axially strengthened column

According to ACI, fischer offers CF Fabrics as shear strengthening options.

Our solution for axial strengthening

Carbon Fiber (CF) Fabric
CF Fabric FRS-W U300 & FRS-W U600
- 500 mm
- 200 mm

Further product information:
Catalogue Structural Strengthening

2. Products for Structural Strengthening

The fischer C-Fiber Force Strengthening System consists of two different strengthening solutions for the strengthening of concrete or masonry structures covered by the ESR-4774, ICC-ES Evaluation Report:

- externally bonded precured unidirectional CFRP Laminates consisting of two types of precured CFRP Laminates: the FRS-L-S CFRP Laminates with a standard elastic modulus and the FRS-L-H with a high tensile elastic modulus. Each offered in six different geometries, with thicknesses of 1.2 mm to 1.4 mm and widths of 50 mm, 75 mm and 100 mm. The precured CFRP Laminates are bonded to the concrete substrate using the two-component epoxy paste FRS-CS

or/and

- externally bonded unidirectional CF Fabrics of standard tensile strength and modulus with two area densities of 300 g/m^2 (FRS-W U300) and 600 g/m^2 (FRS-W U600) in two geometries each. The CF Fabrics are saturated using the FRS-CF two component epoxy saturating resin also used to prime the surface forming the CFRP composite in-situ via manual impregnation.

The C-Fiber Force Strengthening System contains a protective coating (interior finish) against flamespread and smoke development providing a class A interior finish rating for both systems in accordance with AC125 and ASTM E84.

Information regarding product geometries, packaging units, certificates and assessments, shelf life and technical parameters can be found in the corresponding product catalogue, technical datasheets and labels at *www.fischer-international.com*.

Installation instructions are given in the corresponding manufacturers product installation manuals deposited at ICC-ES, and can be downloaded from *www.fischer-international.com*. For installation training and for further technical support, please contact your local fischer Technical Teams in one of our 52 subsidiaries worldwide.

In this section, the products contained in and regulated by the ESR-4774 (ICC-ES Evaluation Report) are described briefly. For the detailed product information (item number, sales units, etc.) and for further technical data, please refer to the corresponding technical datasheets, catalogues and brochures available at *www.fischer-international.com*. For the structural design & calculations, the technical parameters provided in the ESR-4774 (ICC-ES Evaluation Report) are decisive.

2.1. Externally bonded CFRP Laminates FRS-L-S, FRS-L-H

The FRS-L-S and FRS-L-H CFRP Laminates are externally bonded to the concrete surface using the epoxy mortar FRS-CS. The FRS-L-S and FRS-L-H CFRP Laminate contribute to the load-bearing capacity of members, on which they are applied.

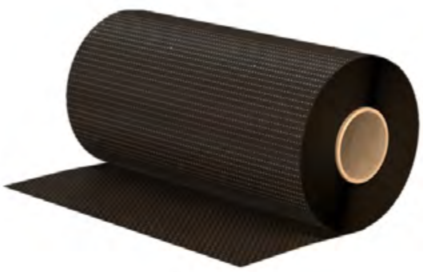
- Structural strengthening of reinforced concrete and pre-stressed concrete members
- Enhancing the tensile reinforcement area of existing concrete members
- Buildings, infrastructure and park decks
- Beams, concrete slabs, walls and ceiling openings
- The CFRP Laminates are available with the following cross-section geometries: 50 × 1.2 mm; 75 × 1.2 mm; 100 × 1.2 mm; 50 × 1.4 mm; 75 × 1.4 mm; 100 × 1.4 mm

2.2. Epoxy mortar FRS-CS

The FRS-CS is a two-component, epoxy-based, thixotropic structural bonding agent (epoxy paste) for the installation of the FRS-L-S and FRS-L-H externally bonded CFRP Laminates.

Available as a can system with 5 kg or 10 kg filling capacity, or as an injection cartridge with 585 ml filling volume.

2.3. Carbon fiber fabric FRS-W U300 and FRS-W U600



The FRS-W U300 and FRS-W U600 are unidirectional, high-strength carbon fiber fabrics for structural strengthening purposes. The FRS-W carbon fiber fabric contributes to the load-bearing capacity of members.

Available with 300 g/m² and 600 g/m² area densities.

2

2.4. Saturating resin FRS-CF

The FRS-CF is a two-component, epoxy based saturating resin for the impregnation and bonding of FRS-W U300 and FRS-W U600 carbon fiber fabrics.

- Available as a can system with 5 kg or 10 kg filling capacity
- Suitable for dry-layup or wet-layup application techniques

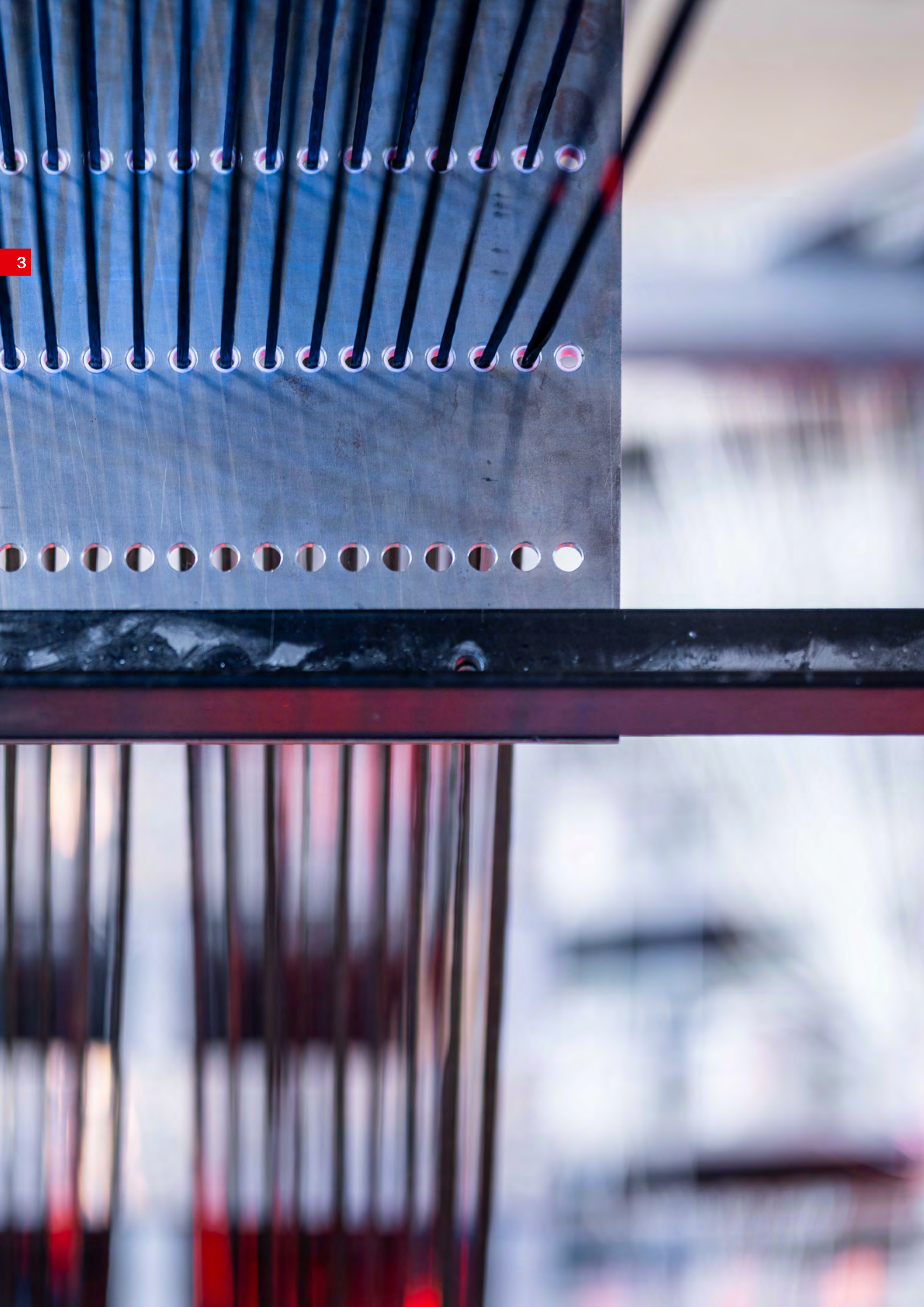
2.5. Fire protection coating FRS-FP

The FRS-FP is a one-component, water-based, intumescent coating (interior finish) to be applied on the precured FRS-L-S and FRS-L-H carbon fiber laminates and/or on the FRS-W carbon fiber fabrics after installation to prevent smoke development in case of a fire event.

2.6. Complementary products

There are several other products that complement the C-Fiber Force Strengthening System. While these products are not included in the ICC approval scope, they are nonetheless highly beneficial for a comprehensive application.

Examples include the FRS-FC Spike Anchor for end anchorage of carbon fiber fabrics, FRS-PC 11 Epoxy repair mortar for concrete repairs before the application of strengthening or FRS-CP Corrosion Protection Coating for shear strengthening with steel plates.



3

3. Material Parameters

This chapter gives a detailed overview about the material parameters contained in the ESR-4774. The experimental material parameters can be found in chapter 3.1. Chapter 3.2 shows the relevant parameters for the structural design.

3.1. Experimental material parameters

Tab. 1 – FRS-W U300 composites

Property	Test norm	Performance	Unit
Ultimate tensile strength	ASTM D3039	1061 (154)	MPa (ksi)
Tensile modulus	ASTM D3039	83 (12.0)	GPa (Msi)
Ultimate tensile strain	ASTM D3039	1.28	%
Glass transition temperature (loss modulus)	ASTM E1640	67 (153)	°C (°F)
Coefficient of thermal expansion	ASTM E831	5.1 (0° direction) 71.8 (90° direction)	10^{-6} K^{-1} 10^{-6} K^{-1}
Void content	ASTM D3171	3.5	%
Interlaminar shear	ASTM D2344	31.4 (4.6)	MPa (ksi)
Creep	ASTM D2990	No failure	failure / no failure

Nominal composite thickness: 0.51 mm per ply.

Tab. 2 – FRS-W U600 composites

Property	Test norm	Performance	Unit
Ultimate tensile strength	ASTM D3039	967 (140)	MPa (ksi)
Tensile modulus	ASTM D3039	77 (11.2)	GPa (Msi)
Ultimate tensile strain	ASTM D3039	1.26	%
Glass transition temperature (loss modulus)	ASTM E1640	67 (153)	°C (°F)
Coefficient of thermal expansion	ASTM E831	5.0 (0° direction) 66.9 (90° direction)	10^{-6} K^{-1} 10^{-6} K^{-1}
Void content	ASTM D3171	4.7	%
Interlaminar shear	ASTM D2344	35.6 (5.2)	MPa (ksi)
Creep	ASTM D2990	No failure	failure / no failure

Nominal composite thickness: 1.02 mm per ply.

3

Tab. 3 – FRS-L-S externally bonded precured CFRP Laminates

Property	Test norm	Performance	Unit
Ultimate tensile strength	ASTM D3039	2969 (431)	MPa (ksi)
Tensile modulus	ASTM D3039	168 (24.4)	GPa (Msi)
Ultimate tensile strain	ASTM D3039	1.77	%
Glass transition temperature (loss modulus)	ASTM E1640	111	°C (°F)
Coefficient of thermal expansion	ASTM E831	-0.6 (0° direction) 28.4 (90° direction)	10^{-6} K^{-1} 10^{-6} K^{-1}
Void content	ASTM D3171	3.9	%
Creep	ASTM D2990	No failure	failure / no failure

Laminate thickness: 1.2 mm or 1.4 mm.

Tab. 4 – FRS-L-H externally bonded precured CFRP Laminates

Property	Test norm	Performance	Unit
Ultimate tensile strength	ASTM D3039	3318 (481)	MPa (ksi)
Tensile modulus	ASTM D3039	198 (28.7)	GPa (Msi)
Ultimate tensile strain	ASTM D3039	1.68	%
Glass transition temperature (loss modulus)	ASTM E1640	111	°C (°F)
Coefficient of thermal expansion	ASTM E831	-0.3 (0° direction) 29.3 (90° direction)	10^{-6} K^{-1} 10^{-6} K^{-1}
Void content	ASTM D3171	3.9	%
Creep	ASTM D2990	No failure	failure / no failure

Laminate thickness: 1.2 mm or 1.4 mm.

3.2. Material design parameters

Tab. 5 – FRS-W U300 composites

Property	Test norm	Performance	Unit
Guaranteed tensile strength	ASTM D3039	918 (133)	MPa (ksi)
Tensile modulus	ASTM D3039	83 (12.0)	GPa (Msi)
Guaranteed tensile strain	ASTM D3039	1.07	%
Guaranteed fracture force per unit width	ASTM D3039	0.47	kN/mm

Tab. 6 – FRS-W U600 composites

Property	Test norm	Performance	Unit
Guaranteed tensile strength	ASTM D3039	794 (115)	MPa (ksi)
Tensile modulus	ASTM D3039	77 (11.2)	GPa (Msi)
Guaranteed tensile strain	ASTM D3039	1.02	%
Guaranteed fracture force per unit width	ASTM D3039	0.81	kN/mm

Tab. 7 – FRS-L-S externally bonded precured CFRP Laminates

Property	Test norm	Performance	Unit
Guaranteed tensile strength	ASTM D3039	2585 (375)	MPa (ksi)
Tensile modulus	ASTM D3039	168 (24.4)	GPa (Msi)
Guaranteed tensile strain	ASTM D3039	1.47	%
Guaranteed fracture force per unit width	ASTM D3039	3.10 3.62	kN/mm kN/mm

Tab. 8 – FRS-L-H externally bonded precured CFRP Laminates

Property	Test norm	Performance	Unit
Guaranteed tensile strength	ASTM D3039	2808 (407)	MPa (ksi)
Tensile modulus	ASTM D3039	198 (28.7)	GPa (Msi)
Guaranteed tensile strain	ASTM D3039	1.28	%
Guaranteed fracture force per unit width	ASTM D3039	3.37 3.93	kN/mm kN/mm

3.3. Reduction factors

Tab. 9 – Environmental reduction factor C_E

Environment	FRS-W U300	FRS-W U600	FRS-L-S	FRS-L-H
Interior exposure	0.95			
Exterior exposure	0.85			
Aggressive environment	0.85			

Tab. 10 – Sustained load and fatigue load reduction

	FRS-W U300	FRS-W U600	FRS-L-S	FRS-L-H
Sustained plus cyclic stress limit	0.55 f_{fu}			

4. Design Approach according to ACI PRC-440.2-23

The design approach described in this document is based on Section 7.3.2 of ICC AC125.
The design equations are based on the provisions of ACI PRC-440.2-23.

4.1. Notations

A_c = cross-sectional area of concrete in compression member, in.^{2} (mm^2)

A_e = cross-sectional area of effectively confined concrete section, in.^{2} (mm^2)

A_f = $Nt_f w_f$, area of FRP external reinforcement, in.^{2} (mm^2)

A_{fv} = area of FRP shear reinforcement with spacing sf, in.^{2} (mm^2)

A_s = area of nonprestressed steel reinforcement, in.^{2} (mm^2)

b = width of a rectangular cross section, in. (mm)

b_w = web width or effective diameter of circular section, in. (mm)

c = distance from extreme compression fiber to the neutral axis, in. (mm)

C_E = environmental reduction factor

d = distance from extreme compression fiber to the centroid of the nonprestressed steel tension reinforcement, in. (mm)

D = diameter of compression member for circular cross sections or diagonal distance equal to for prismatic cross section (diameter of equivalent circular column), in. (mm)

d_{fv} = effective depth of FRP shear reinforcement, in. (mm)

d_f = distance from extreme compression fiber to the centroid of the FRP laminate on the tension side of the flexural member, in. (mm)

E_c = modulus of elasticity of concrete, psi (MPa)

E_f = tensile modulus of elasticity of FRP, psi (MPa)

E_s = modulus of elasticity of steel, psi (MPa)

f_c = compressive stress in concrete, psi (MPa)

f'_c = specified compressive strength of concrete, psi (MPa)

f'_{cc} = compressive strength of confined concrete, psi (MPa)

f_{fe} = effective stress in the FRP, psi (MPa)

f_{fu} = design ultimate tensile strength of FRP = $C_E f_{fu}^*$, psi (MPa)

f_{fu}^* = ultimate tensile strength of the FRP material as reported by the manufacturer, psi (MPa)

f_l = maximum confining pressure due to FRP jacket, psi (MPa)

f_s = stress in nonprestressed steel reinforcement, psi (MPa)

f_y = specified yield strength of nonprestressed steel reinforcement, psi (MPa)

h = overall thickness of a member, in. (mm)

k = ratio of depth of neutral axis to reinforcement depth measured from extreme compression

k_1 = modification factor applied to k_v to account for concrete strength

k_2 = modification factor applied to k_v to account for wrapping scheme

L_e	=	active bond length of FRP laminate, in. (mm)
M_n	=	nominal moment strength, in.-lb (Nmm)
M_u	=	factored moment at section, in.-lb (Nmm)
N	=	number of plies of FRP reinforcement
r_c	=	radius of edges of a prismatic cross section confined with FRP, in. (mm)
s_f	=	center-to-center spacing of FRP strips, in. (mm)
t_f	=	nominal thickness of one ply of the FRP reinforcement, in. (mm)
V_c	=	nominal shear strength provided by concrete, lb (N)
V_f	=	nominal shear strength provided by FRP, lb (N)
V_n	=	nominal shear strength, lb (N)
V_s	=	nominal shear strength provided by steel stirrups, lb (N)
V_u	=	factored shear force at section, lb (N)
w_f	=	width of the FRP reinforcing plies, in. (mm)
α_1	=	multiplier on f_c' to determine intensity of an equivalent rectangular stress distribution for concrete
β_1	=	ratio of the depth of the equivalent rectangular stress block to the depth to the neutral axis
ε_{bi}	=	strain level in the concrete substrate at the time of the FRP installation (tension is positive), in./in. (mm/mm)
ε_c	=	strain level in the concrete, in./in. (mm/mm)
ε_{cu}	=	ultimate axial strain of unconfined concrete corresponding to $0.85f''_c$ or maximum usable strain of unconfined concrete, in./in. (mm/mm), which can occur at $f_c = 0.85f''_c$ or $\varepsilon_c = 0.003$, depending on the obtained stress-strain curve
ε_{ccu}	=	ultimate axial compressive strain of confined concrete corresponding to $0.85f'_{cc}$ in a lightly confined member (member confined to restore its concrete design compressive strength), or ultimate axial compressive strain of confined concrete corresponding to failure in a heavily confined member
ε_f	=	strain level in the FRP reinforcement at designated strength, in./in. (mm/mm)
ε_{fd}	=	design rupture strain of FRP reinforcement, in./in. (mm/mm)
ε_{fe}	=	effective strain in FRP reinforcement attained at failure, in./in. (mm/mm)
ε_{fu}	=	design rupture strain of FRP reinforcement = $C_E e_{fu}^*$, in./in. (mm/mm)
ε_{fu}^*	=	ultimate rupture strain of FRP reinforcement, in./in. (mm/mm)
ε_s	=	strain level in the nonprestressed steel reinforcement, in./in. (mm/mm)
ε_{sy}	=	strain corresponding to the yield strength of nonprestressed steel reinforcement, in./in. (mm/mm)
ϕ	=	strength-reduction factor
ψ_f	=	additional FRP strength-reduction factor

4.2. Flexural strengthening of reinforced concrete members

CF Fabrics and CFRP Laminates can be used to enhance the flexural strength of existing reinforced concrete elements such as beams, joists, and slabs. They are effective for both positive and negative moment strengthening.
The basic design requirement is, $\phi Mn \geq Mu$

The section analysis is performed under the following assumptions:

1. Plane sections remain plane after loading
2. The bond between FRP and substrate remains perfect
3. The maximum usable compressive strain in the concrete is 0.003
4. FRP has a linear-elastic behavior to failure

The FRP is typically placed at the location of tension force caused by the flexural demand with the primary fibers oriented along the member axis. Per ACI PRC-440.2-23, flexural strengthening with FRP is considered a bond-critical application in accordance with Section 1.2.1.4 and the surface should be prepared in accordance with Section 6.4.2.1. The section analysis of an element being considered for flexural enhancement assumes that the strains in the concrete, steel reinforcement and FRP are proportional to their distance from the neutral axis, as shown in Fig. 7.

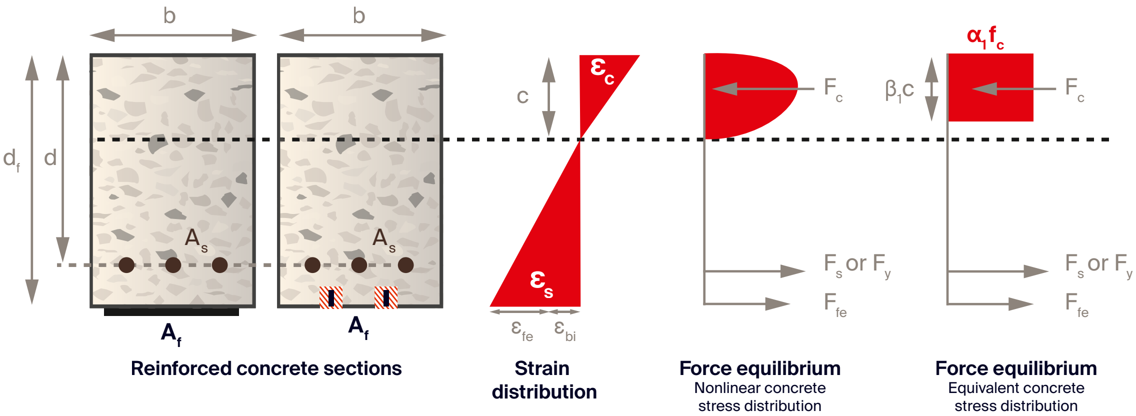


Fig. 7 – Internal stress strain distribution for a singly-reinforced rectangular section (reference ACI PRC-440.2-23, Figure 10.2.10)

Per ACI PRC 440.2 Section 10.1, when designing flexural enhancement with FRP, all applicable failure modes should be considered. These include:

1. Crushing of concrete before yielding of reinforcing steel: This failure mode typically occurs when the section is over-reinforced and the concrete compressive strain, ε_c, reaches the limit ε_{cu} (defined as 0.003 in ACI PRC-440.2-23) before tensile strain in the reinforcement, ε_s, reaches yield strain, ε_{sy}.

2. Yielding of the reinforcing steel followed by rupture of the FRP: This failure mode typically occurs when the section is under-reinforced. At the ultimate capacity, the concrete compressive strain, ε_c, is below the limit ε_{cu}, the tensile strain in the reinforcement, ε_s, exceeds the yield strain, ε_{sy}, and the tensile strain in the FRP reaches the design rupture strain ε_{fu} (per section 9.4 of ACI PRC-440.2-23).

3. Yielding of the reinforcing steel followed by debonding of the FRP: This failure mode typically also occurs when the section is under-reinforced. At the ultimate moment capacity, the concrete compressive strain, ε_c, is below the limit ε_{cu}, the tensile strain in the reinforcement, ε_s, exceeds the yield strain, ε_{sy}, and the tensile strain in the FRP reaches the debonding strain ε_{fd} (per Eq. 10.1.1 of ACI PRC-440.2-23).

