SEMINAR SERIES 2018

fib-Short Course

REINFORCING & STRENGTHENING OF STRUCTURES WITH ADVANCED COMPOSITES

Presented by

Concrete NZ – Learned Society &
The International Federation for Structural Concrete (*fib*)

Seminar Notes

(TR69)

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Thanasis Triantafillou, University of Patras

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Representatives from, BBR Contech, Concrete Solutions and Mapei

fib-Short Course
Reinforcing & Strengthening of Structures with Advanced Composites
July 2018

SEMINAR NOTES DIRECTORY

Session 1

The *fib* Model Codes

FRP materials and bond

Balázs György

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fib-Short Course
REINFORCING & STRENGTHENING OF
STRUCTURES WITH ADVANCED COMPOSITES

FRP materials and bond

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