4. Yielding of the reinforcing steel followed by crushing of the concrete: This failure mode typically also occurs when the section is under-reinforced. At the ultimate moment capacity, the concrete compressive strain, ε_c, reaches the limit ε_{cu}, the tensile strain in the reinforcement, ε_s, exceeds the yield strain, ε_{sy}, and the tensile strain in the FRP is below the debonding strain ε_{fd} (per Eq. 10.1.1 of ACI PRC-440.2-23).

Flexural enhancement using externally bonded FRP is based on Chapter 10 of ACI PRC 440.2 and Section 7.3.2.1 of AC125. The major steps in the design process are described below, with references to equations and sections in ACI PRC 440.2.

a. Determine the strain in FRP, ε_{fd}, at which debonding may occur. This strain is the maximum permissible strain in the FRP and can be determined as, — Eq. 10.1.1

$$\varepsilon_{fd} = 0.41\sqrt{\frac{f'_c}{NE_f t_f}} \leq 0.9\varepsilon_{fu}$$

b. The effective strain in the FRP, ε_{fe}, is calculated using the strain profiles, as shown in Fig. 7, and further reduced by the initial strain in the substrate, ε_{bi}. — Eq. 10.2.5

$$\varepsilon_{fe} = \varepsilon_{cu}\left(\frac{d_f - c}{c}\right) - \varepsilon_{bi} \leq \varepsilon_{fd}$$

c. The governing failure mode is determined by the strains in the different components of the section. To calculate the strain profile in the section, an initial assumption is made for the depth of the neutral axis, c, from the top of the section. Using this initial assumption, strains can be calculated for each component using their distance from the neutral axis. — Eq. 10.2.10a

The strain at the extreme compression fiber, ε_c, when the FRP reaches a strain of ε_{fe} is given by rearranging Eq. 10.2.5.

$$\varepsilon_c = (\varepsilon_{fe} + \varepsilon_{bi})\left(\frac{c}{d_f - c}\right)$$

If $\varepsilon_c \geq \varepsilon_{cu}$, concrete crushing will govern.
The strain in the reinforcing steel, ε_s, can be calculated as,

$$\varepsilon_s = (\varepsilon_{fe} + \varepsilon_{bi})\left(\frac{d - c}{d_f - c}\right)$$

If $\varepsilon_s \geq \varepsilon_{sy}$, the reinforcing steel will yield.

d. Using the strains calculated above, the stresses and corresponding forces in the different components can be calculated to check internal equilibrium.

$$\alpha_1 f'_c \beta_1 bc = A_s f_s + A_f f_{fe}$$

Eq. 10.2.10c

$$\alpha_1 = \frac{3\varepsilon'_c\varepsilon_c - \varepsilon_c^2}{3\beta_1\varepsilon_c'^2}$$

Eq. 10.2.6

$$\beta_1 = \frac{4\varepsilon'_c - \varepsilon_c}{6\varepsilon'_c - 2\varepsilon_c}$$

$$f_{fe} = E_f\varepsilon_{fe}$$

$$f_s = E_s\,\varepsilon_s$$

If equilibrium is not satisfied, the depth of the neutral axis is revised. This iterative calculation is performed till internal equilibrium is satisfied.

e. At equilibrium, the flexural strength of the strengthened section can be determined as, — Eq. 10.2.10d

$$M_n = A_s f_s\left(d - \frac{\beta_1 c}{2}\right) + \psi_f A_f f_{fe}\left(d_f - \frac{\beta_1 c}{2}\right)$$

where, ψ_f =0.85

f. The strengthening is adequate when, — Eq. 10.2.10d

$$\phi Mn \geq Mu$$

Where, ϕ is calculated per Eq. 10.2.7.

4

g. ACI PRC 440.2, Section 10.2.8 requires that the section must be checked for serviceability considerations to ensure that the member will not undergo inelastic deformations at service level loads.

The stress in steel reinforcing at service level loads is,

$$f_{s,s} = \frac{\left[M_s + \varepsilon_{bi} A_f E_f \left(d_f - {}^{kd}/_{3}\right)\right](d-kd)E_s}{\left[A_s E_s \left(d - {}^{kd}/_{3}\right)(d-kd) + A_f E_f \left(d_f - {}^{kd}/_{3}\right)(d_f - kd)\right]}$$

Eq. 10.2.10.1

The stress in the FRP at service level loads is,

$$f_{f,s} = f_{s,s}\left(\frac{E_f}{E_s}\right)\frac{d_f - kd}{d - kd} - \varepsilon_{bi} E_f \leq 0.80 f_y$$

Eq. 10.2.10.2

$f_{f,s}$ should be limited to the creep and fatigue limits in Section 10.2.9.

The stress at the extreme compression fiber can be calculated as,

$$f_{c,s} = {}^{C}/_{(b\ x\ kd)/2} = \frac{2C}{b\ x\ kd} \leq 0.60 f'_c$$

Where, the total compression in the section is,

$$C = f_{s,s} A_s + f_{f,s} A_f$$

h. ACI PRC 440.2 Section 9.2 places some strengthening limits to ensure that a member strengthened with FRP will not collapse, should the FRP or its bond to the concrete be damaged due to vandalism or fire. These limits should be checked carefully to determine the eligibility of a member for FRP strengthening.

4.3. Shear strengthening of reinforced concrete members

Fischer CF Fabrics can be used to enhance the shear strength of existing reinforced concrete elements such as beams. The FRP is placed such that the primary fibers are either transverse or at an angle, α, to the member axis. Shear strengthening with FRP is considered a bond-critical application in accordance with Section 1.2.1.4 and the surface should be prepared in accordance with Section 6.4.2. A typical T-beam strengthening scheme is shown in Fig. 8.

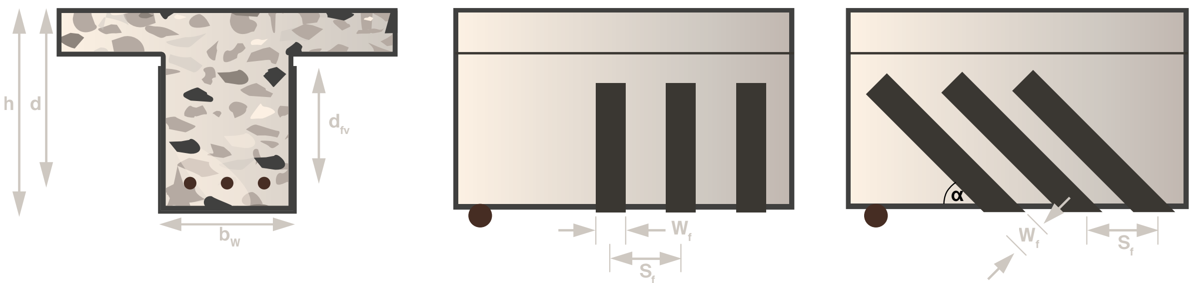


Fig. 8 – Typical FRP orientation for U-wraps for shear strengthening of a T-beam (reference ACI PRC-440.2-23, Figure 11.4)

The basic design requirement is,

$$\phi Vn \geq Vu$$

where,

$$\phi V_n = \phi\left(V_c + V_s + \psi_f V_f\right)$$

Where, ψ_f = 0.95 for completely wrapped members; 0.85 for three or two opposite-sided schemes.

Shear enhancement using FRP is based on Chapter 11 of ACI PRC 440.2 and Section 7.3.2.6 of AC125. The major steps in the design process are described hereinafter, with references to equations and sections in ACI PRC 440.2.

a. Determine the effective strain, ε_{fe}, and corresponding stress in FRP, f_{fe}.
For completely wrapped members,

$$\varepsilon_{fe} = 0.004 \leq 0.75\varepsilon_{fu}$$ Eq. 11.4.1.1

For U-wraps or FRP bonded on opposite faces,

$$\varepsilon_{fe} = \kappa_v \varepsilon_{fu} \leq 0.004$$ Eq. 11.4.1.2a

$$\kappa_v = \frac{k_1 k_2 L_e}{11{,}900\varepsilon_{fu}} \leq 0.75\ (SI)$$

$$\kappa_v = \frac{k_1 k_2 L_e}{468\varepsilon_{fu}} \leq 0.75\ (in-lb)$$ Eq. 11.4.1.2b

$$L_e = \frac{23{,}300}{(Nt_f E_f)^{0.58}}\ (SI)$$

$$L_e = \frac{2500}{(Nt_f E_f)^{0.58}}\ (in-lb)$$ Eq. 11.4.1.2c

$$k_1 = \left(\frac{f'_c}{27}\right)^{\frac{2}{3}}\ (SI)$$

$$k_1 = \left(\frac{f'_c}{4000}\right)^{\frac{2}{3}}\ (in-lb)$$ Eq. 11.4.1.2d

$$k_2 = \frac{d_{fv} - L_e}{d_{fv}}\ (U-wraps)$$

$$k_2 = \frac{d_{fv} - 2L_e}{d_{fv}}\ (FRP\ on\ opposite\ faces)$$ Eq. 11.4.1.2e

$$f_{fe} = E_f \varepsilon_{fe}$$ Eq. 11.4d

b. Calculate the shear strength contribution of the FRP as,

$$V_f = \frac{A_{fv} f_{fe} (sin\alpha + cos\alpha) d_{fv}}{s_f}$$ Eq. 11.4a

Where A_{fv} is calculated as,

$$A_{fv} = 2Nt_f w_f\ \ (Rectangular\ sections)$$ Eq. 11.4b

$$A_{fv} = (\pi/2)Nt_f w_f\ \ (Circular\ sections)$$ Eq. 11.4c

c. Check that the total shear strength provided by the existing steel reinforcement and the external FRP reinforcement does not exceed Eq. 11.4.3

$$V_s + V_f \leq 0.66\sqrt{f'_c} b_w d\ \ (SI)$$

$$V_s + V_f \leq 0.66\sqrt{f'_c} b_w d\ \ (in-lb)$$

For circular sections $b_w d = 0.8D^2$, where D is the diameter.

d. The strengthening is adequate when, Eq. 10.2.10d

$$\phi Vn \geq Vu$$

Where, ϕ is taken from the standard based on which the strengthening is being performed.

e. ACI PRC 440.2 Section 9.2 places some strengthening limits to ensure that a member strengthened with FRP will not collapse, should the FRP or its bond to the concrete be damaged due to vandalism or fire. These limits should be checked carefully to determine the eligibility of a member for FRP strengthening.

4.4. Strengthening of members subjected to pure axial compression

4

fischer CF Fabrics can be used to enhance the axial compression strength of existing non-slender circular and rectangular reinforced concrete columns. The FRP is placed such that the primary fibers are transverse to the member axis. Confinement of a column with FRP can also result in an increase in the ultimate strain that the section can resist in compression, which can lead to an enhancement of ductility. Confinement of a section with FRP is considered a contact-critical application in accordance with Section 1.2.1.4 and the surface should be prepared in accordance with Section 6.4.2.2.

The FRP jacket provides a passive confinement to the section and stresses are developed in the FRP due to lateral expansion (also called dilation) of the section under compression. At factored loads, the concrete may suffer significant cracking in the radial direction. As such, confinement with FRP to increase axial compressive strength should be used to address overloading conditions that are temporary in nature, such as during a seismic event. See Section 12.1.3 for more discussion on serviceability considerations.

The basic design requirement is,

$\phi Pn \geq Pu$

For columns with existing transverse steel-tie reinforcement,

$$\phi P_n = 0.8\,\phi\left[0.85f'_c\left(A_g - A_{st}\right) + f_y A_{st}\right]$$ Eq. 12.1a

For columns with existing transverse steel spiral reinforcement,

$$\phi P_n = 0.85\,\phi\left[0.85f'_c\left(A_g - A_{st}\right) + f_y A_{st}\right]$$ Eq. 12.1b

ψ_f = 0.95 for completely wrapped members

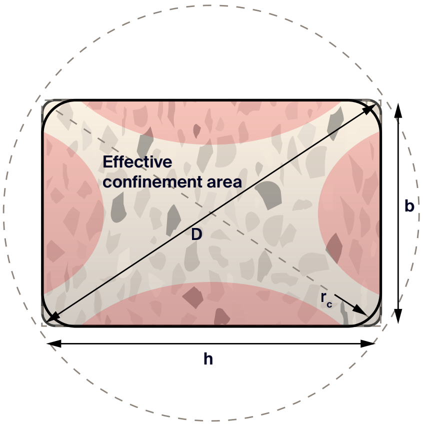


Fig. 9 – Typical rectangular section with FRP confinement (reference ACI PRC-440.2-23, Figure 12.1.2)

For confined columns, f'_c in the above equations can be substituted with f'_{cc}, to compute the axial compressive load carrying capacity of the confined column. For rectangular sections, ACI PRC 440.2 Section 12.1.2 limits axial load capacity enhancement for sections with an aspect ratio h/b less than or equal to 2.0 and face dimensions less than or equal to 36 in (900 mm). Fig. 9 shows a rectangular column with rounded columns and FRP confinement.

Pure axial compressive load capacity enhancement using FRP is based on Chapter 12 of ACI PRC 440.2 and Section 7.3.2.3 of AC125. The major steps in the design process are described below, with references to equations and sections in ACI PRC 440.2.

a. Determine the effective strain, ε_{fe}.

$\varepsilon_{fe} = \kappa_\varepsilon \varepsilon_{fu}$ Eq. 12.1i

$\varepsilon_{fe} = 0.55\varepsilon_{fu}$ Section 12.1

b. Calculate the maximum confinement pressure due to the FRP confinement. Eq. 12.1h

$$f_l = \frac{2E_f N t_f \varepsilon_{fe}}{D}$$ Section 12.1

The confinement ratio, f_l/f'_c, shall equal or exceed 0.08. Eq. 12.1.2a

For rectangular sections, the equivalent diameter is,

$D = \sqrt{b^2 + h^2}$

c. Calculate the maximum confined compressive strength.

$f'_{cc} = f'_c + \psi_f 3.3\kappa_a f_l$ Eq. 12.1g

where,

$\kappa_a = 1.0\ (Circular\ sections)$

c. $\kappa_a = \frac{A_e}{A_c}\left(\frac{b}{h}\right)^2 \quad (Rectangular\ sections)$ Eq. 12.1.2b

$$\frac{A_e}{A_c} = \frac{1 - \frac{\left[\left(\frac{b}{h}\right)(h-2r_c)^2 + \left(\frac{h}{b}\right)(b-2r_c)^2\right]}{3A_g} - \rho_g}{1-\rho_g}$$ Eq. 12.1.2d

d. The section's axial compressive load-carrying capacity is determined based on the confined concrete's compressive strength.
For columns with existing steel-tie reinforcement,

$$\phi P_n = 0.8\,\phi\left[0.85 f'_{cc}\left(A_g - A_{st}\right) + f_y A_{st}\right]$$ Eq. 12.1a

For columns with existing steel spiral reinforcement,

$$\phi P_n = 0.85\,\phi\left[0.85 f'_{cc}\left(A_g - A_{st}\right) + f_y A_{st}\right]$$ Eq. 12.1b

ψ_f = 0.95 for completely wrapped members

e. In addition to the compressive strength of the concrete, confinement with FRP can also enhance the maximum strain, ε_{ccu} and the section can resist in compression.

$$\varepsilon_{ccu} = \varepsilon'_c\left(1.50 + 12\kappa_b \frac{f_l}{f'_c}\left(\frac{\varepsilon_{fe}}{\varepsilon'_c}\right)^{0.45}\right)$$ Eq. 12.1j

$$\varepsilon'_c = \frac{1.7 f'_c}{E_c}$$

$$\kappa_b = \frac{A_e}{A_c}\left(\frac{h}{b}\right)^2$$ Eq. 12.1.2c

f. The strengthening is adequate when, Eq. 10.2.10d

$$\phi Pn \geq Pu$$

Where, ϕ is taken from the standard based on which the strengthening is being performed for the appropriate existing transverse reinforcement.

g. ACI PRC 440.2 Section 9.2 places some strengthening limits to ensure that a member strengthened with FRP will not collapse, should the FRP or its bond to the concrete be damaged due to vandalism or fire. These limits should be checked carefully to determine the eligibility of a member for FRP strengthening. When axial compressive load capacity enhancement is used solely for temporary loading such as seismic loads, the fire resistance check is not required per Section 9.2 and Chapter 13.

Confinement with FRP can also be used to provide strength enhancement to reinforced concrete members subjected to combined axial compression and flexure. See Section 12.2 of ACI PRC-440.2-23 for specific requirements and limits.

4.5. Detailing of FRP strengthening

Except when a concrete section is completely confined with FRP, the typical governing failure mode for FRP is debonding of the FRP from the concrete substrate. Chapter 14 of ACI PRC-440.2-23 provides detailed recommendations to mitigate and control FRP debonding across various applications.

For flexural strengthening, Section 14.1.2 has recommendations for termination of the flexural FRP for simply-supported and continuous beams and slabs. If the flexural FRP has to be terminated at locations with high shear demands, U-wraps can be used to mitigate end debonding of the FRP.

For shear strengthening with U-wraps, Section 14.1.4 provides prescriptive guidance on the design of fiber anchors to prevent end debonding of the FRP and to achieve the maximum permissible strain of 0.004 in the U-wraps, thus enhancing the efficiency of the shear strengthening scheme.

For seismic strengthening, ACI PRC-440.2-23 recognizes the beneficial effect of anchoring the FRP to the concrete substrate in controlling debonding and in preventing the complete debonding of the FRP from the concrete member.

“With their world-class products and technically superior and advanced software, fischer is going to be a key-player in the field of structural strengthening. It gives me great pleasure to be associated with the software development of the REINFORCE-FIX module of FiXperience.”

Dr.-Ing. Akanshu Sharma

Jack and Kay Hockema Associate Professor in Civil Engineering, Lyles School of Civil and Construction Engineering, Purdue University, USA

5. REINFORCE-FIX

The fischer software REINFORCE-FIX enables the modeling and calculation of structural strengthening. It allows for the analysis of rectangular and T-beams. Additional modules will be integrated for extended functionality. This expert guide outlines the software's features for modeling, calculation, and result evaluation.

5.1. GUI and settings

In this chapter, a brief description of the REINFORCE-FIX graphical user interface is given as well as information on navigating in the work window and in the tables. In addition, basic setting options for the program are presented.

The REINFORCE-FIX user interface is based on the fischer C-FIX software to ensure intuitive operation. In the following the general structure is shown.

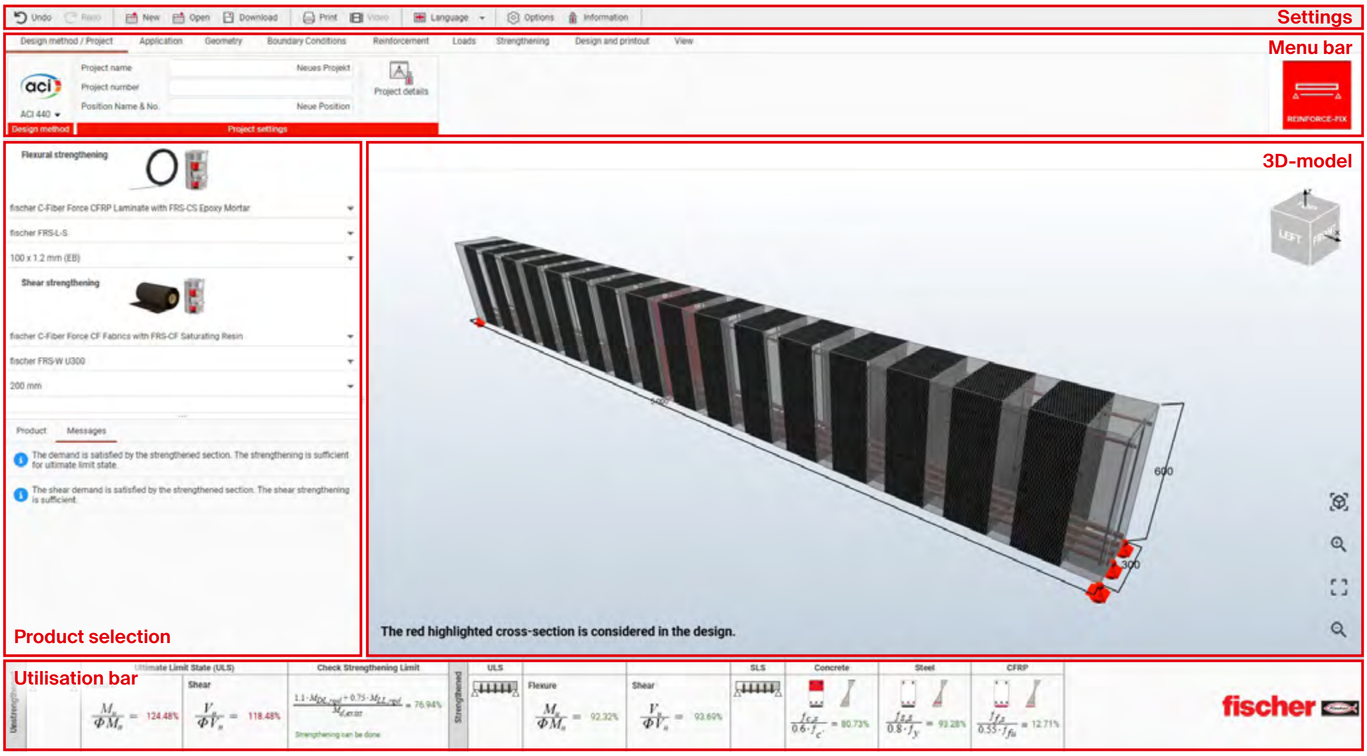
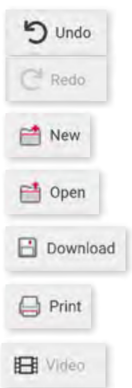


Fig. 10 – Structure of REINFORCE-FIX

5.1.1. Settings

The menu bar provides the general settings and is located at the top.

From left to right you can



undo or **redo** the last entry

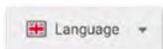
start a **new** project

open a saved project

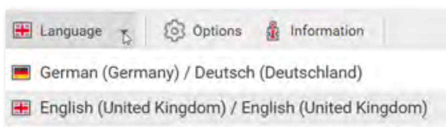
download and save the current project

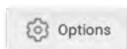
print a report, available in full or short version

watch a **video** of the design

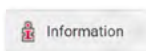


change the **language** settings





customize the unit system
(imperial or metric system) in **options**



find additional **information**, such as the software developer, license agreement, etc.

5.1.2. Menu bar

The menu bar at the top is the main navigation bar that guides the user from left to right during the design of the project.

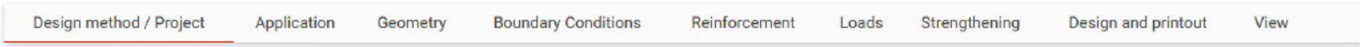


Fig. 11 – General menu bar

Design method / Project — General data, such as company name, address, phone number, and the applicable standard or guideline, can be defined in the '**Design Method/Project**' section

Application — The '**Application**' tab allows the user to select the structural element they wish to strengthen.

Geometry — The **Geometry** of the structural element as well as the concrete parameters can be defined

Boundary Conditions — Supports can be specified in **Boundary conditions**

Reinforcement — The **Reinforcement**, material, diameter and number, can be specified in this tab

Loads — In the **Loads** tab, moments and shear forces can be specified. Input masks for parabolic and triangular moment curvature are available

Strengthening — **Strengthening** products can be defined based on the number of plies, laminates, design, and other specific parameters for CFRP Laminates or CF Fabrics

Design and printout — In the **Design and printout** tab the user can download the printout

View — **View** offers many options to visualize the 3D-model in various ways.

5.1.3. Product selection

A product choice selection menu is displayed on the left-hand side, which shows the products with their various geometries.

Messages — Error messages are also shown below the product selection

5.1.4. 3D-model

The realistic 3D-model adapts immediately when changing any geometric values in the menu bar. The dimensions can be changed directly on the model using the dimension lines. It can be rotated by pressing the right mouse button. There is also a navigation cube on the right-hand side to easily receive a front, side or back view.

5.1.5. Utilization bar

The interactive utilization bar shows immediately the results after any adaption of geometry, concrete, or loads. Utilizations are presented in percent for any relevant verification. The user can see directly from the clear coloring whether the 100 % utilization has been exceeded (red color) or falls short (green color).

The utilization bar of the ACI module, see Fig. 12, is structured as follows: on the far left, the unstrengthened section is checked whether strengthening is necessary or not. Furthermore, the strengthening limit is checked. If strengthening is needed, the right part of the bar shows the utilizations for ULS and SLS verifications.

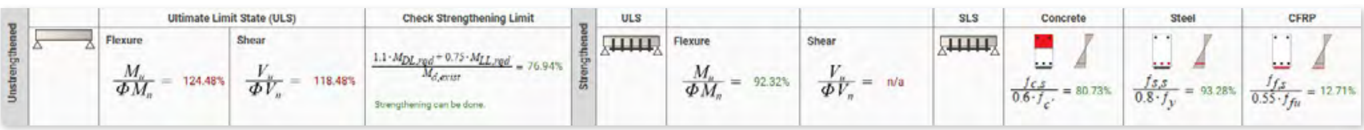


Fig. 12 – ACI utilization bar

5.2. Design method/project

REINFORCE-FIX offers different standards and guidelines to choose from.

The German DAfStb Strengthening guideline can be chosen for flexural and shear strengthening with CFRP Laminates or steel plates.

The American ACI PRC-440.2-23 “Design and Construction of Externally Bonded Fiber Reinforced Polymer (FRP) Systems for Strengthening Concrete Structures” can be chosen for flexural and shear strengthening with CFRP Laminates or CF Fabrics.

5.3. Application

Descriptions in the form of tool tips are available.

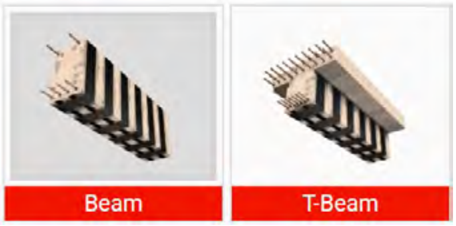


Fig. 13 – Application menu

The following applications can be calculated:
- Rectangular Beam with flexural and shear strengthening according to ACI PRC-440.2-23.
- Rectangular Beam with flexural and shear strengthening according to DAfStb Strengthening Guideline.

5.4. Geometry

Descriptions in the form of tool tips are available.

Fig. 14 – Geometry Menu bar, ACI

The concrete strength f_{ck} can be defined in the Material tab. The permitted minimum concrete compressive strength according to ACI PCR 440-14, 1.2.1.4 is 2500 psi. ACI doesn’t demand a maximum concrete compressive strength, but according to appendix B, v. “effects of high concrete strength on behavior of FRP-strengthened members” is a topic for future research. Therefore, it’s recommended to keep to the German DAfStb limits of a maximum concrete strength of 50 N/mm².
On the **Geometry** tab, the user can define the width, height, and length of the structural element. All dimensions can also be adjusted directly on the 3D-model itself.

5.5. Boundary conditions

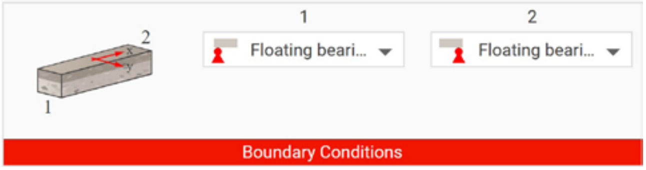


Fig. 15 – Boundary conditions

The boundary conditions can be defined here, see Fig. 15. Currently, only the floating bearing option is available.

5.6. Reinforcement

The reinforcement menu is structured into four tabs: material, stirrups, clear cover, and longitudinal reinforcement – see Fig. 16. In the Material menu, the yield strength f_{yk} can be defined. Yield strengths of 414 N/mm², 550 N/mm² or 690 N/mm² can be chosen when using the A615/A706 steel. The yield strength can also be defined by the user.

The input of the modulus of elasticity E_s is given according to ACI 318-14, 20.2.2.2 as 200 000 N/mm².
The diameter and the spacing of the stirrups can be specified. The limits for the diameter are a minimum of 4 mm and a maximum of 20 mm. Limits for the spacing are a minimum of 50 mm and a maximum of 1000 mm.

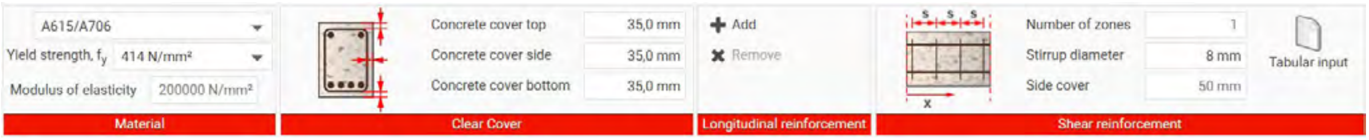


Fig. 16 – Overview ACI reinforcement input

Fig. 17 – Concrete cover

The concrete cover can be defined in **Clear cover** within the limits of 10 mm and 100 mm for the top, side, and bottom – see Fig. 16. Additional **Longitudinal reinforcement** can be added by the Add button; layers that are not required anymore can be deleted by the Remove button.

5.7. Loads

Load input according to ACI is divided into required $_{reqd}$ and existing $_{exist}$ loads. Both load cases are differentiated into dead load, DL, and live load, LL. Dead load includes the self-weight of the structure and the superimposed dead load. Live load refers to any moving load that the structure experiences. The input should be given in characteristic values as it is factored by the software. The load factors ϕ are overtaken from ACI 318 – 14, Table 5.3.1, but can be changed by the user. Design loads are estimated by the software and given in the two columns on the right, $M_{u,reqd}$ and $M_{u,existing}$. The service moment $M_{service}$ is determined for verifications in the Serviceability Limit State and is the sum of $M_{DL,reqd}$ and $M_{LL,reqd}$.

The following load cases according to ACI 318-14, Tab. 5.3.1, see Tab. 11, are determined in REINFORCE-FIX, as they are distinctive for the design. Additional load cases can be added.

Moment / Shear		Characteristic Loads					Load Factors Φ		Design Loads	
Load Combination	Load type	$M_{DL,exist}$	$M_{LL,exist}$	$M_{DL,reqd}$	$M_{LL,reqd}$	$M_{service}$	Factor DL	Factor LL	$M_{u,reqd}$	$M_{u,existing}$
Load Case 1	Static or quasi-static	100 kNm	80 kNm	100 kNm	150 kNm	250 kNm	1.4	0	140 kNm	140 kNm
Load Case 2	Static or quasi-static	100 kNm	80 kNm	100 kNm	150 kNm	250 kNm	1.2	1.6	360 kNm	248 kNm
Minimum unstrengthen...	Static or quasi-static	N/A	N/A	100 kNm	150 kNm	N/A	1.1	0.75	222,5 kNm	N/A

Fig. 18 – ACI load input

Tab. 11 – Load cases ACI

Load case 1	$U = 1{,}4 \cdot D = 1{,}4 \cdot M_{DL,reqd}$ $U = 1{,}4 \cdot D = 1{,}4 \cdot M_{DL,exist}$	ACI 318-14, Tab-5.3.1.
Load case 2	$U = 1{,}2 \cdot D + 1{,}6 \cdot L = 1{,}2 \cdot M_{DL,reqd} + 1{,}6 \cdot M_{LL,reqd}$ $U = 1{,}2 \cdot D + 1{,}6 \cdot L = 1{,}2 \cdot M_{DL,exist} + 1{,}6 \cdot M_{LL,exist}$	ACI 318-14, Tab-5.3.1.
Minimum unstrengthened limit	$U = 1{,}1 \cdot D + 0{,}75 \cdot L = 1{,}1 \cdot M_{DL,reqd} + 0{,}75 \cdot M_{LL,reqd}$	ACI PCR 440-14, Eq. 9.2

For a structural component to be considered for FRP strengthening, the existing component must possess a minimum level of strength to ensure that, in the event of incidents such as a fire, the structure can withstand potential collapse. This requirement according to ACI PRC-440.2-23, Eq. 9. is intended to ensure that the load-bearing capacity of the structural component without FRP reinforcement is greater than a design force that corresponds to the expected operating loads under typical situations.

It is checked as following:

$$(\phi R_n)_{exisiting} \geq (1.1 S_{DL} + 0.75 S_{LL})_{new} \quad (4.1)$$

The result is presented in the utilization bar.

5.8. Strengthening

The different strengthening methods are divided into two separate menus, one for flexural strengthening and the other for shear strengthening. The flexural strengthening is always selected, the shear strengthening can be added by the checkbox on the left.

5.8.1. Flexural strengthening

Fig. 19 – Flexural strengthening with externally bonded laminates, ACI

In the flexural strengthening menu, see Fig. 19, the number of plies and laminates can be defined. Furthermore, the user can choose in which exposure (interior, exterior, aggressive) the structural element is exposed. The environmental reduction factor is then adjusted automatically.

Additional inputs like the reduction factor Ψ_f and the adhesive thickness $t_{Adhesive}$ can be defined. The product menu on the left allows the user to choose whether he wants to perform strengthening with 'externally bonded' or 'near-surface mounted' laminates.

5.8.2. Shear strengthening

Fig. 20 – Shear strengthening with CF Fabrics, ACI

The shear strengthening menu enables users to select either a 'Closed' or 'Open' configuration for CF Fabrics. If an 'Open' design is chosen, the height h_f can be specified. In the longitudinal direction, the arrangement can be set to either 'Equally distributed' or 'Full surface'. Additionally, the number of plies must be defined, and if an 'Equally distributed' arrangement is selected, the spacing s_f must also be specified.

5.9. Design and printout

Fig. 21 – Printout

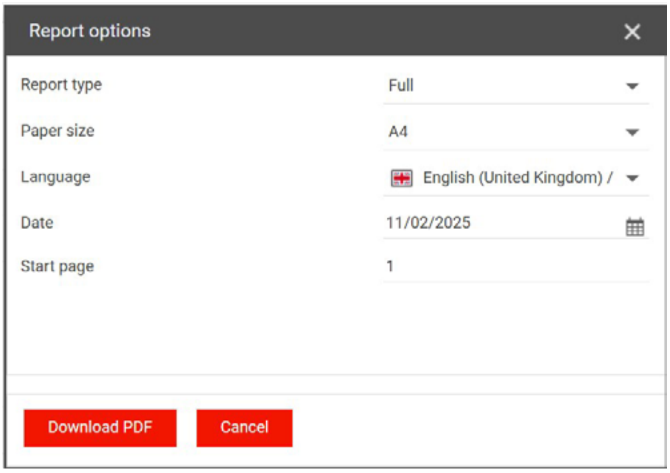


Fig. 22 – Printout options

Two different types can be chosen, a full and a short version. Language and date options as well as settings for the starting page are available.

5.10. View

Different view options can be chosen to display the 3D-model in different ways, see Fig. 23.

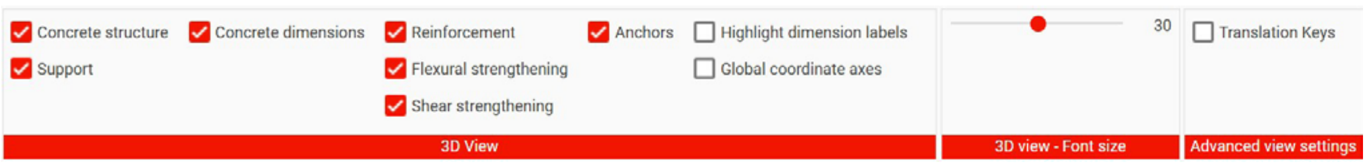
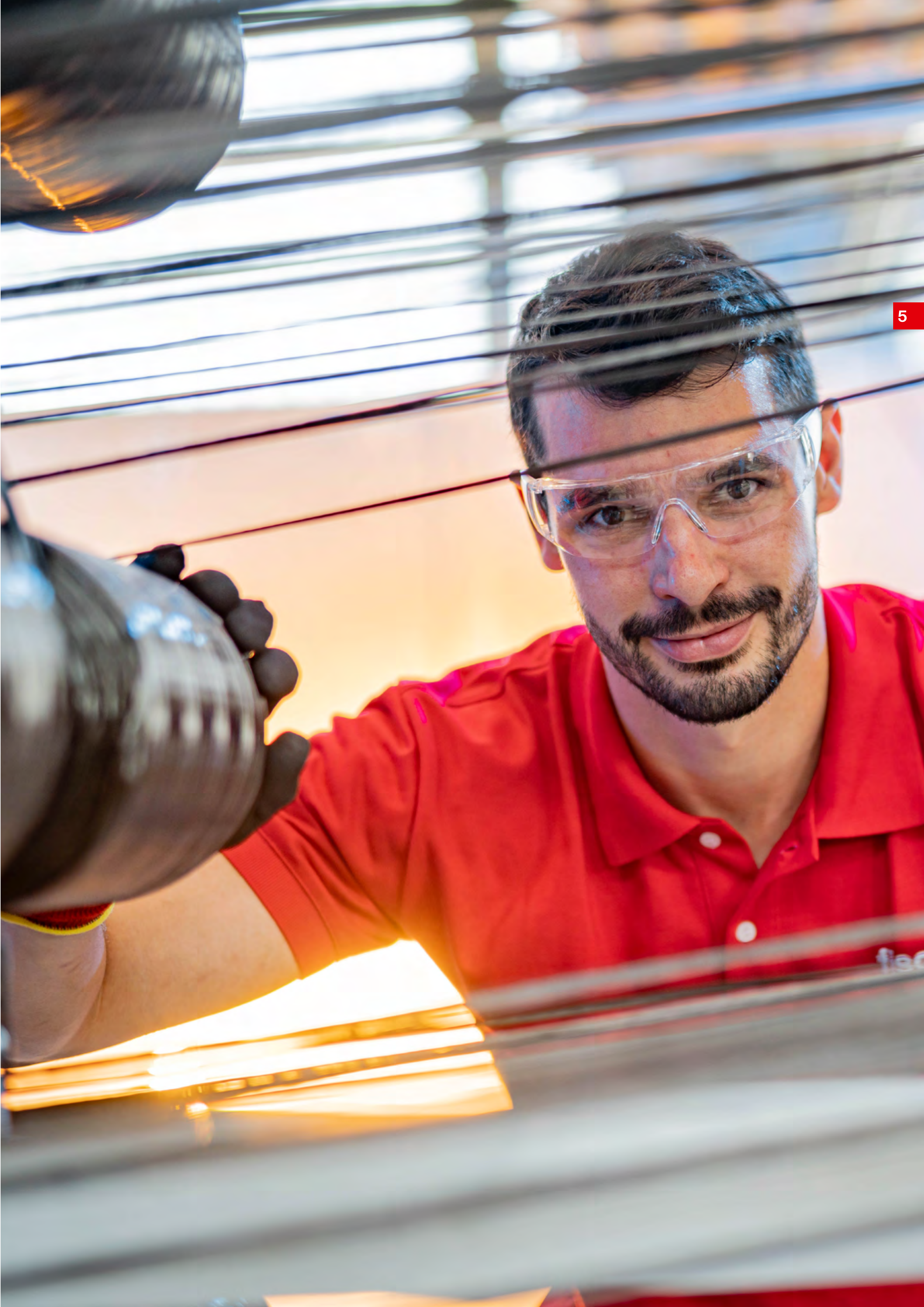


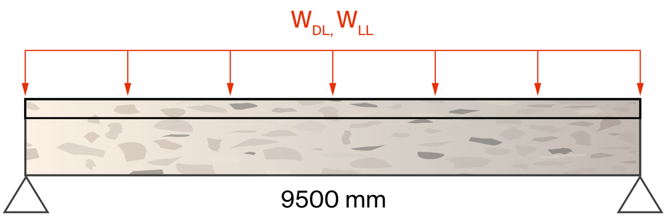
Fig. 23 – View settings



6. Design Examples according to ACI PRC-440.2-23

6.1. Example 1: Flexural and shear strengthening of a RC T-Beam with FRP

The simply supported T-beam shown in Fig. 24 is to be strengthened with FRP to resist increased superimposed dead loads (due to new flooring, finishes, etc.) and live loads in the building.



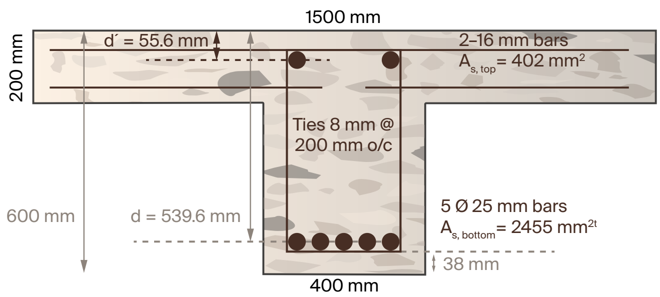


Fig. 24 – Beam section, span, and loading

Tab. 12 – Existing beam material properties

Concrete compressive strength,
$f'_c = 20\ MPa$

Steel reinforcement yield strength,
$f_y = 420\ MPa$

Steel reinforcement elastic modulus,
$E_s = 200\ GPa$

The tributary width of slab, that is the width of slab whose load is supported by the beam, is 3000 mm. The span of the beam is 9500 mm. The thickness of the slab is 200 mm.

The renovation of the building will increase the superimposed dead load (SDL) from 50 kg/m² to 125 kg/m² and the design live load (LL) from 250 kg/m² to 500 kg/m². Tab. 13 shows the different gravity loads acting on the beam. Tab. 14 and Tab. 15 show the resulting flexural and shear demands, respectively.

Tab. 13 – Existing and new gravity loads on the beam

Type		Existing	New
Dead Load (DL) w_{DL}		17.95 kN/m	17.95 kN/m
Superimposed Dead Load (SDL) w_{SDL}		1.50 kN/m	3.75 kN/m
Live Load (LL) w_{LL}		7.50 kN/m	15.00 kN/m
Existing capacity check limit (ACI 440.2-23 Eq. 9.2)	$1.1(w_{DL} + w_{SDL}) + 0.75w_{LL}$	NA	34.83 kN/m
Fire resistance check (ACI 440.2-23, Eq. 9.2.1)	$0.9(w_{DL} + w_{SDL}) + 0.50w_{LL}$	NA	26.82 kN/m
Factored design load	$1.2(w_{DL} + w_{SDL}) + 1.6w_{LL}$	35.08 kN/m	49.50 kN/m

Tab. 14 – Flexural demands on the beam

Type	Existing	New
Dead load moment $M_{DL} + S_{DL}$	218.7 kNm	243.6 kNm
Live load moment M_{LL}	82.9 kNm	165.7 kNm
Service level moment M_S	301.5 kNm	409.3 kNm
Existing capacity limit check (ACI 440.2-23 Eq. 9.2) $M_{u,limit} = 1.1(M_{DL} + M_{SDL}) + 0.75\ M_{LL}$	NA	392.2 kNm
Fire resistance check (ACI 440.2-23, Eq. 9.2.1) $M_{u,fire} = 0.9(M_{DL} + M_{SDL}) + 0.50\ M_{LL}$	NA	302.1 kNm
Factor design moment M_u	395.0 kNm	557.5 kNm

Tab. 15 – Shear demands on the beam

Type	Existing	New
Dead load shear $V_{DL} + S_{DL}$	92.0 kN	102.5 kN
Live load shear V_{LL}	34.9 kN	69.8 kN
Existing capacity limit check (ACI 440.2-23 Eq. 9.2) $V_{u,limit} = 1.1(V_{DL} + V_{SDL}) + 0.75\ V_{LL}$	NA	165.1 kN
Fire resistance check (ACI 440.2-23, Eq. 9.2.1) $V_{u,fire} = 0.9(V_{DL} + V_{SDL}) + 0.50\ V_{LL}$	NA	127.1 kN
Factored design shear V_u	166.3 kN	234.6 kN

The maximum moment demands occur at the midspan of the simply supported beam, while the maximum shear demands occur at the supports.

The existing beam section at midspan has the following capacities:

Flexure: $\phi M_n = 470.1\ KNm$
Shear: $\phi V_n = 177.3\ KN$

Tab. 16 and Tab. 17 detail the design of the FRP strengthening for flexure, per the provisions of ACI 440.2-23, with CF Fabrics and CFRP Laminates, respectively.

Tab. 18 details the design of the FRP strengthening for shear, in accordance with the provisions of ACI PRC-440.2-23, with CF fabrics.

ACI 440.2-23 has a strengthening limit based on the fire-resistance of the existing structural member. Per Section 9.2.1, an evaluation is required to ensure that the strengthened beam will not collapse in a fire. In Tab. 14 and Tab. 15, the values for $M_{u,fire}$ and $V_{u,fire}$ are calculated per Eq. 9.2.1. The capacity of the existing beam has to be calculated without FRP and based on reduced material properties corresponding to the fire exposure period required for the fire-resistance rating of the building. It is important to note that the existing beam is assumed to fulfill the fire-resistance rating requirement. By engineering judgement, it can be determined that the level of strengthening required is acceptable to meet this requirement. If the required strengthening is significantly higher, the fire-resistance of the beam without FRP and with reduced material properties due to fire exposure should be explicitly checked. More details and an illustrative example can be found in ACI PRC 440.10-21, Fire Resistance of FRP-Strengthened Concrete Members-Technote.

Tab. 16– Procedure for design of FRP strengthening for flexure with CF Fabric

5a. Select the CF Fabric U600 for flexural strengthening.
The following properties are provided by fischer:

ESR-4774
ICC-ES Ev. Report

$t_f = 1.02\ mm$

$E_f = 77\ GPa$

$f_{fu}* = 794\ MPa$

$\varepsilon_{fu}* = 1.26\% = 0.0126$

5b. Does the beam need to be strengthened for the new loads?

ACI 440.2-23
Eq. 10.1

$M_u = 557.5\ KNm$

$\phi M_{n,existing} = 470.1\ KNm$

$\phi M_{n,existing} < M_u$, therefore flexural strengthening is needed.

5c. Does the beam meet the minimum strength limits of Section 9.2?

ACI 440.2-23
Eq. 9.2

$M_{u,limit} = 392.2\ KNm$

$\phi M_{n,existing} = 470.1\ KNm$

$\phi M_{n,existing} > M_{u,limit}$, therefore FRP can be used.

5d. Initial assumption on FRP layering needed for flexural strengthening.
Assume that 2 layers, 300 mm wide, are required at the beam bottom to achieve the desired level of flexural strengthening.

$A_f = 2\ layers\ x\ 300\ mm\ x\ 1.02\ mm = 612\ mm^2$

Effective depth for the FRP, that is the distance from the extreme compression fiber to the centroid of the FRP, is,

$d_f = 600\ mm + \frac{2\ x\ 1.02\ mm}{2} = 601\ mm$

5e. Calculate the FRP system design material properties.
Since the slab is inside a building and will be strengthened with a carbon FRP, per Table 9.4, an environmental reduction factor of 0.95 is used.

ACI 440.2-23
Eq 9.4a
Eq. 9.4b

$f_{fu} = C_E f_{fu}* = 0.95\ x\ 794 = 754{,}3\ MPa$

$\varepsilon_{fu} = C_E \varepsilon_{fu}* = 0.95\ x\ 0.0126 = 0.0120$

5f. Determine the existing substrate strain.
The slab will have some existing level of strain due to existing loads. This strain level at the bottom of the slab is preexisting and cannot be mobilized in the FRP. Assuming that at the time of the strengthening only the dead and superimposed dead loads are acting on the slab, the existing (initial) substrate strain can be calculated from elastic analysis of the section using cracked section properties. The following equation can be used:

ACI 440.2-23
Ex. 16.3, Table 16.3c

$$\varepsilon_{bi} = \frac{M_{DL}(d_f - kd)}{I_{cr} E_c}$$

The common approach is to use a cracked transformed section wherein the top and bottom steel reinforcement area is represented in terms of concrete area by multiplying them by the modular ratio, E_s/E_c.

$E_c = 21{,}174\ MPa$

$E_s = 200{,}000\ MPa$

$n = E_s/E_c = 9.44$

The transformed reinforcement areas are:
$A_{sT,bot} = nA_{s,bot} = 23{,}189\ mm^2$
$A_{sT,top} = (n-1)A_{s,top} = 3{,}394\ mm^2$

It is assumed that the top reinforcement is within the neutral axis depth and within the flange; hence the modular ratio is reduced by 1.0 to account for the displaced concrete area.
Solving the resultant quadratic equation,
$c = kd = depth\ of\ the\ neutral\ axis = 113.6\ mm$

The top bar is within the neutral axis depth as assumed.
The resulting cracked transformed section is shown below in Fig. 25.

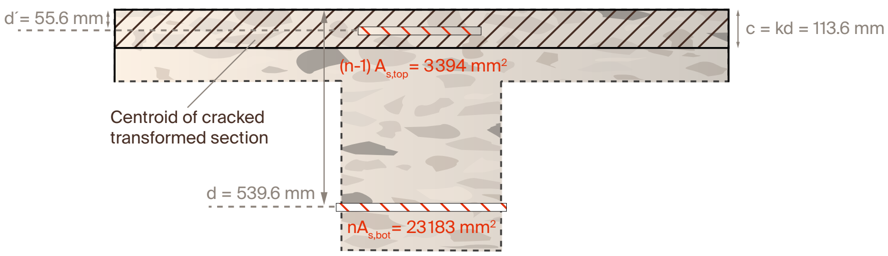


Fig. 25 – Transformed cracked section for elastic analysis

The transformed cracked section modulus is,
$I_{cr} = 4.953\ x\ 10^9$

There, the initial strain at the concrete before FRP strengthening can be calculated as,

$$\varepsilon_{bi} = \frac{243.6\ x\ 10^3\ KNmm\ (601 - 113.6)}{4.953\ x\ 10^9\ x\ 21.174\ KPa} = 0.00113$$

5g. Determine the effective strain FRP at which debonding may occur. ACI 440.2-23 Eq. 10.1.1
The strain in the FRP has to be limited to the strain at which debonding will occur. The effective strain, ε_{fd}, can be calculated as,

$$\varepsilon_{fd} = 0.41\sqrt{\frac{f'_c}{NE_f t_f}} \leq 0.9\varepsilon_{fu}$$

$$\varepsilon_{fd} = 0.41\sqrt{\frac{20\ MPa}{2\ x\ 77{,}000\ MPa\ x\ 1.02\ mm}} \leq 0.0108$$

$$\varepsilon_{fd} = 0.00463 \leq 0.0108$$

5h. Calculate the state of strain in the section. Chapter 2, Notations & Definitions
Equilibrium in the section depends on the failure mode at the ultimate limit state.
- If concrete crushing controls, then equilibrium is when strain at the extreme compression fiber, ε_{cu}, reaches the maximum usable value of 0.003. This limit is typical for design per ACI 318 & 440.2-23.
- If debonding of the FRP controls, then equilibrium is checked when strain in FRP reaches ε_{fd}.

Assume that the depth of the neutral axis (NA) below the top of the beam is at 15 % of the total beam depth. For rectangular beams, a good initial approximation is 20 % of the beam depth.
$c = 0.15\ x\ 600\ mm = 90\ mm$

Assuming that the compressive strain at the extreme compression fiber reaches 0.003, calculate the strain in the FRP,

$$\varepsilon_{fe} = \varepsilon_{cu}\left(\frac{d_f - c}{c}\right) - \varepsilon_{bi} \leq \varepsilon_{fd}$$ Eq. 10.2.5

$$\varepsilon_{fe} = 0.003\left(\frac{601\,mm - 90\,mm}{90\,mm}\right) - 0.00113 \leq 0.00463$$

$$\varepsilon_{fe} = 0.0159 > 0.00463$$

It is observed that the FRP will debond before the strain reaches 0.003 at the extreme compression fiber. This indicates that FRP debonding will control the failure mode.

For design, the strain in the FRP must be limited to ε_{fd},

$$\varepsilon_{fe} = \varepsilon_{fd} = 0.00463$$

Calculate the strain at the extreme compression fiber at ε_{fe}. Rearranging Eq. 10.2.5,

$$\varepsilon_c = (\varepsilon_{fe} + \varepsilon_{bi})\left(\frac{c}{d_f - c}\right)$$

$$\varepsilon_c = (0.00463 + 0.00113)\left(\frac{90\,mm}{601\,mm - 90\,mm}\right)$$

$$\varepsilon_c = 0.00101 \leq 0.003$$

The strain at which concrete is no longer linearly elastic is taken as $0.85f'_c/E_c = 0.00080$. Since the strain at the extreme compression fiber is greater than this value, it can be considered that the concrete compression block will experience a parabolic distribution of the strain.

Calculate the strain in the top and bottom reinforcement.
Using similar triangles, the strain in the bottom steel can be determined as,

$$\varepsilon_s = (\varepsilon_{fe} + \varepsilon_{bi})\left(\frac{d - c}{d_f - c}\right)$$ Eq. 10.2.10a

$$\varepsilon_s = (0.00463 + 0.00113)\left(\frac{539.6\,mm - 90\,mm}{601\,mm - 90\,mm}\right)$$

$$\varepsilon_s = 0.0051 > \varepsilon_y = \frac{f_y}{E_s} = 0.0021$$

The bottom reinforcing steel is yielding.
Similarly, the strain in the top reinforcement is,

$$\varepsilon'_s = 0.00039 < \varepsilon_y = 0.0021$$

The top reinforcing steel is below yield.
Using the strains calculated above, determine the forces in the different components, and check equilibrium.

5i. Check equilibrium in the section. ACI 440.2-23 Eq. 10.2.10c

Assuming the neutral axis lies within the flange, internal equilibrium can be checked using,

$$\alpha_1 f'_c \beta_1 bc + A_{s,top} f_{s\prime} = A_{s,bot} f_s + A_f f_{fe}$$

If the top reinforcing is below the NA, it will add a tensile component to the section.
The parameters defining the rectangular stress block for the nonlinear distribution of stress in the concrete compression block are,

$$\alpha_1 = \frac{3\varepsilon_c'\varepsilon_c - \varepsilon_c^2}{3\beta_1 \varepsilon_c'^2}$$

$$\beta_1 = \frac{4\varepsilon_c' - \varepsilon_c}{6\varepsilon_c' - 2\varepsilon_c}$$

Where, ε'_c corresponds to the strain at f'_c,

$$\varepsilon'_c = \frac{1.7f'_c}{E_c} = \frac{1.7\ x\ 20\ MPa}{21{,}174\ MPa} = 0.00161$$

$$\beta_1 = \frac{4\ x\ 0.00161 - \ 0.00101}{6\ x\ 0.00161 - 2\ x\ 0.00101} = 0.711$$

$$\alpha_1 = \frac{3\ x\ 0.00161\ x\ 0.00101 - 0.00101^2}{3\ x\ 0.711\ x\ 0.00161^2} = 0.699$$

The compressive component in concrete,

$C = \alpha_1 f'_c \beta_1 bc = 0.703\ x\ 20\ MPa\ x\ 0.711\ x\ 1{,}500\ mm\ x\ 90\ mm$

$C = 1{,}341.9\ KN$

Compressive force in the top reinforcement,

$C_{s,top} = A_{s,top}\varepsilon'_s E_s = 402\ mm^2\ x\ 0.00039$ x 200,000 MPa

$C_{s,top} = 31.4\ KN$

Tensile force in the bottom reinforcement,

$T_{s,bot} = A_{s,bot} x\ Min(\varepsilon_s E_s, F_y)$

$T_{s,bot} = 2{,}455\ mm^2\ x\ Min\ (0.0051\ x\ 200{,}000\ MPa, 420\ MPa)$

$T_{s,bot} = 1{,}031.1\ KN$

Tensile force in the FRP,

$T_f = A_f \varepsilon_{fe} E_f = 612\ mm^2\ x\ 0.00463$ x 77,000 MPa

$T_f = 218.2\ KN$

Check equilibrium,

$C + C_{s,top} = 1{,}308.4\ KN + 31.4\ KN = 1{,}373.3\ KN$

$T_{s,bot} + T_f = 1{,}031.1\ KN + 218.2\ KN = 1{,}249.3\ KN$

$C + C_{s,top} \neq T_{s,bot} + T_f$

Therefore, equilibrium is not satisfied.

Since the compressive force is larger than the tensile force, the depth of the NA is smaller than the initial assumption.

After iteration, the depth of the NA to achieve equilibrium is calculated as,

$c = 85.48\ mm$

Recalculate the strains in the section using FRP debonding as the controlling failure mode at,

$\varepsilon_{fe} = 0.00463$

$\varepsilon_c = 0.00095 \leq 0.003$

$\varepsilon_s = 0.0051 > \varepsilon_y = 0.0021$

$\varepsilon'_s = 0.00033 < \varepsilon_y = 0.0021$

The different force components are calculated as below:

$\beta_1 = 0.708$

$\alpha_1 = 0.674$

$C = 1{,}249.17\ KN$

$T_{s,top} = 26.53\ KN$

$T_{s,bot} = 1{,}031.09\ KN$

$T_f = 218.2\ KN$

$C + T_{s,top} = 1{,}249.17\ KN + 26.53\ KN = 1{,}249.17\ KN$

$T_{s,bot} + T_f = 1{,}031.09\ KN + 218.2\ KN = 1{,}249.3\ KN$

$C + T_{s,top} = \ T_{s,bot} + T_f$

Equilibrium is satisfied.

5j. Determine the flexural capacity of the strengthened section.
The flexural capacity provided by each force component is calculated as the product of the force times the level from the extreme compression fiber, i.e., top of the beam section.

Component	Force F	Lever arm y¯	Moment M = F$_{y}$.
Concrete compression	-1249.17 kN	$L_c = \beta_1 c/2$ $L_c = 0.708 \times 85.48$ mm/2 $L_c = 30.26$ mm	-37800 kNmm
Top reinf. compression	-26.53 kN	d' = 55.6 mm	-1475 kNmm
Bottom reinf. compression	1031.09 kN	d = 539.6 mm	556327 kNmm
FRP tension	218.2 kN	d_f = 601 mm	131138 kNmm

The negative sign for the force indicates compression while the positive sign indicates tension. From Step 5h, the strain in the bottom reinforcement is calculated as 0.0051. The strength reduction factor, ϕ, is calculated as,

ACI 440.2-23 Eq. 10.2.7

$$\phi = 0.65 + \frac{0.25(\varepsilon_s - \varepsilon_y)}{(0.005 - \varepsilon_y)}$$

$$\phi = 0.65 + \frac{0.25(0.0051 - 0.0021)}{(0.005 - 0.0021)}$$

$$\phi = 0.9086$$

An additional reduction factor of y = 0.85 s applied to the FRP's effect on the flexural capacity.
The final moment capacity of the strengthened section is,

Section 10.2.10

$$\phi M_n = 0.9086(556{,}376\ KNmm + 0.85\ x\ 131{,}138\ KNmm - 37{,}800\ KNmm\ - 1{,}475 KNmm)$$
$$\phi M_n = 571{,}117\ KNmm$$
$$\phi M_n = 571.1\ KNm$$

The required capacity is,
$$M_u = 557.5\ KNm$$

Therefore,
$$\phi M_n > M_u$$
$$DCR = \frac{557.5\ KNm}{571.1\ KNm} = 0.98 \leq 1.0\ \therefore OK$$

2 layers of U600, 300 mm wide, are adequate to achieve the desired level of flexural strengthening.

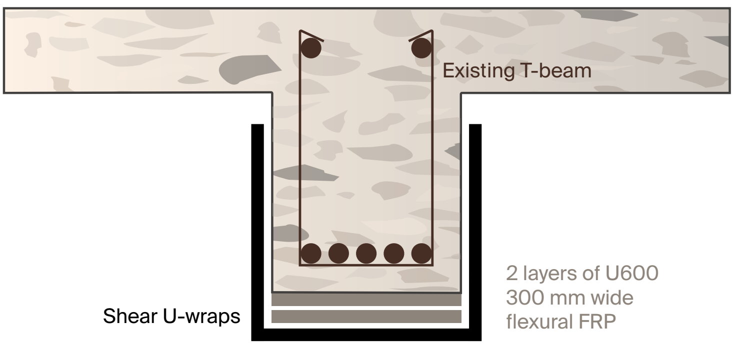


Fig. 26 – Final configuration of flexural strengthening with CF Fabric

5k. Check the service level stresses in the reinforcing steel, concrete and FRP.
It is important to ensure that the strengthened beam will fulfill serviceability requirements, such as deflections and crack widths. Such checks are performed using a transformed section analysis, similar to Step 5f. The stresses in the reinforcing steel and concrete are restricted to,

$f_{s,s} \leq 0.80 f_y$ — ACI 440.2-23 Eq. 10.2.8a

$f_{c,s} \leq 0.60 f'_c$ — Eq. 10.2.8b

The stress in the bottom reinforcing steel under service loads can be found by, — Eq. 10.2.10.1

$$f_{s,s} = \frac{\left[M_s + \varepsilon_{bi} A_f E_f \left(d_f - {}^{kd}/_3\right)\right](d-kd)E_s}{\left[A_s E_s \left(d - {}^{kd}/_3\right)(d-kd) + A_f E_f \left(d_f - {}^{kd}/_3\right)(d_f - kd)\right]}$$

6

It is also possible to calculate the stress in the bottom reinforcing steel by iterating on the depth of the NA and computing the strains, and associated stresses, in the different components. It should be noted that $f_{s,s}$ from the two approaches may not be identical.
To calculate *kd*, a cracked transformed section is developed like in Step 5f. Since FRP is present at the bottom of the strengthened section, a transformed area of FRP has to be included. The resulting section will resemble the figure in Step 5f with additional transformed FRP area, $nA_f = \left(\frac{E_f}{E_c}\right) A_f$ at the bottom. From such a transformed section,

$c = kd = 119.0\ mm$

For a service level moment $M_s = 409.4\ KNm$

$$f_{s,s} = \frac{\begin{matrix}\left[409.4\ KNm + 0.00113x612\ mm^2 x\ 77\ GPa\left(601\ mm - {}^{119.0\ mm}/_3\right)x10^{-3}\right]x \\ (539.6\ mm - 119.0\ mm)x\ 200\ GPa\ x\ 10^{-3}\end{matrix}}{\begin{bmatrix}2{,}455\ mm^2 x200\ GPa\left(539.6\ mm - {}^{119.0\ mm}/_3\right)(539.6\ mm - 119.0\ mm) + \\ 612\ mm^2 x77\ GPa\left(601\ mm - {}^{119.0\ mm}/_3\right)(601\ mm - 119.0\ mm)\end{bmatrix} x10^{-9}}$$

$f_{s,s} = 318.6\ MPa < 0.8 f_y = 336.2\ MPa\ \therefore OK$

The stress in the FRP under service loads can be calculated by, — Eq. 10.2.10.2

$$f_{f,s} = f_{s,s}\left(\frac{E_f}{E_s}\right)\frac{d_f - kd}{d - kd} - \varepsilon_{bi} E_f$$

$$f_{f,s} = 318.6\ MPa\left(\frac{77\ GPa}{200\ GPa}\right)\frac{601\ mm - 119.0\ mm}{539.6\ mm - 119.0\ mm} - 0.00113x77{,}000\ MPa$$

$f_{f,s} = 53.6\ MPa < 0.55 f_{fu} = 414{,}9\ MPa\ \therefore OK$

The $0.55f_{fu}$ limit is intended to avoid creep rupture of the FRP under sustained service loads. — Section 10.2.9, Table 10.2.9
The stress in the top steel under service loads can be calculated by,

$$f_{s,s'} = f_{s,s}\frac{kd - d'}{d - kd}$$

$f_{s,s'} = 48.02\ MPa$

The stress at the extreme compression fiber in concrete can be found by,

$f_{c,s} = {}^{C}/_{(b\ x\ kd)/2} = 2C/(b\ x\ kd)$

To achieve equilibrium, the compressive force is,

$C = f_{s,s} A_{s,bot} + f_{f,s} A_f - f_{s,s'} A_{s,top}$

$C = 318.6\ MPa\ x\ 2{,}455\ mm^2 + 53.6\ MPa\ x\ 612\ mm^2 - 48.02\ MPa\ x\ 402\ mm^2$

$C = 795.66\ KN$

$$f_{c,s} = \frac{2\ x\ 795.66\ x\ 1000\ N}{(1{,}500\ mm\ x\ 119.0\ mm)}$$

$f_{c,s} = 8.91\ MPa < 0.6 f'_c = 12\ MPa\ \therefore OK$

The service level stresses comply with the limits in ACI 440.2-23.

5l. Detailing of flexural FRP.
For simply supported beams, the flexural FRP should be extended beyond the point along the span at which the flexural demand is below the cracking moment by at least a distance equal to the development length,

ACI 440.2-23 Eq. 14.1.3

$$l_{df} = \sqrt{\frac{NE_f t_f}{\sqrt{f'_c}}}$$

$$l_{df} = \sqrt{\frac{2\ x\ 77{,}000\ MPa\ x\ 1.02\ mm}{\sqrt{20\ MPa}}} = 187\ mm$$

6

Since there are two layers, the first layer should be extended a further 150 mm beyond this length to achieve a staggered termination of the FRP.
Generally, it is recommended to extend the FRP to within 25 mm of the support to mitigate concrete cover delamination.

Figure 14.1.2b

If the FRP is terminated where the shear demand, V_u, is greater than $0.67V_c$, the flexural FRP should be anchored to the beam with transverse clamping FRP U-wraps. The area of the U-wraps can be found per Eq. 14.1.2.

Section 14.1.2

Tab. 17 – Procedure for design of FRP strengthening for flexure with laminate

6a. Select the laminate FRS-L-S for beam strengthening.
The following properties are provided by fischer:

ESR-4774 ICC-ES Ev. Report

$t_f = 1.02\ mm$
$w_f = 100\ mm$
$E_f = 168\ GPa$
$f_{fu}* = 2585\ MPa$
$\varepsilon_{fu}* = 1.77\ \% = 0.0177$

6b. Does the beam need to be strengthened for the new loads?

ACI 440.2-23 Eq. 10.1

$M_u = 557.5\ KNm$
$\phi M_{n,existing} = 470.1\ KNm$
$\phi M_{n,existing} < M_u$, therefore flexural strengthening is needed.

6c. Does the beam meet the minimum strength limits of Section 9.2?

ACI 440.2-23 Eq. 9.2

$M_{u,limit} = 392.2\ KNm$
$\phi M_{n,existing} = 470.1\ KNm$
$\phi M_{n,existing} > M_{u,limit}$, therefore FRP can be used.

6d. Initial assumption on FRP layering needed for flexural strengthening.
Assume that seven (7) 50 mm wide × 1.2 mm thick laminates strips are required at the beam bottom to achieve the desired level of flexural strengthening.
$A_f = 1\ layer\ x\ 7\ strips\ x\ 50\ mm\ x\ 1.2\ mm = 420\ mm^2$

Effective depth for the FRP, that is the distance from the extreme compression fiber to the centroid of the FRP, is,
$d_f = 600\ mm + (1.2\ mm)/2 = 600.6\ mm$

6e. Calculate the FRP system design material properties.

ACI 440.2-23 Eq 9.4a Eq. 9.4b

Since the beam is inside a building and will be strengthened with a carbon FRP, per Table 9.4, an environmental reduction factor of 0.95 is used.
$f_{fu} = C_E f_{fu}* = 0.95\ x\ 2{,}585 = 2{,}456\ MPa$
$\varepsilon_{fu} = C_E \varepsilon_{fu}* = 0.95\ x\ 0.0177 = 0.0168$

6f. Determine the existing substrate strain.
Similar to Step 5f, the existing (initial) substrate strain at the bottom of the beam can be calculated from elastic analysis of the section using cracked section properties,

ACI 440.2-23 Ex. 16.3, Table 16.3c

$$\varepsilon_{bi} = \frac{M_{DL}(d_f - kd)}{I_{cr}E_c}$$

$$\varepsilon_{bi} = \frac{243.6\ x\ 10^3\ KNmm\ (601 - 113.6)}{4.953\ x\ 10^9\ x\ 21.174\ KPa} = 0.00113$$

6g. Determine the effective strain in the FRP at which debonding may occur.
The strain in the FRP has to be limited to the strain at which debonding will occur. The effective strain, ε_{fd}, can be calculated as,

ACI 440.2-23 Eq. 10.1.1

$$\varepsilon_{fd} = 0.41\sqrt{\frac{f'_c}{NE_f t_f}} \leq 0.9\varepsilon_{fu}$$

$$\varepsilon_{fd} = 0.41\sqrt{\frac{20\ MPa}{1\ x\ 168{,}000\ MPa\ x\ 1.2\ mm}} \leq 0.0103$$

$$\varepsilon_{fd} = 0.00408 \leq 0.0103$$

6h. Calculate the state of the strain in the section.
As described in Step 5h, the depth of the neutral axis at equilibrium is required to calculate the strains at different components in the section. After iteration, it is determined that the depth of the neutral axis to achieve equilibrium is,

Eq. 10.2.5

$c = 91.45\ mm$

Assuming that the compressive strain at the extreme compression fiber reaches 0.003, calculate the strain in the FRP,

$$\varepsilon_{fe} = \varepsilon_{cu}\left(\frac{d_f - c}{c}\right) - \varepsilon_{bi} \leq \varepsilon_{fd}$$

$$\varepsilon_{fe} = 0.003\left(\frac{600.6\ mm - 91.45\ mm}{91.45\ mm}\right) - 0.00113 \leq 0.00408$$

$$\varepsilon_{fe} = 0.0156 > 0.00408$$

It is observed that the FRP will debond before the strain reaches 0.003 at the extreme compression fiber. This indicates that FRP debonding will control the failure mode.

For design the strain in the FRP must be limited to ε_{fd}

$\varepsilon_{fe} = \varepsilon_{fd} = 0.00408$

Eq. 10.2.10a

Calculate the strain at the extreme compression fiber at ε_{fe}. Rearranging Eq. 10.2.5

$$\varepsilon_c = (\varepsilon_{fe} + \varepsilon_{bi})\left(\frac{c}{d_f - c}\right)$$

$$\varepsilon_c = (0.00408 + 0.00113)\left(\frac{91.45\ mm}{600.6\ mm - 91.45\ mm}\right)$$

$$\varepsilon_c = 0.00094 \leq 0.003$$

The strain at which concrete is no longer linearly elastic is taken as $0.85f'_c/E_c$ = 0.00080. Since the strain at the extreme compression fiber is greater than this value, it can be considered that the concrete compression block will experience a parabolic distribution of strain.

Calculate the strain in the top and bottom reinforcement.
Using similar triangles, the strain in the bottom steel can be determined as,

$$\varepsilon_s = (\varepsilon_{fe} + \varepsilon_{bi})\left(\frac{d-c}{d_f-c}\right)$$

$$\varepsilon_s = (0.00408 + 0.00113)\left(\frac{539.6\ mm - 91.45\ mm}{600.6\ mm - 91.45\ mm}\right)$$

$$\varepsilon_s = 0.0046 > \varepsilon_y = \frac{f_y}{E_s} = 0.0021$$

The bottom reinforcing steel is yielding. Similarly, the strain in the top reinforcement is

$$\varepsilon'_s = 0.00037 < \varepsilon_y = 0.0021$$

The top reinforcing steel is below yield.
Using the strains calculated above, determine the forces in the different components, and check equilibrium.

6i. Check equilibrium in the section.
Internal equilibrium can be checked using,

ACI 440.2-23
Eq. 10.2.10c

$$\alpha_1 f'_c \beta_1 bc + A_{s,top} f_{s'} = A_{s,bot} f_s + A_f f_{fe}$$

If the top reinforcement is below the neutral axis, a tensile component will be added to the section. The parameters defining the rectangular stress block for the nonlinear distribution of stress in the concrete compression block are,

$$\alpha_1 = \frac{3\varepsilon_c' \varepsilon_c - \varepsilon_c^2}{3\beta_1 \varepsilon_c'^2}$$

$$\beta_1 = \frac{4\varepsilon_c' - \varepsilon_c}{6\varepsilon_c' - 2\varepsilon_c}$$

Where, ε'_c corresponds to the strain at f'_c,

$$\varepsilon'_c = \frac{1.7 f'_c}{E_c} = \frac{1.7\ x\ 20\ MPa}{21{,}174\ MPa} = 0.00161$$

$$\beta_1 = \frac{4\ x\ 0.00161 -\ 0.00094}{6\ x\ 0.00161 - 2\ x\ 0.00094} = 0.707$$

$$\alpha_1 = \frac{3\ x\ 0.00161\ x\ 0.00094 - 0.00094^2}{3\ x\ 0.707\ x\ 0.00161^2} = 0.6647$$

The compressive component in concrete,

$$C = \alpha_1 f'_c \beta_1 bc = 0.6647\ x\ 20\ MPa\ x\ 0.707\ x\ 1{,}500\ mm\ x\ 91.45\ mm$$

$$C = 1{,}289.3\ KN$$

Compressive force in the top reinforcement,

$$T_{s,top} = A_{s,top} \varepsilon'_s E_s = 402\ mm^2\ x\ 0.00037\ \text{x}\ 200{,}000\ \text{MPa}$$

$$T_{s,top} = 29.75\ KN$$

Tensile force in the bottom reinforcement,

$T_{s,bot} = A_{s,bot} x\ Min(\varepsilon_s E_s, F_y)$

$T_{s,bot} = 2{,}455\ mm^2\ x\ Min\ (0.0046\ x\ 200{,}000\ MPa, 420\ MPa)$

$T_{s,bot} = 1{,}031.09\ KN$

Tensile force in the FRP,

$T_f = A_f \varepsilon_{fe} E_f = 420\ mm^2\ x\ 0.00408 \text{ x } 168{,}000 \text{ MPa}$

$T_f = 287.9\ KN$

Check equilibrium,

$C + T_{s,top} = 1{,}289.3\ KN + 29.75\ KN = 1{,}319.0\ KN$

$T_{s,bot} + T_f = 1{,}031.1\ KN + 287.9\ KN = 1{,}319.0\ KN$

$C + T_{s,top} = T_{s,bot} + T_f$

Therefore, equilibrium is satisfied.

6j. Determine the flexural capacity of the strengthened section.
The flexural capacity provided by each force component is calculated as the product of the force times the level from the extreme compression fiber, i.e., top of the beam section.

Component	Force F	Lever arm $\bar{y}$	Moment M = F$\bar{y}$
Concrete compression	-1289.3 kN	$L_c = \beta_1 c/2$ L_c = 0.707 × 91.45 mm/2 L_c = 32.33 mm	-41683 kNmm
Top reinf. compression	-29.75 kN	d' = 55.6 mm	-1654 kNmm
Bottom reinf. compression	1031.1 kN	d = 539.6 mm	556382 kNmm
FRP tension	287.9 kN	d_f = 600.6 mm	172913 kNmm

The negative sign for the force indicates compression while the positive sign indicates tension. From Step 6h, the strain in the bottom reinforcement is calculated as 0.0047. The strength reduction factor, f, is calculated as,

ACI 440.2-23 Eq. 10.2.7

$$\phi = 0.65 + \frac{0.25(\varepsilon_s - \varepsilon_y)}{(0.005 - \varepsilon_y)}$$

$$\phi = 0.65 + \frac{0.25(0.0046 - 0.0021)}{(0.005 - 0.0021)}$$

$$\phi = 0.8655$$

Section 10.2.10

An additional reduction factor of ψ = 0.85 is applied to the FRP's effect on the flexural capacity.
The final moment capacity of the strengthened section is,

$\phi M_n = 0.8655(556{,}382\ KNmm + 0.85\ x\ 172{,}913\ KNmm - 41{,}683\ KNmm\ - 1{,}654\ KNmm)$

$\phi M_n = 571{,}248\ KNmm$

$\phi M_n = 571.2\ KNm$

The required capacity is,

$M_u = 557.5\ KNm$

Therefore,

$\phi M_n > M_u$

$$DCR = \frac{557.5\ KNm}{571.2\ KNm} = 0.98 < 1.0\ \therefore OK$$

1 layer of 7 – 50 mm wide FRS-L-S laminate strips are adequate to achieve the desired level of flexural strengthening.

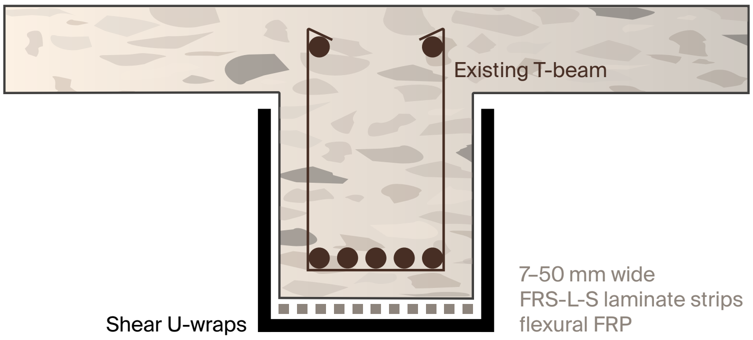


Fig. 27 – Final configuration of flexural strengthening with laminates

6k. Check the service level stresses in the reinforcing steel, concrete and FRP. As described in Step 5k, the stresses in the reinforcing steel and concrete at service level loads are to be restricted to,

ACI 440.2-23
Eq. 10.2.8a
Eq. 10.2.8b

$f_{s,s} \leq 0.80 f_y$

$f_{c,s} \leq 0.60 f'_c$

The stress in the bottom reinforcing steel under service loads can be found by,

Eq. 10.2.10.1

$$f_{s,s} = \frac{\left[M_s + \varepsilon_{bi} A_f E_f \left(d_f - {}^{kd}/_3\right)\right](d - kd)E_s}{\left[A_s E_s \left(d - {}^{kd}/_3\right)(d - kd) + A_f E_f \left(d_f - {}^{kd}/_3\right)(d_f - kd)\right]}$$

To calculate kd, a cracked transformed section is developed with the top and bottom reinforcing steel as well as the FRP expressed in terms of transformed areas. From such a transformed section,

$c\ =\ kd\ =\ 121.5\ mm$

For a service level moment $M_s = 409.4\ KNm$

$f_{s,s} = 312.8\ MPa < 0.8 f_y = 336.2\ MPa\ \therefore OK$

The stress in the FRP under service loads can be calculated by,

$$f_{f,s} = f_{s,s}\left(\frac{E_f}{E_s}\right)\frac{d_f - kd}{d - kd} - \varepsilon_{bi} E_f$$

$f_{f,s} = 111.3\ MPa < 0.55 f_{fu} = 1{,}350.8\ MPa\ \therefore OK$

Eq. 10.2.10.2

The $0.55 f_{fu}$ limit is intended to avoid creep rupture of the FRP under sustained service loads. The stress in the top steel under service loads can be calculated by,

Section 10.2.9
Table 10.2.9

$$f_{s,s'} = f_{s,s}\frac{kd - d'}{d - kd}$$

$f_{s,s'} = 49.3\ MPa$

The stress at the extreme compression fiber in concrete can be found by,

$$f_{c,s} = {}^{C}/_{(b\ x\ kd)/2} = 2C/(b\ x\ kd)$$

To achieve equilibrium, the compressive force is,

$$C = f_{s,s}A_{s,bot} + f_{f,s}A_f - f_{s,s'}A_{s,top}$$

$$C = 312.8\ MPa\ x\ 2455\ mm^2 + 111.3\ MPa\ x\ 420\ mm^2 - 49.3\ MPa\ x\ 402\ mm^2$$

$$C = 794.85\ KN$$

$$f_{c,s} = \frac{2\ x\ 794.84\ x\ 1000\ N}{1{,}500\ mm\ x\ 121.5\ mm}$$

$$f_{c,s} = 8.72\ MPa < 0.6f'_c = 12\ MPa \therefore OK$$

The service level stresses comply with the limits in ACI 440.2-23.

6l. Detailing of flexural FRP. (ACI 440.2-23 Eq. 14.1.3)

For simply supported beams, the flexural FRP should be extended beyond the point along the span at which the flexural demand is below the cracking moment by at least a distance equal to the development length,

$$l_{df} = \sqrt{\frac{NE_f t_f}{\sqrt{f'_c}}}$$

$$l_{df} = \sqrt{\frac{2\ x\ 168{,}000\ MPa\ x\ 1.2\ mm}{\sqrt{20\ MPa}}} = 300\ mm$$

Generally, it is recommended to extend the FRP to within 25 mm of the support to mitigate concrete cover delamination. If the FRP is terminated where the shear demand, V_u, is greater than 0.67V_c, the flexural FRP should be anchored to the beam with transverse clamping FRP U-wraps. The area of the U-wraps can be found per Eq. 14.1.2. (Section 14.1.2)

Tab. 18 – Procedure for design of FRP strengthening for shear with CF Fabrics

7a. Select the CF Fabric U600 for beam strengthening. (ESR-4774 ICC-ES Ev. Report)

The following properties are provided by fischer:

$$t_f = 1.02\ mm$$

$$E_f = 77\ GPa$$

$$f_{fu}* = 794\ MPa$$

$$\varepsilon_{fu}* = 1.26\ \% = 0.0126$$

7b. Check if the beam needs to be strengthened for the new loads. (ACI 440.2-23)

$$V_u = 234.6\ KN$$

$$\phi V_{n,existing} = \phi(V_c + V_s)_{existing} = 177.3\ KN$$

$\phi V_{n,existing} < V_u$, therefore shear strengthening is needed. (Eq. 11.3a)

The total shear strength of the strengthened section is expressed as,

$$\phi V_n = \phi\left(V_c + V_s + \psi V_f\right)$$ (Eq. 11.3b)

Which can be rewritten as,

$$\phi V_n = \phi(V_c + V_s) + \phi\psi V_f$$

The shear contribution required from the FRP is,

$$\phi\psi V_f = 234.6\ KN - 177.3\ KN = 57.3\ KN$$

7c. Check if the beam meets the minimum strength limits of Section 9.2.

ACI 440.2-23
Eq. 9.2

$V_{u,limit} = 165.1\ KN$

$\phi V_{n,existing} = 177.3\ KN$

$\phi V_{n,existing} > V_{u,limit}$, therefore FRP can be used.

7d. Calculate the FRP system design material properties.
Since the beam is inside a building and will be strengthened with a carbon FRP, per Table 9.4, an environmental reduction factor C_E = 0.95 is used.

ACI 440.2-23
Eq 9.4a
Eq. 9.4b

$f_{fu} = C_E f_{fu}* = \ 0.95\ x\ 794 = 754\ MPa$

$\varepsilon_{fu} = C_E \varepsilon_{fu}* = 0.95\ x\ 0.0126 = 0.0120$

7e. FRP strengthening requirements for unanchored U-wraps.
Assume that a 1-layer continuous U-wrap without fiber anchors is required to achieve the desired level of shear strengthening. The area of FRP contributing to the shear strength is calculated as,

ACI 440.2-23
Ea. 11.4b

$A_{fv} = 2Nt_f w_f$

For a continuous U-wrap, $w_f = d_{fv}$, as shown below in Fig. 28.

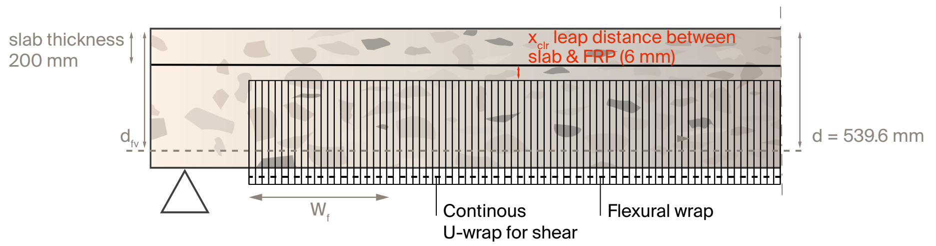


Fig. 28 – U-wrap layout

$w_f = d_{fv} = (d - t_{slab} - x_{clr})$

$w_f = d_{fv} = (539.6\ mm - 200\ mm - 6\ mm) = 333.6\ mm$

Thus, the area of FRP contributing to shear strength is,

$A_{fv} = 2\ sides\ x\ 1\ layer\ x\ 1.02\ mm\ x\ 333.6\ mm$

$A_{fv} = 680.5\ mm^2$

7f. Determine the effective strain and corresponding stress in the FRP.
The tensile stress in the FRP is given by,

ACI 440.2-23

$f_{fe} = E_f \varepsilon_{fe}$

Eq. 11.4d

Since the U-wraps are not anchored at the top, the effective strain is,

$\varepsilon_{fe} = \kappa_v \varepsilon_{fu}\ \leq 0.004$

Eq. 11.4.1.2a

The bond reduction coefficient, k_v, is calculated as,

$$\kappa_v = \frac{k_1 k_2 L_e}{11{,}900\varepsilon_{fu}} \leq 0.75$$

Eq. 11.4.1.2b

The active bond length, L_e, is calculated as,

$$L_e = \frac{23{,}300}{(Nt_f E_f)^{0.58}} = \frac{23{,}300}{(1\ x\ 1.02\ mm\ x\ 77{,}000\ MPa)^{0.58}} = 31.7\ mm$$

The factor to account for the concrete compressive strength, k_1, is calculated as,

$$k_1 = \left(\frac{f'_c}{27}\right)^{2/3} = \left(\frac{20\ MPa}{27}\right)^{2/3} = 0.82$$

Eq. 11.4.1.2c

The factor to account for the U-wraps, k_2, is calculated as, — Eq. 11.4.1.2d

$$k_2 = \frac{d_{fv} - L_e}{d_{fv}} = \frac{333.6\ mm - 31.7\ mm}{333.6\ mm} = 0.90$$

Note: for FRP bonded just on each side face of the beam, see Eq. 11.4.1.2e for k_2.

The resulting bond reduction coefficient, k_v, is calculated as, — Eq. 11.4.1.2e

$$\kappa_v = \frac{0.82\ x\ 0.90\ x\ 31.7\ mm}{11{,}900\ x\ 0.0114} = 0.179\ \leq 0.7$$

$$\varepsilon_{fe} = \kappa_v \varepsilon_{fu} = 0.179\ x\ 0.012 = 0.0021 \leq 0.004$$

$$f_{fe} = E_f \varepsilon_{fe} = \ 77{,}000\ MPa\ x\ 0.0021 = 161.7\ MPa$$

7g. Calculate the shear strength contribution of the FRP. — ACI 440.2-23 Eq. 11.4a

The shear contribution of the FRP is given by,

$$V_f = \frac{A_{fv} f_{fe} (sin\alpha + cos\alpha) d_{fv}}{s_f}$$

Where, α is the angle of the FRP U-wrap relative to the beam axis (ACI 440.2-23, Figure 11.4). For this example, α = 90 degrees.

As discussed in Step 5, for a continuous U-wrap, $s_f = d_{fv}$.

The shear contribution of the FRP is,

$$V_f = \frac{680.5\ mm^2\ x\ 161.7\ MPa\ x\ (sin 90 + cos 90)\ x\ 333.6\ mm}{333.6\ mm}$$

$$V_f = 110.0\ KN$$

7h. Calculate the total shear strength of the section. — ACI 440.2-23 Eq. 11.3b

$$\phi V_n = \phi(V_c + V_s + \psi V_f)$$

The strength reduction factor, ϕ, applied to the shear strength is per ACI 318, Chapter 21. — Section 11.3 ACI 318-19 Table 21.2.1

$$\phi = 0.75$$

An additional reduction factor, ψ, is applied to the shear contribution of FRP. This reduction factor is based on reliability analysis of different wrapping types.

For U-wraps, $\psi = 0.85$ — Table 11.3

The shear strength of strengthened section is,

$$\phi V_n = \phi(V_c + V_s) + \phi\psi V_f$$

$$\phi V_n = 177.3\ KN + 0.75\ x\ 0.85\ x\ 110.0\ KN$$

$$\phi V_n = 247.4\ KN$$

The required capacity is,

$$V_u = 234.6\ KN$$

Therefore,

$$\phi V_n > V_u$$

$$DCR = \frac{234.6\ KN}{247.4\ KN} = 0.95 < 1.0\ \therefore OK$$

A continuous unanchored U-wrap of 1 layer of U600 is adequate to achieve the desired level of shear strengthening.

The U-wrap shear strengthening can be discontinued at a distance away from the support where the existing shear strength, $V_{u,existing}$, can resist the factored shear demand, V_u.

The shear strength provided by the internal shear steel reinforcement and the external U-wraps cannot exceed,

$$V_s + V_f \leq 0.66\sqrt{f'_c} b_w d$$

$$0.66\sqrt{f'_c} b_w d = 0.66\sqrt{20\ Mpa}\ x\ 400\ mm\ x\ 539.6\ mm = 637\ KN$$

$$V_u = 234.6\ KN < 637\ KN\ \therefore OK$$

6

7i. FRP strengthening for U-wraps anchored with fiber anchors.
In Step 5, it was assumed that the U-wraps were not anchored with fiber anchors as allowed by ACI 440.2-23, Section 11.4.1.2. Per this provision, U-wraps that are anchored in accordance with Section 14.1.4 may be assumed to develop a maximum strain of 0.004 per Eq. 11.4.1.1.

ACI 440.2-23 Eq. 11.4.1.1

$\varepsilon_{fe} = 0.004 \leq 0.75\varepsilon_{fu}$

This means that the calculations for effective strain per Section 11.4.1.2 are not required and $\varepsilon_{fe} = 0.004$ can be used, with ψ = 0.85, as long as the fiber anchors are designed and detailed per 14.1.4.
Assume that 100 mm wide strips of 1 layer of U600 are spaced at 200 mm on center, as shown below in Fig. 29. Assume that the fiber anchors are placed such that the center of the hole is in 25 mm distance from the end of the U-wrap leg.

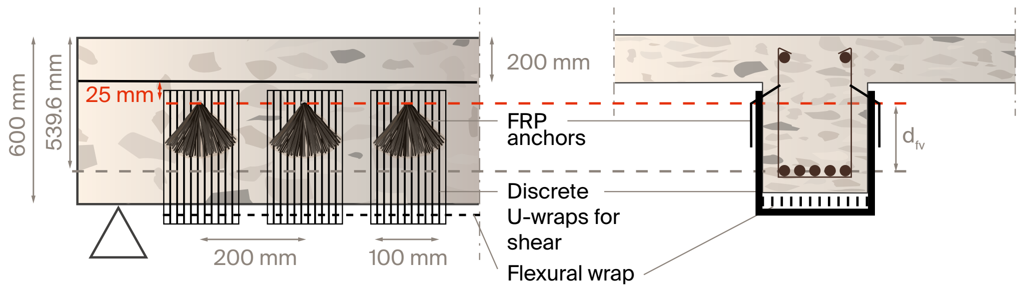


Fig. 29 – Assumed layout of anchored discrete U-wraps

For the anchored U-wraps,

Section 14.1.4

$d_{fv} = d - t_{slab} - x_{clr} - 25\ mm$

$d_{fv} = 539.6\ mm - 200\ mm - 6\ mm - 25\ mm = 308.6\ mm$

The area of FRP contributing to the shear strength is calculated as,

$A_{fv} = 2Nt_f w_f$

$A_{fv} = 2\ sides\ x\ 1\ x\ 1.02\ mm\ x\ 100\ mm = 204\ mm^2$

The tensile stress in the FRP is given by,

$f_{fe} = E_f \varepsilon_{fe} = 77{,}000\ MPa\ x\ 0.004 = 308\ MPa$

The shear contribution of the FRP is,

$$V_f = \frac{A_{fv} f_{fe}(sin\alpha + cos\alpha) d_{fv}}{s_f}$$

$$V_f = \frac{204\ mm^2\ x\ 308\ MPa\ x\ (sin90 + cos90)\ x\ 308.6\ mm}{200\ mm}$$

$V_f = 97\ KN$

The shear strength of strengthened section is,

$\phi V_n = \phi(V_c + V_s) + \phi\psi V_f$

$\phi V_n = 177.3\ KN + 0.75\ x\ 0.85\ x\ 97\ KN$

$\phi V_n = 239.1\ KN$

The required capacity is,

$V_u = 234.1\ KN$

Therefore,

$\phi V_n > V_u$

$$DCR = \frac{234.1\ KN}{239.1\ KN} = 0.98 < 1.0\ \therefore OK$$

Discrete anchored 100 mm strips of U-wraps of 1 layer of U600 spaced at 200 mm on center are adequate to achieve the desired level of shear strengthening. The U-wraps have to be anchored at the top of the legs on each side of the beam.

The fiber anchor requirements per Table 14.1.4 (SI).
For the U-wrap strips,
$NE_f t_f = 1\ layer\ x\ 1.02\ mm\ x\ 77{,}000\ MPa = 78.5\ KN/mm$

The spacing of the discrete U-wraps is 200 mm on center. Assume that the anchors will be installed into the beam at an angle β_{anc} between 110 degrees and 125 degrees.
From Table 14.1.4, using the first row for $s_{anc} = w_f \leq 100\ mm\ and\ 50\ < NE_f t_f \leq 100,$
$R_A = 1.25$

6

The area of the anchor required at each leg of the U-wrap is,
$A_{anc} \geq R_A(Nt_f s_{anc})$
Using, $s_{anc} = w_f = 100\ mm,$
$A_{anc} \geq 1.25(1\ layer\ x\ 1.02\ mm\ x\ 100\ mm)$
$A_{anc} \geq 127.5\ mm^2$

Section 14.1.4
Table 14.1.4(SI)

Eq. 14.1.4

Each anchor will have a minimum laminate area of 127.5 mm².
Note: If the anchor is installed straight up into the slab, i.e., 125 degrees < β_{anc} ≤ 180 degrees,
$R_A\ =\ 1.0$, i.e., the area of the anchor would be the same as the area of each leg of the U-wrap.

The resulting minimum diameter of the fiber anchor is,

$$d_{anc} \geq \sqrt{4\ x\ 127.5\ mm^2/_{\pi}} = 12.74\ mm$$

The anchor splay will have an enclosed angle of no more than 60 degrees and a minimum splay length of,
$r_{anc} = s_{anc} =\ w_f = 100\ mm$

Section 14.1.4
Table 14.1.4(SI)

The minimum embedment depth for the anchor will be,
$h_{anc} = larger\ of\ 150\ mm\ \&\ 7A_{anc}^{0.5}$
$h_{anc} = larger\ of\ 150\ mm\ \&\ 79\ mm$
$h_{anc} = 150\ mm$

Note: If the anchor is installed at right angle to the beam, i.e., 90 degrees < β_{anc} ≤ 110 degrees, $h_{anc} = 100\ mm.$

Each leg of each U-wrap will be anchored on each side of the beam. An additional patch of one layer of U600, with fibers perpendicular to the U-wrap, will be installed over the anchor splay. The final anchor layout is shown below in Fig. 30.

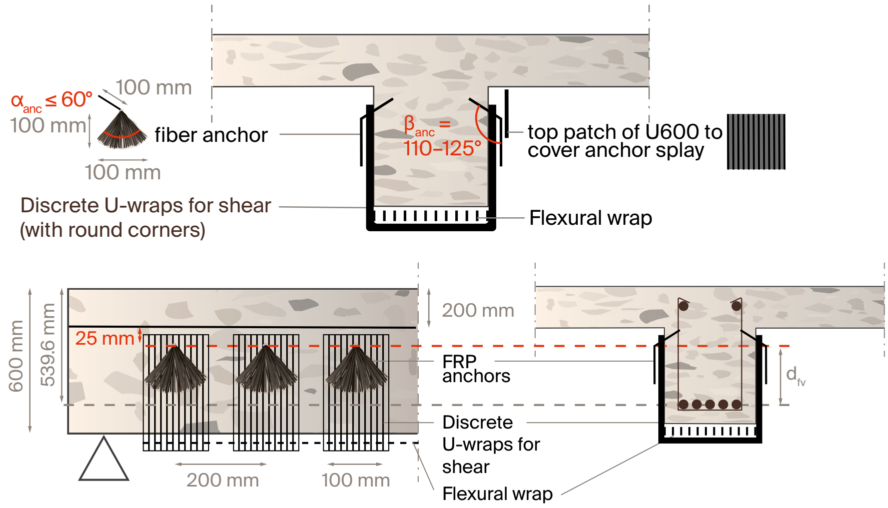


Fig. 30 – Final configuration of fiber anchors

See Section 14.1.4 for other detailing requirements.

6.2. Example 2: Flexural strengthening of an interior reinforced concrete slab with FRP

The continuous slab shown in Fig. 31 is to be strengthened with FRP to resist increased superimposed dead loads (due to new flooring, finishes, etc.) and live loads in the building.

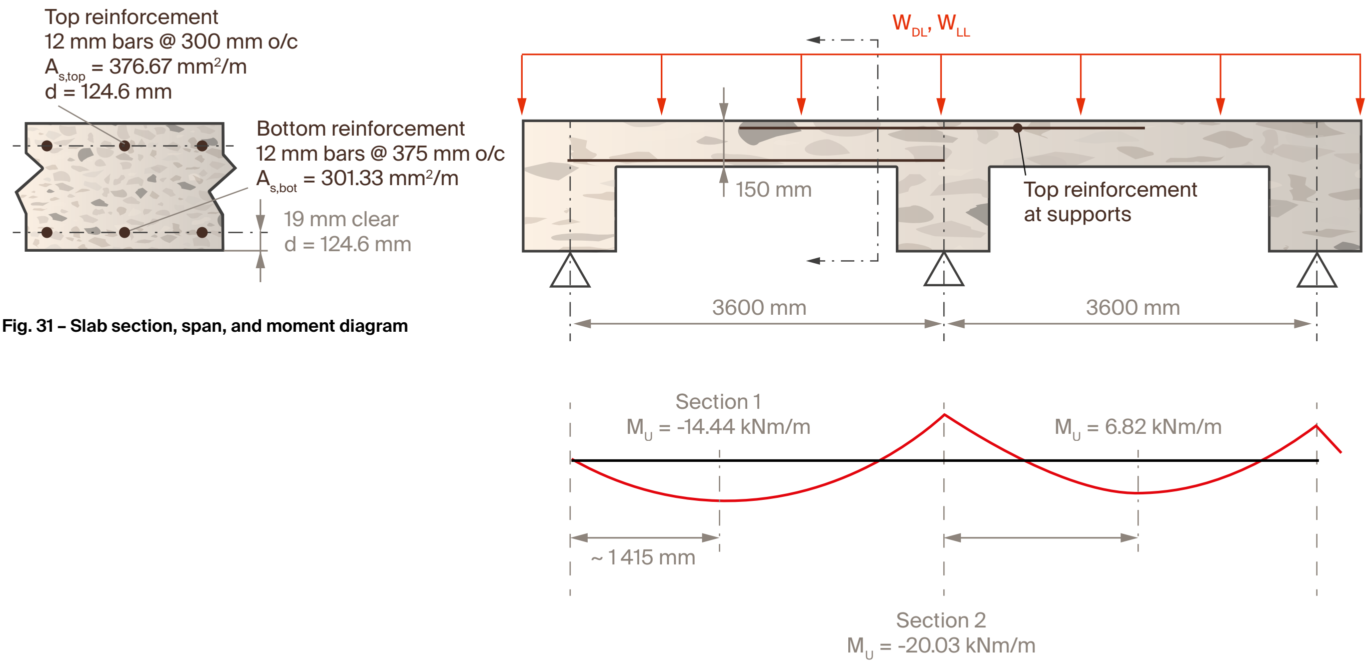


Fig. 31 – Slab section, span, and moment diagram

Tab. 19 – Existing slab material properties

Concrete compressive strength,
$f'_c = 20\ MPa$

Steel reinforcement yield strength,
$f_y = 420\ MPa$

Steel reinforcement elastic modulus,
$E_s = 200\ GPa$

The weight of the existing slab is 360 kg/m². The renovation of the building will increase the superimposed dead load (SDL) from 75 kg/m² to 200 kg/m² and the design live load (LL) from 250 kg/m² to 500 kg/m². As shown in Fig. 31, there are two critical sections at which the slab is to be checked. Tab. 20 shows the different gravity loads acting on the slab for a unit width of one meter of slab. Tab. 21 and Tab. 22 show the resulting flexural demands at Sections 1 and 2, respectively.

Tab. 20 – Existing and new gravity loads on 1 meter width of slab

Type		Existing	New
Dead Load (DL) w_{DL}		3.54 kN/m	3.54 kN/m
Superimposed Dead Load (SDL) w_{SDL}		0.74 kN/m	1.96 kN/m
Live Load (LL) w_{LL}		2.45 kN/m	4.91 kN/m
Existing capacity check limit (ACI 440.2-23 Eq. 9.2)	$1.1(w_{DL} + w_{SDL}) + 0.75w_{LL}$	NA	9.73 kN/m
Fire resistance check (ACI 440.2-23, Eq. 9.2.1)	$0.9(w_{DL} + w_{SDL}) + 0.50w_{LL}$	NA	7.41 kN/m
Factored design load	$1.2(w_{DL} + w_{SDL}) + 1.6w_{LL}$	9.06 kN/m	14.46 kN/m

Tab. 21 – Positive moment demands at Section 1 for 1 meter width of slab

Type		Existing	New
Dead load moment $M_{DL} + S_{DL}$		4.27 kNm	5.50 kNm
Live load moment M_{LL}		2.45 kNm	4.90 kNm
Service level moment M_S		6.72 kNm	10.40 kNm
Existing capacity limit check (ACI 440.2-23 Eq. 9.2)	$M_{u,limit} = 1.1(M_{DL} + M_{SDL}) + 0.75\ M_{LL}$	NA	9.72 kNm
Fire resistance check (ACI 440.2-23, Eq. 9.2.1)	$M_{u,fire} = 0.9(M_{DL} + M_{SDL}) + 0.50\ M_{LL}$	NA	7.40 kNm
Factor design moment M_u		9.05 kNm	14.44 kNm

6

Tab. 22 – Negative moment demands at Section 2 for 1 meter width of slab

Type		Existing	New
Dead load moment $M_{DL} + S_{DL}$		5.93 kNm	7.63 kNm
Live load moment M_{LL}		3.40 kNm	6.80 kNm
Service level moment M_S		9.33 kNm	14.42 kNm
Existing capacity limit check (ACI 440.2-23 Eq. 9.2)	$M_{u,limit} = 1.1(M_{DL} + M_{SDL}) + 0.75\ M_{LL}$	NA	13.49 kNm
Fire resistance check (ACI 440.2-23, Eq. 9.2.1)	$M_{u,fire} = 0.9(M_{DL} + M_{SDL}) + 0.50\ M_{LL}$	NA	10.26 kNm
Factor design moment M_u		12.55 kNm	20.03 kNm

The existing beam section at midspan has the following capacities:

Section 1: $\phi M_{n,positive} = 13.60\ KNm$
Section 2: $\phi M_{n,negative} = 16.90\ KNm$

Tab. 23 and Tab. 24 detail the design of the FRP strengthening for flexure, per the provisions of ACI 440.2-23, with CF Fabrics at sections 1 and 2, respectively.

ACI 440.2-23 has a strengthening limit based on the fire-resistance of the existing structural member. Per Section 9.2.1, an evaluation is required to ensure that the strengthened slab will not collapse in a fire. In Tab. 21 and Tab. 22, the values for $M_{u,fire}$ are calculated per Eq. 9.2.1. The capacity of the existing slab must be calculated without FRP and based on reduced material properties corresponding to the fire exposure period required for the fire-resistance rating of the building. It is important to note that the existing slab is assumed to fulfill the fire-resistance rating requirement. By engineering judgement, it can be determined that the level of strengthening required is acceptable to meet this requirement. If the required strengthening is significantly higher, the fire-resistance of the beam without FRP and with reduced material properties due to fire exposure should be explicitly checked. More details and an illustrative example can be found in ACI PRC 440.10-21, Fire Resistance of FRP-Strengthened Concrete Members-Technote.

Tab. 23 – Procedure for design of FRP strengthening for positive moment at section 1 with CF Fabric

5a. Select the CF Fabric U600 for flexural strengthening.
The following properties are provided by fischer:

ESR-4774 ICC-ES Ev. Report

$t_f = 1.02\ mm$

$E_f = 77\ GPa$

$f_{fu}* = 794\ MPa$

$\varepsilon_{fu}* = 1.26\ \% = 0.0126$

5b. Does the slab section 1 need to be strengthened for the new loads?

ACI 440.2-23 Eq. 10.1

$M_u = 14.44\ KNm/m$

$\phi M_{n,existing} = 13.60\ KNm/m$

$\phi M_{n,existing} < M_u$, therefore flexural strengthening is needed.

5c. Does the beam meet the minimum strength limits of Section 9.2?

ACI 440.2-23 Eq. 9.2

$M_{u,limit} = 7.40\ KNm/m$

$\phi M_{n,existing} = 13.60\ KNm/m$

$\phi M_{n,existing} > M_{u,limit}$, therefore FRP can be used.

5d. Initial assumption on FRP layering needed for flexural strengthening.
Assume that two 100 mm strips, placed per 1 m width of the slab at a spacing of 500 mm, are required at the slab bottom to achieve the desired level of flexural strengthening.

$A_f = 1\ layer\ x\ 2\ strips\ x\ 100\ mm\ x\ 1.02\ mm = 204\ mm^2$

Effective depth for the FRP, that is the distance from the extreme compression fiber to the centroid of the FRP, is,

$d_f = 150\ mm + (1\ x\ 1.02\ mm)/2 = 150.5\ mm$

5e. Calculate the FRP system design material properties.
Since the slab is inside a building and will be strengthened with a carbon FRP, per Table 9-4, an environmental reduction factor of 0.95 is used.

ACI 440.2-23 Eq 9.4a Eq. 9.4b

$f_{fu} = C_E f_{fu}* = 0.95\ x\ 794 = 754{,}3\ MPa$

$\varepsilon_{fu} = C_E \varepsilon_{fu}* = 0.95\ x\ 0.0126 = 0.0120$

5f. Determine the existing substrate strain.
The slab will have some existing level of strain due to existing loads. This strain level at the bottom of the slab is preexisting and cannot be mobilized in the FRP. Assuming that at the time of the strengthening only the dead and superimposed dead loads are acting on the slab, the existing (initial) substrate strain can be calculated from elastic analysis of the section using cracked section properties. The following equation can be used:

ACI 440.2-23 Ex. 16.3, Table 16.3c

$$\varepsilon_{bi} = \frac{M_{DL}(d_f - kd)}{I_{cr} E_c}$$

The common approach is to use a cracked transformed section wherein the top steel reinforcement area is represented in terms of concrete area by multiplying it by the modular ratio, E_s/E_c.

$E_c = 21{,}174\ MPa$

$E_s = 200{,}000\ MPa$

$n = E_s/E_c = 9.44$

The area of the bottom reinforcement is:
$A_{s,bot} = 113\ mm^2\ x\ (1000\ mm\ /\ 375\ mm)\ =\ 301.33\ mm^2\ per\ meter\ width$

The transformed reinforcement area of the bottom reinforcement is:
$A_{sT,bot} = nA_{s,bot} = 301.33\ x\ 9.44 = 2{,}846\ mm^2$

The resulting cracked transformed section is shown in Fig. 32

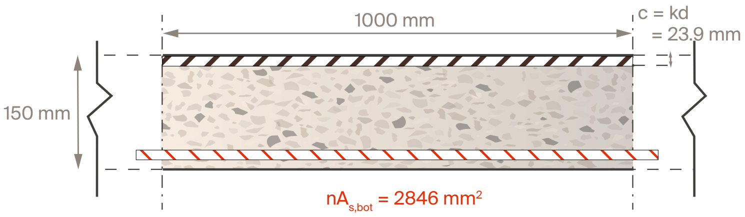


Fig. 32 – Transformed cracked section for elastic analysis

The depth of the neutral axis can be determined by summing the moments of the areas relative to the neutral axis. If we assume the depth to the neutral axis to be c, as shown in Fig. 32, and equate $\Sigma A\overline{y} = 0$, the resulting quadratic equation is:
$500c^2 + 2{,}846c - (2{,}846\ mm^2\ x\ 124.6\ mm) = 0$

Note: See any textbook on concrete design for development of cracked transformed sections.

Solving the resultant quadratic equation,
$c = kd = depth\ of\ the\ neutral\ axis = 23.9\ mm$

The transformed cracked section modulus,
$I_{cr} = 3.341\ x\ 10^7$

There the initial strain at the concrete before FRP strengthening can be calculated as,

$$\varepsilon_{bi} = \frac{5.50\ x\ 10^3\ KNmm\ (150.5 - 23.9)}{3.341\ x\ 10^7\ x\ 21.174\ GPa} = 0.00098$$

5g. Determine the effective strain in the FRP at which debonding may occur. ACI 440.2-23 Eq. 10.1.1
The strain in the FRP has to be limited to the strain at which debonding will occur. The effective strain, ε_{fd}, can be calculated as,

$$\varepsilon_{fd} = 0.41\sqrt{\frac{f'_c}{NE_f t_f}} \leq 0.9\varepsilon_{fu}$$

$$\varepsilon_{fd} = 0.41\sqrt{\frac{20\ MPa}{1\ x\ 77{,}000\ MPa\ x\ 1.02\ mm}} \leq 0.0108$$

$$\varepsilon_{fd} = 0.00654\ \leq 0.0108$$

5h. Calculate the state of strain in the section.
Equilibrium in the section depends on the failure mode at the ultimate limit state.

- If concrete crushing controls, equilibrium is checked when the strain at the extreme compression fiber, ε_{cu}, reaches the maximum usable value of 0.003. This limit is typical for design per ACI 318 & 440.2-23.
- If debonding of the FRP controls, equilibrium is checked when the strain in the FRP reaches ε_{fd}.

Assume that the depth of the neutral axis (NA) below the top of the slab is at 20 % of the total beam depth.
$c = 0.2\ x\ 150\ mm = 30\ mm$

Assuming that the compressive strain at the extreme compression fiber reaches 0.003, calculate the strain in the FRP, — Eq. 10.2.5

$$\varepsilon_{fe} = \varepsilon_{cu}\left(\frac{d_f - c}{c}\right) - \varepsilon_{bi} \leq \varepsilon_{fd}$$

$$\varepsilon_{fe} = 0.003\left(\frac{150.5\ mm - 30\ mm}{30\ mm}\right) - 0.00098 \leq 0.00654$$

$$\varepsilon_{fe} = 0.01107 > 0.00654$$

It is observed that the FRP will debond before the strain reaches 0.003 at the extreme compression fiber. This indicates that FRP debonding will control the failure mode.

6

For design, the strain in the FRP is limited to ε_{fd},

$$\varepsilon_{fe} = \varepsilon_{fd} = 0.00654$$

Calculate the strain at the extreme compression fiber at ε_{fe}. Rearranging Eq. 10.2.5,

$$\varepsilon_c = (\varepsilon_{fe} + \varepsilon_{bi})\left(\frac{c}{d_f - c}\right)$$

$$\varepsilon_c = (0.00654 + 0.00098)\left(\frac{30\ mm}{150.5\ mm - 30\ mm}\right)$$

$$\varepsilon_c = 0.00187 \leq 0.003$$

The strain at which concrete is no longer linearly elastic is taken as $0.85f'_c/E_c = 0.00080$. Since the strain at the extreme compression fiber is greater than this value, it can be considered that the concrete compression block will experience a parabolic distribution of strain.

Calculate the strain in the bottom reinforcement.

Using similar triangles, the strain at in the bottom steel can be determined as, — Eq. 10.2.10a

$$\varepsilon_s = (\varepsilon_{fe} + \varepsilon_{bi})\left(\frac{d - c}{d_f - c}\right)$$

$$\varepsilon_s = (0.00654 + 0.00098)\left(\frac{124.6\ mm - 30\ mm}{150.5\ mm - 30\ mm}\right)$$

$$\varepsilon_s = 0.0059 > \varepsilon_y = \frac{f_y}{E_s} = \frac{420\ MPa}{200,000\ MPa} = 0.0021$$

The bottom reinforcing steel is yielding.

Using the strains calculated above, determine the forces in the different components, and check equilibrium.

5i. Check equilibrium in the section. — ACI 440.2-23

Internal equilibrium can be checked using, — Eq. 10.2.10c

$$\alpha_1 f'_c \beta_1 bc + A_{s,top} f_{s'} = A_{s,bot} f_s + A_f f_{fe}$$

The parameters defining the rectangular stress block for the nonlinear distribution of stress in the concrete compression block are,

$$\alpha_1 = \frac{3\varepsilon'_c\varepsilon_c - \varepsilon_c^2}{3\beta_1\varepsilon_c'^2}$$

$$\beta_1 = \frac{4\varepsilon'_c - \varepsilon_c}{6\varepsilon'_c - 2\varepsilon_c}$$

Where, ε'_c corresponds to the strain at f'_c,

$$\varepsilon'_c = \frac{1.7f'_c}{E_c} = \frac{1.7\ x\ 20\ MPa}{21{,}174\ MPa} = 0.00161$$

$$\beta_1 = \frac{4\ x\ 0.00161 - 0.00187}{6\ x\ 0.00161 - 2\ x\ 0.00187} = 0.772$$

$$\alpha_1 = \frac{3\ x\ 0.00161\ x\ 0.00187 - 0.00187^2}{3\ x\ 0.772\ x\ 0.00161^2} = 0.922$$

The compressive component in concrete,
$C = \alpha_1 f'_c \beta_1 bc = \ 0.922\ x\ 20\ MPa\ x\ 0.772\ x\ 1{,}000\ mm\ x\ 30\ mm$
$C = 427.07\ KN$

Tensile force in the bottom reinforcement,
$T_{s,bot} = A_{s,bot} x\ Min(\varepsilon_s E_s, F_y)$
$T_{s,bot} = \ 301.3\ mm^2\ x\ Min\ (0.0059\ x\ 200{,}000\ MPa, 420\ MPa)$
$T_{s,bot} = \ 126.6\ KN$

Tensile force in the FRP,
$T_f = A_f \varepsilon_{fe} E_f = \ 204\ mm^2\ x\ 0.00654\ \text{x}\ \ 77{,}000\ \text{MPa}$
$T_f = \ 102.8\ KN$

Check equilibrium,
$C = 427.07\ KN$
$T_{s,bot} + T_f = 126.6\ KN + 102.7\ KN = 229.3\ KN$
$C \neq \ T_{s,bot} + T_f$

Therefore, equilibrium is not satisfied.

Since the compressive force is larger than the tensile force, the depth of the NA is smaller than the initial assumption.
After iteration, the depth of the NA to achieve equilibrium is calculated as,
$c \ = \ 20.52\ mm$

Recalculate the strains in the section using FRP debonding as the controlling failure mode at,
$\varepsilon_{fe} = 0.00654$
$\varepsilon_c = 0.00119 \leq 0.003$
$\varepsilon_s = 0.0061 > \varepsilon_y = 0.0021$

The different force components are calculated as below:
$\beta_1 = 0.721$
$\alpha_1 = 0.774$
$C = 229.3\ KN$
$T_{s,bot} = \ 126.6\ KN$
$T_f = \ 102.8\ KN$
$T_{s,bot} + T_f = 126.6\ KN + 102.8\ KN = 229.4\ KN$
$C = \ T_{s,bot} + T_f$
Equilibrium is satisfied.

5j. Determine the flexural capacity of the strengthened section.
The flexural capacity provided by each force component is calculated as the product of the force times the level from the extreme compression fiber, i.e., top of the slab section.

Component	Force F	Lever arm $\bar{y}$	Moment M = $F_{\bar{y}}$
Concrete compression	-229.6 kN	$L_c = \beta_1 c/2$ L_c = 0.721 × 20.52 mm/2 L_c = 7.4 mm	-1 697 kNmm
Bottom reinf. compression	126.6 kN	d = 124.6 mm	15 769 kNmm
FRP tension	102.8 kN	d_f = 150.51 mm	15 472 kNmm

The negative sign for the force indicates compression while the positive sign indicates tension. ACI 440.2-23
From Step 5h, the strain in the bottom reinforcement is calculated as 0.0060. Eq. 10.2.7
Since $\varepsilon_t = \varepsilon_s \geq 0.005$ Section 10.2.10
$\phi = 0.90$

An additional reduction factor of $\varphi = 0.85$ is applied to the FRP's effect on the flexural capacity.
The final moment capacity of the strengthened section is,
$\phi M_n = 0.90(15{,}769\ KNmm + 0.85\ x\ 15{,}472\ KNmm - 1{,}697\ KNmm)$
$\phi M_n = 24{,}501\ KNmm$
$\phi M_n = 24.50\ KNm$

6

The required capacity is,
$M_u = 14.44\ KNm$

Therefore,
$\phi M_n > M_u$
$$DCR = \frac{14.44\ KNm}{24.50\ KNm} = 0.59 < 1.0\ \therefore OK$$
Two 100 mm wide strips of 1 layer of U 600 per meter width are adequate to achieve the desired level of flexural strengthening.
Provide 100 mm wide strips of 1 layer of U 600 at a spacing of 500 mm on center at the bottom of the slab.

Note: The required flexural strength could be achieved with 100 mm strips of U 600 at a spacing of 1000 mm on center. ACI PRC-440.2-23 does not provide a maximum spacing for flexural FRP. The user can investigate the optimization of the design if desired.

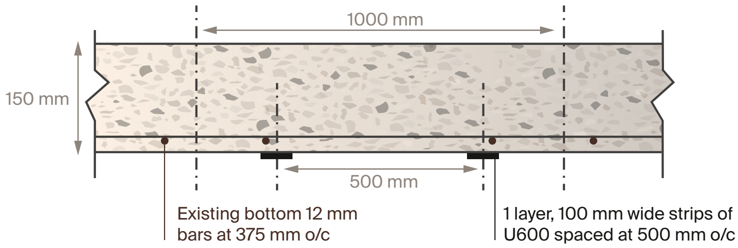


Fig. 33 – Final configuration of positive moment strengthening

5k. Check the service level stresses in the reinforcing steel, concrete and FRP. ACI 440.2-23
It is important to ensure that the strengthened slab will fulfill serviceability requirements, such as deflections and crack widths. Such checks are performed using a transformed section analysis, similar to Step 5f. The stresses in the reinforcing steel and concrete are restricted to, Eq. 10.2.8a Eq. 10.2.8b
$f_{s,s} \leq 0.80 f_y$
$f_{c,s} \leq 0.60 f'_c$

The stress in bottom reinforcing steel under service loads can be found by,
$$f_{s,s} = \frac{\left[M_s + \varepsilon_{bi} A_f E_f \left(d_f - {kd}/{3}\right)\right](d - kd) E_s}{\left[A_s E_s \left(d - {kd}/{3}\right)(d - kd) + A_f E_f \left(d_f - {kd}/{3}\right)(d_f - kd)\right]}$$
Eq. 10.2.10.1

To calculate kd, a cracked transformed section is developed like in Step 5f. Since FRP is present at the bottom of the strengthened section, a transformed area of FRP has to be included.
The resulting section will resemble the figure in Step 5f with an additional transformed FRP area, $nA_f = (E_f/E_c)A_f$ at the bottom. From such a transformed section,
$c = kd = 27.16\ mm$
For a service level moment $M_s = 10.4\ KNm$
$$f_{s,s} = \frac{\left[10.4\ KNm + 0{,}00095 x 204\ mm^2 x\ 77\ GPa\left(150.5\ mm - {27.16\ mm}/{3}\right) x 10^{-3}\right] x \ (124.6\ mm - 27.16\ mm) x\ 200\ GPa\ x\ 10^{-3}}{\begin{bmatrix} 301.33\ mm^2 x 200\ GPa\left(124.6\ mm - {27.16\ mm}/{3}\right)(124.6\ mm - 27.16\ mm) + \\ 204\ mm^2 x 77\ GPa\left(150.5\ mm - {27.16\ mm}/{3}\right)(150.5\ mm - 27.16\ mm) \end{bmatrix} x 10^{-9}}$$

$f_{s,s} = 255.95\ MPa \leq 0.8 f_y = 0.8\ x\ 420\ MPa = 336.2\ MPa\ \therefore OK$

The stress in the FRP under service loads can be calculated by,

$$f_{f,s} = f_{s,s}\left(\frac{E_f}{E_s}\right)\frac{d_f - kd}{d - kd} - \varepsilon_{bi}E_f$$

$$f_{f,s} = 255.95\ MPa\left(\frac{77\ GPa}{200\ GPa}\right)\frac{150.5\ mm - 27.16\ mm}{124.6\ mm - 27.16\ mm} - 0.00095\ x\ 77{,}000\ MPa$$

$$f_{f,s} = 51.58\ MPa \leq 0.55 f_{fu} = 0.55\ x\ 754.3\ MPa = 414.9\ MPa\ \therefore OK$$

The $0.55f_{fu}$ limit is intended to avoid creep rupture of the FRP under sustained service loads. — Eq. 10.2.10.2

The stress at the extreme compression fiber in concrete can be found by,

$$f_{c,s} = {}^{C}/_{(b\ x\ kd)/2} = 2C/(b\ x\ kd)$$

To achieve equilibrium, the compressive force is, — Section 10.2.9, Table 10.2.9

$$C = f_{s,s}A_{s,bot} + f_{f,s}A_f$$

$$C = 255.95\ MPa\ x\ 301.33\ mm^2 + 51.58\ MPa\ x\ 204\ mm^2$$

$$C = 87.6\ KN$$

$$f_{c,s} = {}^{2\ x\ 87.6\ x\ 1000\ N}/_{(1{,}000\ mm\ x\ 27.16\ mm)}$$

$$f_{c,s} = 6.45\ MPa \leq 0.6f'_c = 0.6\ x\ 20\ MPa = 12\ MPa \therefore OK$$

The service level stresses comply with the limits in ACI 440.2-23.

5l. Detailing of flexural FRP. — ACI 440.2-23 Eq. 14.1.3

For simply supported spans, the flexural FRP should be extended beyond the point along the span at which the flexural demand is below the cracking moment by at least a distance equal to the development length,

$$l_{df} = \sqrt{\frac{NE_f t_f}{\sqrt{f'_c}}}$$

$$l_{df} = \sqrt{\frac{1\ x\ 77{,}000\ MPa\ x\ 1.02\ mm}{\sqrt{20\ MPa}}} = 132.5\ mm$$

Generally, it is recommended to extend the FRP to within 25 mm of the support to mitigate concrete cover delamination.

Tab. 24 – Procedure for design of FRP strengthening for negative moment at section 2 with CF Fabric

6a. The following properties are provided by fischer: — ESR-4774 ICC-ES Ev. Report

$$t_f = 1.02\ mm$$
$$E_f = 77\ GPa$$
$$f_{fu}* = 794\ MPa$$
$$\varepsilon_{fu}* = 1.26\ \% = 0.0126$$

6b. Does the slab section 1 need to be strengthened for the new loads? — ACI 440.2-23 Eq. 10.1

$M_u = 20.03\ KNm/m$

$\phi M_{n,existing} = 16.90\ KNm/m$

$\phi M_{n,existing} < M_u$, therefore flexural strengthening is needed.

6c. Does the beam meet the minimum strength limits of Section 9.2? — ACI 440.2-23 Eq. 9.2

$M_{u,limit} = 13.49\ KNm/m$

$\phi M_{n,existing} = 16.90\ KNm/m$

$\phi M_{n,existing} < M_u$, therefore FRP can be used

6d. Initial assumption on FRP layering needed for flexural strengthening.
Assume that two 100 mm strips, placed per 1 m width of the slab at a spacing of 500 mm, are required at the top of the slab to achieve the desired level of flexural strengthening.
$A_f = 1\ layer\ x\ 2\ strips\ x\ 100\ mm\ x\ 1.02\ mm = 204\ mm^2$

Under a negative moment, compression occurs at the bottom of the slab. Effective depth for the FRP, that is the distance from the extreme compression fiber to the centroid of the FRP, is,
$d_f = 150\ mm + (1\ x\ 1.02\ mm)/2 = 150.5\ mm$

6

6e. Calculate the FRP system design material properties.
Since the slab is inside a building and will be strengthened with a carbon FRP, per Table 9.4, an environmental reduction factor of 0.95 is used.

ACI 440.2-23
Eq 9.4a
Eq. 9.4b

$f_{fu} = C_E f_{fu} * = 0.95\ x\ 794 = 754.3\ MPa$
$\varepsilon_{fu} = C_E \varepsilon_{fu} * = 0.95\ x\ 0.0126 = 0.012$

6f. Determine the existing substrate strain.
Assuming that at the time of the strengthening only the dead and superimposed dead loads are acting on the slab, the existing (initial) substrate strain at the top of the slab can be calculated from elastic analysis of the section using cracked section properties. The following equation can be used:

ACI 440.2-23
Ex. 16.3, Table 16.3c

$$\varepsilon_{bi} = \frac{M_{DL}(d_f - kd)}{I_{cr} E_c}$$

The common approach is to use a cracked transformed section wherein the top steel reinforcement area is represented in terms of concrete area by multiplying it by the modular ratio, E_s/E_c.
$E_c = 21{,}174\ MPa$
$E_s = 200{,}000\ MPa$
$n = E_s/E_c = 9.44$

The area of the top reinforcement is:
$A_{s,top} = 113\ mm^2\ x\ (1000\ mm\ /\ 300\ mm) = 376.67\ mm^2\ per\ meter\ width$

The transformed reinforcement area of the top reinforcement is:
$A_{sT,top} = nA_{s,top} = 376.67\ x\ 9.44 = 3{,}556\ mm^2$

Using a process similar to Step 5f,
$c = kd = depth\ of\ the\ neutral\ axis = 26.4\ mm$

The transformed cracked section modulus,
$I_{cr} = 4.043\ x\ 10^7$

There, the initial strain at the concrete before FRP strengthening can be calculated as,

$$\varepsilon_{bi} = \frac{7.63\ x\ 10^3\ KNmm\ (150.5 - 26.4)}{4.043\ x\ 10^7\ x\ 21.174\ GPa} = 0.0011$$

6g. Determine the effective strain in the FRP at which debonding may occur.
The strain in the FRP has to be limited to the strain at which debonding will occur. The effective strain, ε_{fd}, can be calculated as,

ACI 440.2-23
Eq. 10.1.1

$$\varepsilon_{fd} = 0.41\sqrt{\frac{f'_c}{NE_f t_f}} \leq 0.9\varepsilon_{fu}$$

$$\varepsilon_{fd} = 0.41\sqrt{\frac{20\ MPa}{1\ x\ 77{,}000\ MPa\ x\ 1.02\ mm}} \leq 0.0103$$

$$\varepsilon_{fd} = 0.00654 \leq 0.0103$$

6h. Calculate the state of strain in the section.
For negative moment demand, compression will occur at the bottom of the slab and tension will occur at the top of the slab. Equilibrium in the section depends on the failure mode at the ultimate limit state.

- If concrete crushing controls, equilibrium is checked when the strain at the extreme compression fiber, ε_{cu}, reaches the maximum usable value of 0.003. This limit for ultimate compression strain is typical for design per ACI 318 & 440.2-23.
- If debonding of the FRP controls, equilibrium is checked when the strain in the FRP reaches ε_{fd}.

Chapter 2, Notations & Definitions

After iteration, the depth of the NA to achieve equilibrium is calculated as,
$c = 21.96\ mm$

Assuming that the compressive strain at the extreme compression fiber (at the top of the slab) reaches 0.003, calculate the strain in the FRP,

Eq. 10.2.5

$$\varepsilon_{fe} = \varepsilon_{cu}\left(\frac{d_f - c}{c}\right) - \varepsilon_{bi} \leq \varepsilon_{fd}$$

$$\varepsilon_{fe} = 0.003\left(\frac{150.5\ mm - 21.96\ mm}{21.96\ mm}\right) - 0.0011 \leq 0.00623$$

$$\varepsilon_{fe} = 0.0165 > 0.00623$$

This indicates that the FRP will debond before the strain reaches 0.003 at the extreme compression fiber. This indicates that FRP debonding will control the failure mode.
For design, the strain in the FRP is limited to ε_{fd},
$\varepsilon_{fe} = \varepsilon_{fd} = 0.00654$

Calculate the strain at the extreme compression fiber at ε_{fe}. Rearranging Eq. 10.2.5,

$$\varepsilon_c = (\varepsilon_{fe} + \varepsilon_{bi})\left(\frac{c}{d_f - c}\right)$$

$$\varepsilon_c = (0.00654 + 0.0011)\left(\frac{21.96\ mm}{150.5\ mm - 21.96\ mm}\right)$$

$$\varepsilon_c = 0.00131 \leq 0.003$$

The strain at which concrete is no longer linearly elastic is taken as $0.85f'_c/E_c = 0.00080$. Since the strain at the extreme compression fiber is greater than this value, it can be considered that the concrete compression block will experience a parabolic distribution of strain.

Eq. 10.2.10a

Calculate the strain in the top reinforcement.
Using similar triangles, the strain in the top steel can be determined as,

$$\varepsilon_s = (\varepsilon_{fe} + \varepsilon_{bi})\left(\frac{d - c}{d_f - c}\right)$$

$$\varepsilon_s = (0.00654 + 0.0011)\left(\frac{124.6\ mm - 21.96\ mm}{150.51\ mm - 21.96\ mm}\right)$$

$$\varepsilon_s = 0.0061 > \varepsilon_y = \frac{f_y}{E_s} = 0.0021$$

The top reinforcing steel is yielding.
Using the strains calculated above, determine the forces in the different components, and check equilibrium.

6i. Check equilibrium in the section.
Internal equilibrium can be checked using,
$\alpha_1 f'_c \beta_1 bc = A_{s,bot} f_s + A_f f_{fe}$

ACI 440.2-23
Eq. 10.2.10c

The parameters defining the rectangular stress block for the nonlinear distribution of stress in the concrete compression block are,

ACI 440.2-23 Ex. 16.3, Table 16.3c

$$\alpha_1 = \frac{3\varepsilon_c^{'}\varepsilon_c - \varepsilon_c^2}{3\beta_1 \varepsilon_c^{'2}}$$

$$\beta_1 = \frac{4\varepsilon_c^{'} - \varepsilon_c}{6\varepsilon_c^{'} - 2\varepsilon_c}$$

Where, ε'_c corresponds to the strain at f'_c,

$$\varepsilon'_c = \frac{1.7 f'_c}{E_c} = \frac{1.7\ x\ 20\ MPa}{21{,}174\ MPa} = 0.00161$$

$$\beta_1 = \frac{4\ x\ 0.00161 - \ 0.00131}{6\ x\ 0.00161 - 2\ x\ 0.00131} = 0.729$$

$$\alpha_1 = \frac{3\ x\ 0.00161\ x\ 0.00131 - 0.00131^2}{3\ x\ 0.729\ x\ 0.00161^2} = 0.815$$

The compressive component in concrete,

$$C = \alpha_1 f'_c \beta_1 bc = \ 0.815\ x\ 20\ MPa\ x\ 0.729\ x\ 1{,}000\ mm\ x\ 21.96\ mm$$

$$C = 260.96\ KN$$

Tensile force in the top reinforcement,

$$T_{s,top} = A_{s,bot} x\ Min(\varepsilon_s E_s, F_y)$$

$$T_{s,top} = \ 376.7\ mm^2\ x\ Min\ (0.0061\ x\ 200{,}000\ MPa, 420\ MPa)\ x\ 10^{-3}$$

$$T_{s,top} = \ 158.2\ KN$$

Tensile force in the FRP,

$$T_f = A_f \varepsilon_{fe} E_f = \ 204\ mm^2\ x\ 0.00654\ \text{x}\ \ 77{,}000\ \text{MPa}\ x\ 10^{-3}$$

$$T_f = \ 102.8\ KN$$

Check equilibrium,

$$C = 260.96\ KN$$

$$T_{s,top} + T_f = 158.2\ KN + 102.8\ KN = 261.0\ KN$$

$$C = \ T_{s,top} + T_f$$

Therefore, equilibrium is satisfied.

6j. Determine the flexural capacity of the strengthened section.
The flexural capacity provided by each force component is calculated as the product of the force times the level from the extreme compression fiber, i.e., top of the beam section.

Component	Force	Lever arm y	Moment M = F_y
Concrete compression	-261.0 kN	$L_c = \beta_1 c/2$ L_c = 0.729 × 21.96 mm/2 L_c = 8.0 mm	-2 088 kNmm
Top reinf. compression	158.2 kN	d = 124.6 mm	19 712 kNmm
FRP tension	102.8 kN	d_f = 150.5 mm	15 471 kNmm

The negative sign for the force indicates compression while the positive sign indicates tension.
From Step 6h, the strain in the top reinforcement is calculated as 0.0061.
Since $\varepsilon_t \ = \ \varepsilon_s \geq 0.005$
$\phi = 0.90$

ACI 440.2-23 Eq. 10.2.7

An additional reduction factor of φ= 0.85 is applied to the FRP's effect on the flexural capacity. Section 10.2.10

The final moment capacity of the strengthened section is,

$\phi M_n = 0.90(19{,}712\ KNmm + 0.85\ x\ 15{,}471\ KNmm - 2{,}0888\ KNmm)$

$\phi M_n = 27{,}697\ KNmm$

$\phi M_n = 27.70\ KNm$

The required capacity is,

$M_u = 20.03\ KNm$

Therefore,

$\phi M_n > M_u$

$$DCR = \frac{20.03\ KNm}{27.70\ KNm} = 0.72 < 1.0\ \therefore OK$$

6

Two 100 mm wide strips of 1 layer of U600 per meter width are adequate to achieve the desired level of flexural strengthening. Provide 100 mm wide strips of 1 layer of U600 at a spacing of 500 mm on center at the top of the slab.

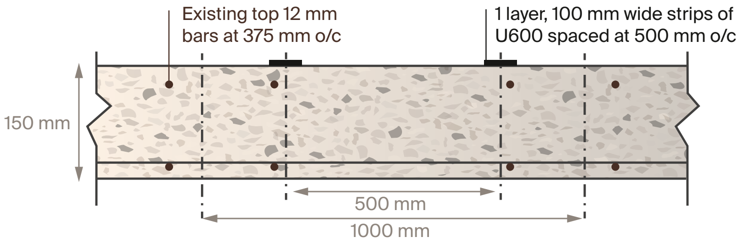


Fig. 34 – Final configuration of negative moment strengthening

6k. Check the service level stresses in the reinforcing steel, concrete and FRP.

ACI 440.2-23 Eq. 10.2.8a Eq. 10.2.8b

It is important to ensure that the strengthened slab will fulfill serviceability requirements, such as deflections and crack widths. Such checks are performed using a transformed section analysis, similar to Step 6f. The stresses in the reinforcing steel and concrete are restricted to,

$f_{s,s} \leq 0.80 f_y$

$f_{c,s} \leq 0.60 f'_c$

The stress in the bottom reinforcing steel under service loads can be found by, Eq. 10.2.10.1

$$f_{s,s} = \frac{\left[M_s + \varepsilon_{bi} A_f E_f \left(d_f - {}^{kd}/_3\right)\right](d - kd)E_s}{\left[A_s E_s \left(d - {}^{kd}/_3\right)(d - kd) + A_f E_f \left(d_f - {}^{kd}/_3\right)(d_f - kd)\right]}$$

To calculate kd, a cracked transformed section is developed like in Step 6f. Since FRP is present at the top of the strengthened section, a transformed area of FRP has to be included. The transformed FRP area is calculated as $nA_f = \left(\frac{E_f}{E_c}\right) A_f$.

From such a transformed section,

$c = kd = 29.3\ mm$

For a service level moment $M_s = 14.42\ KNm$

$$f_{s,s} = \frac{\begin{gathered}[14.42\ KNm + 0.0011x204\ mm^2 x\ 77\ GPa(150.5\ mm - {}^{29.3\ mm}/_3)x10^{-3}]x \\ (124.6\ mm - 29.3\ mm)x\ 200\ GPa\ x\ 10^{-3}\end{gathered}}{\begin{bmatrix}376.7\ mm^2 x200\ GPa(124.6\ mm - {}^{29.3\ mm}/_3)(124.6\ mm - 29.3\ mm) + \\ 204\ mm^2 x77\ GPa(150.5\ mm - {}^{29.3\ mm}/_3)(150.5\ mm - 29.3\ mm)\end{bmatrix} x10^{-9}}$$

$f_{s,s} = 295.2\ MPa \leq 0.8 f_y = 0.8\ x\ 420\ MPa = 336.2\ MPa\ \therefore OK$

The stress in the FRP under service loads can be calculated by, — Eq. 10.2.10.2

$$f_{f,s} = f_{s,s}\left(\frac{E_f}{E_s}\right)\frac{d_f - kd}{d - kd} - \varepsilon_{bi}E_f$$

$$f_{f,s} = 295.2\ MPa\left(\frac{77\ GPa}{200\ GPa}\right)\frac{150.5\ mm - 29.3\ mm}{124.6\ mm - 29.3\ mm} - 0.0011\ x\ 77{,}000\ MPa$$

$$f_{f,s} = 59.84\ MPa \le 0.55 f_{fu} = 0.55\ x\ 754.3\ MPa = 414.87\ MPa\ \therefore OK$$

The $0.55f_{fu}$ limit is intended to avoid creep rupture of the FRP under sustained service loads. — Section 10.2.9

The stress at the extreme compression fiber in concrete can be found by, — Table 10.2.9

6

$$f_{c,s} = {}^{C}/_{(b\ x\ kd)/2} = 2C/(b\ x\ kd)$$

To achieve equilibrium, the compressive force is,

$$C = f_{s,s}A_{s,bot} + f_{f,s}A_f$$

$$C = 295.2\ MPa\ x\ 376.67\ mm^2 + 59.84\ MPa\ x\ 204\ mm^2$$

$$C = 123.4\ KN$$

$$f_{c,s} = {}^{2\ x\ 123.4\ x\ 1000N}/_{(1{,}000\ mm\ x\ 29.3\ mm)}$$

$$f_{c,s} = 8.42\ MPa \le 0.6f'_c = 0.6\ x\ 20\ MPa = 12\ MPa\ \therefore OK$$

The service level stresses comply with the limits in ACI 440.2-23.

6l. Detailing of the flexural FRP. — ACI 440.2-23

For negative moment, the flexural FRP should be extended beyond the point of inflection by at least a distance equal to the development length, — Eq. 14.1.3

$$l_{df} = \sqrt{\frac{NE_f t_f}{\sqrt{f'_c}}}$$

$$l_{df} = \sqrt{\frac{1\ x\ 77{,}000\ MPa\ x\ 1.02\ mm}{\sqrt{20\ MPa}}} = 132.5\ mm$$

6.3. Example 3: Confinement of an exterior reinforced concrete column with FRP

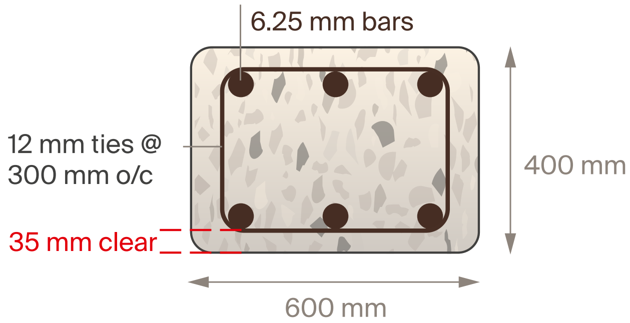


Fig. 35 – Column Section

An existing reinforced concrete column, shown in Fig. 35, is expected to be axially loaded in compression due to a seismic event. This column must be confined with FRP to increase the axial compression strength of the column to resist this expected temporary load. The column has adequate capacity to resist long-term loads for strength and serviceability. The moment demands on this column are insignificant and are neglected in design.

Tab. 25 – Existing column material properties

Concrete compressive strength,
$f'_c = 25\ MPa$

Steel reinforcement yield strength,
$f_y = 420\ MPa$

Steel reinforcement elastic modulus,
$E_s = 200\ GPa$

Tab. 26 – Existing and new factored loads on beam

Type	Existing	New
Factored axial compression P_u	2 613 kN	3 475 kN

Tab. 27 details the design of the FRP confinement for enhancing the axial compressive strength, per the provisions of ACI 440.2-23, with CF Fabrics.

In Section 9.2, ACI 440.2-23 has strengthening limits for the existing structural members to account for damage to the FRP due to vandalism etc. as well as to account for fire resistance. For retrofits required solely for seismic actions, these limits are not applicable per Section 9.2 and Chapter 13.

Tab. 27 – Procedure for design of FRP confinement of a column with CF Fabric

3a.	Column dimensional information. The column section has the following dimensions, $h = 600\ mm$ $b = 400\ mm$	
	The corners of the column will be rounded to achieve a corner radius of, $r_c = 25\ mm$	Section 12.1.2
	Area of longitudinal reinforcement, $A_{st} = 6\ x\ 491\ mm^2 = 2{,}946\ mm^2$	
	Ratio of longitudinal reinforcement to gross area of section, $\rho_g = \frac{A_{st}}{bh} = \frac{2{,}946\ mm^2}{400\ mm\ x\ 600\ mm} = 0.01227$	Section 2.1

3b. Check if the rectangular column is eligible for confinement. — Section 12.1.2

The column section has to comply with two criteria:

$$\frac{h}{b} = \frac{600\ mm}{400\ mm} = 1.5 < 2.0\ \therefore OK$$

$$Maximum\ (h, b) = 600\ mm < 900\ mm \therefore OK$$

3c. Select the CF Fabric U600 for column confinement. — ESR-4774 ICC-ES Ev. Report

The following properties are provided by fischer:

$t_f = 1.02\ mm$
$E_f = 77\ GPa$
$f_{fu} * = 794\ MPa$
$\varepsilon_{fu} * = 1.26\ \% = 0.0126$

3d. Calculate the FRP system design material properties. — ACI 440.2-23 Eq 9.4a Eq. 9.4b

Since the column is on the exterior of the building and will be strengthened with a carbon FRP, per Table 9.4, an environmental reduction factor of 0.85 is used.

$f_{fu} = C_E f_{fu} * = 0.85\ x\ 794 = 675\ MPa$
$\varepsilon_{fu} = C_E \varepsilon_{fu} * = 0.85\ x\ 0.0126 = 0.01071$

3e. Calculate the compressive strength contributions from concrete and steel. — ACI 440.2-23 Eq. 12.1b

The axial compressive strength of a concrete column with steel ties (stirrups) can be calculated as,

$$\phi P_n = 0.8\phi[0.85 f'_c(A_g - A_{st}) + f_y A_{st}]$$

Per ACI 318-19, the strength reduction factor for axial compression is taken as,
$\phi = 0.65$

The concrete contribution to strength is given by,
$P_{n,conc} = 0.85 f'_c(A_g - A_{st})$
$P_{n,conc} = 0.85\ x\ 25\ MPa(400\ mm\ x\ 600\ mm - 6\ x\ 491\ mm^2)$
$P_{n,conc} = 5{,}037{,}398\ N = 5{,}037\ KN$

The steel reinforcement contribution to strength is given by,
$P_{n,steel} = f_y A_{st}$
$P_{n,steel} = 6\ x\ 491\ mm^2\ x\ 420\ MPa$
$P_{n,steel} = 1{,}237{,}32\ N = 1{,}237\ KN$

The axial compressive strength of the column is,
$\phi P_n = 0.8\phi[5{,}037\ KN + 1{,}237\ KN]$
$\phi P_n = 3{,}262\ KN$

Since this strength is less than the required strength of $P_{u,new} = 3{,}475\ KN$, the column needs to be strengthened by confinement with FRP.
Any increase in compressive strength due to FRP confinement is solely attributed to the concrete's contribution. Confinement allows the concrete to achieve higher strains and stresses since the FRP will be activated by dilation of the concrete section under compression.
The additional minimum strength required from the confined concrete section, $P_{n,conc,c}$ can be calculated as,
$P_{n,conc,c} = P_{n,conc} + (P_{u,new} - P_u)/(0.8\phi)$
$P_{n,conc,c} = 5{,}037\ KN + (3{,}475\ KN - 3{,}262\ KN)/(0.8\ x\ 0.65)$
$P_{n,conc,c} = 5{,}447\ KN$

The corresponding minimum confined concrete compressive concrete stress is,

$$f'_{cc,min} = \left(\frac{1}{0.85}\right)\left(\frac{5{,}447\ KN\ x\ 1000}{400\ mm\ x\ 600\ mm}\right) = 26.70\ MPa$$

The confinement has to allow the concrete to reach 26.70 MPa in order for the column to achieve the required compressive strength.

3f. Initial assumption on FRP layering needed for column confinement.
Assume that 3 layers are required transversely to the column axis over the full height of the column.

3g. Calculate the confinement pressure due to the assumed FRP layering.
The maximum confinement pressure due to the FRP is calculated as, — ACI 440.2-23 12.1h

$$f_l = \frac{2E_f N t_f \varepsilon_{fe}}{D}$$

where,

$N = 3$

$$D = \sqrt{b^2 + h^2} = \sqrt{(400\ mm)^2 + (600\ mm)^2} = 721.11\ mm$$ Eq. 12.1.2a

$$\varepsilon_{fe} = \kappa_e \varepsilon_{fu} = 0.55\ x\ 0.01071 = 0.00589$$ Eq. 12.1i, Section 12.1

$$f_l = \frac{2\ x\ 77{,}000\ MPa\ x\ 3\ x\ 1.02\ mm\ x\ 0.00589}{721.11\ mm}$$ Section 12.1

$$f_l = 3{,}85\ MPa > 0.08f'_c = 0.08\ x\ 25\ MPa = 2.0\ MPa\ \therefore OK$$

3h. Calculate the maximum confined compressive strength. — ACI 440.2-23 Eq. 12.1g
The maximum confined concrete compressive strength, f'_{cc}, is calculated as,

$$f'_{cc} = f'_c + \psi_f 3.3\kappa_a f_l$$

where,

$\psi_f = 0.95$ — Section 12.1

$$\frac{A_e}{A_c} = \frac{1 - \dfrac{\left[\left(\frac{b}{h}\right)(h - 2r_c)^2 + \left(\frac{h}{b}\right)(b - 2r_c)^2\right]}{3A_g} - \rho_g}{1 - \rho_g}$$ Eq. 12.1.2d

$$\frac{A_e}{A_c} = \frac{1 - \dfrac{\left[\left(\frac{400\ mm}{600\ mm}\right)(600\ mm - 2\ x\ 25\ mm)^2 + \left(\frac{600\ mm}{400\ mm}\right)(400 - 2\ x\ 25\ mm)^2\right]}{3\ x\ 600\ mm\ x\ 400\ mm} - 0.01227}{1 - 0.01227}$$

$$\frac{A_e}{A_c} = 0.458$$

$$\kappa_a = \frac{A_e}{A_c}\left(\frac{b}{h}\right)^2 = 0.458\left(\frac{400\ mm}{600\ mm}\right)^2 = 0.20$$ Eq. 12.1.2b

$$f'_{cc} = 25\ MPa + 0.95\ x\ 3.3\ x\ 0.20\ x\ 3{,}85\ MPa$$

$$f'_{cc} = 27.41\ MPa > f'_{cc,min} = 26.70\ MPa\ \therefore OK$$

The axial strength of the confined column can be determined as, with f'_{cc} in lieu of f'_c,

$$\phi P_n = 0.8\phi[0.85f'_{cc}(A_g - A_{st}) + f_y A_{st}]$$

$$\phi P_n = 0.8\ x\ 0.65[0.85\ x\ 27.54\ MPa\ (400\ mm\ x\ 600\ mm - 6\ x\ 491\ mm^2) + 420\ MPa\ x\ 6\ x\ 491\ mm^2]/1000$$

$$\phi P_n = 3{,}529\ KN > 3{,}475\ KN\ \therefore OK$$

The column would require three layers of 600U wrapped transversely to achieve the required axial compressive strength. Fig. 36 shows the final configuration for the strengthened section.

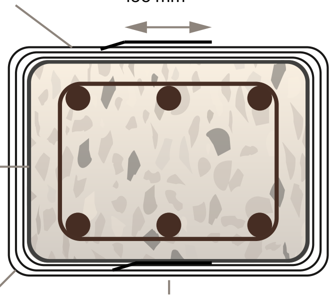


Fig. 36 – Final configuration for column confinement with CF Fabric

Fig. 36 shows the FRP installed in three separate pieces. It is also acceptable to install the FRP continuously around the column to achieve three complete layers. The confinement should be installed over the full height of the column. Transverse FRP does not require overlap between adjacent strips; however, the clear gaps should be limited to 6 mm.

3i. Calculate the maximum confined strain.
The maximum confined strength calculated in the previous step is due to the fact that the confined concrete section can tolerate a higher strain in compression due to the confinement. The calculation of this confined strain is not required for this example, but it is required in case the column is to be checked for deformation capacity under inelastic behavior. Section 12.1, Figure 12.1a and Figure 12.1b of ACI PRC-440.2-23 show how the maximum confined strain, ε_{ccu}, can be used.

$$\varepsilon_{ccu} = \varepsilon'_c \left(1.50 + 12\kappa_b \frac{f_l}{f'_c}\left(\frac{\varepsilon_{fe}}{\varepsilon'_c}\right)^{0.45}\right)$$

ACI 440.2-23
Eq. 12.1j

where,

$$E_c = 4{,}700\sqrt{f'_c} = 4{,}700\ x\ \sqrt{25\ MPa} = 23{,}500\ MPa$$

ACI 318-19

$$\varepsilon'_c = \frac{1.7f'_c}{E_c} = \frac{1.7\ x\ 25\ MPa}{23{,}500\ MPa} = 0.00181$$

Eq. 19.2.2.1b

$$\kappa_b = \frac{A_e}{A_c}\left(\frac{h}{b}\right)^2 = 0.458\left(\frac{600\ mm}{400\ mm}\right)^2 = 1.03$$

Eq. 12.1.2c

$$\varepsilon_{ccu} = 0.00181\left(1.50 + 12\ x\ 1.03\frac{3.85\ MPa}{25\ MPa}\left(\frac{0.00589}{0.00181}\right)^{0.45}\right)$$

$$\varepsilon_{ccu} = 0.00857$$

This confined strain is much higher than the 0.003 strain permitted by ACI 318 for unconfined concrete. Per Section 12.1 of ACI 440.2-23, the value of ε_{ccu} should be limited to 0.01 to prevent excessive cracking and the resulting loss of concrete integrity.

7. Installation Instructions

Installation of the C-Fiber Force Strengthening System must be performed in compliance with the most recent version of the manufacturers product installation instructions (MPII) and the requirements of ACI PRC-440.2-23 and AC125. The manufacturers design drawings, project specifications and product installation instructions, technical datasheets, and materials safety datasheets have to be present on the construction site at all times during installation works. All required installation equipment stated in the MPII should be available on construction site during installation.

Ensure that minimum substrate requirements as stated in the MPII and ACI 440.2-23 are fulfilled and assumptions made in the structural design regarding the substrate quality and construction materials are validated.

Requirements for material and environmental conditions such as material and ambient temperature, relative humidity and substrate moisture as specified in ACI 440.2-23 and MPII must be met prior to application. Product authenticity and shelf life must be verified prior to installation.

Installation works and substrate preparation works should be supervised by the structural engineer, licensed professional, or qualified inspector to ensure correct installation. The list in chapter 8 may provide guidance of the scope of the inspection and documentation.

Detailed installation manuals can be found on *www.fischer-international.com* for the following three applications:

FRS-L-S and FRS-L-H Externally Bonded CFRP Laminates

Carbon Fiber Fabrics

Fire Protection Coating FRS-FP

8. Special Inspection Requirements

FRP composite systems and all associated work should be inspected as required by the applicable local codes. In the absence of such requirements, inspection should be conducted by or under the supervision of a licensed professional or qualified inspector. Inspectors should be knowledgeable of and trained in the installation of FRP systems. The qualified inspector should require compliance with design drawings and project specifications. During installation of the FRP system, the scope of the inspection should include, but is not limited to:

a) Date and time of installation
b) Ambient temperature, relative humidity, and general weather observations
c) Surface temperature of substrate as well as moisture content and dew point
d) Surface preparation methods and resulting profile
e) Qualitative description of surface cleanliness
g) Reinforcement batch number(s) and approximate location in structure
h) Batch numbers, mixture ratios, mixing times, and qualitative descriptions of the appearance of all mixed matrix and additional materials such as primers, putties, and coatings mixed for the day
i) Conformance with installation procedures
j) Strengthening layout, location, orientation and number of plies used. Location of FRP crossings should be checked as well, if present
j) Substrate tensile pull-off test results completed or supervised by a licensed professional or owner's independent testing agency, if required
k) FRP properties from tests of field sample panels or witness panels, if required
l) Presence, location, and size of any defects observed
m) General progress of work and quality

The inspector should provide the licensed professional or owner's representative with inspection records and witness panels when required. The installation contractor should retain sample mortar cubes or cylinders and maintain a record of the placement of each batch as applicable.

Inspection requirements and acceptance
Special inspection must comply with the applicable requirements in Sections 1704 through 1709 of the IBC or Section 1701 of the UBC. Special inspection during the installation of the FRP system must be in accordance with the ICC-ES Acceptance Criteria for Inspection and Verification of Concrete and Unreinforced Masonry Strengthening Using Fiber-reinforced Polymer (FRP) Composite Systems (AC178).

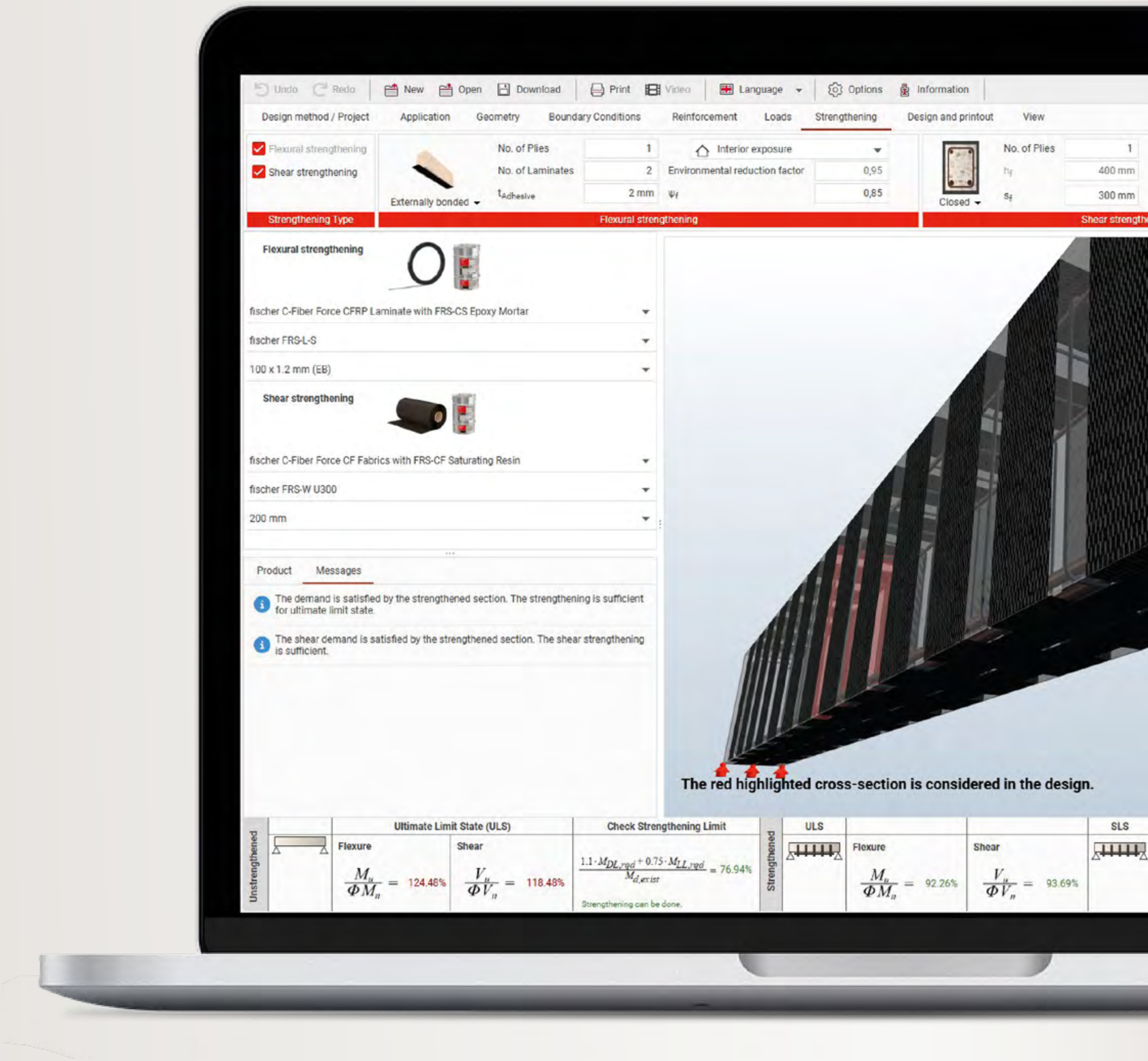
9. References

- ESR-4774 (ICC-ES Evaluation Report), “fischer C-Fiber Force Strengthening System with carbon fiber fabric and with carbon fiber precured laminates”, 2024.
- ACI PRC-440.2-23, “Guide for the Design and Construction of Externally Bonded FRP Systems for Strengthening Concrete Structures”, American Concrete Institute, 2023.
- ACI 440.2R-17, “Guide for the Design and Construction of Externally Bonded FRP Systems for Strengthening Concrete Structures”, American Concrete Institute, 2017.
- ACI 440.7R, “Guide for Design & Construction of Externally Bonded FRP Systems for Strengthening Unreinforced Masonry Structures”, American Concrete Institute, 2017.
- ACI 530, “Building Code Requirements and specification for Masonry Structures, TMS 402-11/ACI 530-11/ASCE 5-11)”, Masonry Standards Joint Committee, 2011.
- IBC, “International Building Code”, International Code Council, 2024, 2021, 2018, and 2015.
- IRC, “International Residential Code”, International Code Council, 2024, 2021, 2018, and 2015.
- ASTM D3039, ”Standard Test Method for Tensile Properties of Polymer Matrix Composite Materials”, ASTM International, 2014.
- ASTM D7234, “Standard Test Method for Pull-Off Adhesion Strength of Coatings on Concrete Using Portable Pull-Off Adhesion Testers”, ASTM International, 2012.

  ASTM D7522, “Standard Test Method for Pull-Off Strength for FRP Laminate Systems Bonded to Concrete Substrate”, ASTM International, 2015.
- ASTM E1640, “Standard Test Method for Assignment of the Glass Transition Temperature By Dynamic Mechanical Analysis”, ASTM International, 2023.
- ASTM E831, “Standard Test Method for Linear Thermal Expansion of Solid Materials by Thermomechanical Analysis”, ASTM International, 2024.
- ICC-ES AC125, “Acceptance Criteria for Concrete and Reinforced and Unreinforced Masonry Strengthening Using Externally Bonded Fiber-Reinforced Polymer Composite Systems”, International Code Council Evaluation Service (ICC-ES), 2019.
- ICRC, ”Guide specifications for externally bonded FRP fabric systems for strengthening concrete structures”, No. 330.2-2016, International Concrete Repair Institute, 2016.
- ICC-ES AC178, “Acceptance Criteria for Inspection and Verification of Concrete and Reinforced and Unreinforced Masonry Strengthening using Fiber-Reinforced Polymer (FRP) or Steel-Reinforced Polymer (SRP) Composite Systems”, International Code Council Evaluation Service (ICC-ES), 2021.

10. Acknowledgements

The contribution of Mr. Ravi Kanitkar (KL Structures, Member of ACI 440 Committee) to Sections 4 and 6 of this Expert Guide is greatly acknowledged.


Undo
Redo
New
Open
Download
Print
Video
Language
Options
Information
Design method / Project
Application
Geometry
Boundary Conditions
Reinforcement
Loads
Strengthening
Design and printout
View
Flexural strengthening
Shear strengthening
Externally bonded
No. of Plies
No. of Laminates
Interior exposure
Environmental reduction factor
0,95
0,85
Closed
Strengthening Type
Flexural strengthening
fischer C-Fiber Force CFRP Laminate with FRS-CS Epoxy Mortar
fischer FRS-L-S
100 x 1.2 mm (EB)
Shear strengthening
fischer C-Fiber Force CF Fabrics with FRS-CF Saturating Resin
fischer FRS-W U300
200 mm
Product
Messages
The demand is satisfied by the strengthened section. The strengthening is sufficient for ultimate limit state.
The shear demand is satisfied by the strengthened section. The shear strengthening is sufficient.
The red highlighted cross-section is considered in the design.
Unstrengthened
Ultimate Limit State (ULS)
Flexure
Shear
124.48%
118.48%
Check Strengthening Limit
76.94%
Strengthening can be done.
Strengthened
ULS
92.26%
93.69%
SLS

FiXperience

FiXperience.
Safe and reliable.

The fischer design Software FiXperience gives you safe and reliable support in dimensioning your projects whether you are a planer, structural engineer, or craftsman.

FiXperience is set up modularly and usable for a variety of applications. The program includes an engineering software with special application modules:

C-FIX

C-FIX
The anchor design program for steel and bonded anchor in concrete, as well as injection systems for masonry. Now with the new FEM design tool for the realistic design of anchorages.

WOOD-FIX

WOOD-FIX
For the calculation of on-rafter insulation systems and joints in structural timber engineering.

INSTALL-FIX

INSTALL-FIX
For the design and dimensioning of MEP installation systems.

REBAR-FIX

REBAR-FIX
For the design of post-installed rebars in reinforced concrete.

REINFORCE-FIX

REINFORCE-FIX
To design structural strengthening of reinforced concrete members.

MORTAR-FIX

MORTAR-FIX
To determine the injection resin volume for bonded anchors in concrete and masonry.

RAIL-FIX

RAIL-FIX
For the design of fixings for railings on reinforced concrete slabs and staircases.

FACADE-FIX

FACADE-FIX
For the design of façade fixings with timber sub-structure.

CHANNEL-FIX

CHANNEL-FIX
For the design of cast-in channels and inserts.

Register on the **myfischer portal** to use **FiXperience online** or **download FiXperience** for free.

Main catalogue Structural Strengthening.

Products and solutions for structural strengthening applications.

The Structural Strengthening Catalogue offers many facts and helps with quick and safe product selection, e. g.:

- Carbon fiber laminates and carbon fiber wrap solutions
- Products for concrete overlay solutions
- Products for post-installed rebars
- Product descriptions with benefits
- Tips for installation
- Basics of structural strengthening
- Technical data

Order now: info@fischer.com

Main catalogue Fixing Systems.

Products for use in fixing technology.

The fixing catalogue offers many facts and helps with quick and safe product selection, e. g.:

- Product descriptions with benefits
- Tips for installation
- Application aids
- Detailed technical data and drawings
- Basics of fastening technology
- All you need to know about professional fixing

Order now: info@fischer.com


fischer
FRS-CF
FRS-CS
FIS EM Plus
FIS RC II

fischerwerke GmbH & Co. KG
72178 Waldachtal
Germany

Sales Organisation:

www.fischer-international.com

fischer stands for

Fixing Systems
fischertechnik
Consulting
Electronic Solutions

fischerwerke GmbH & Co. KG
Klaus-Fischer-Straße 1 · 72178 Waldachtal
Germany
P +49 7443 12 – 0
www.fischer-international.com · structuralretrofitting@fischer.de

577426 · 16/2025 · Printed in Germany · Subject to technical changes.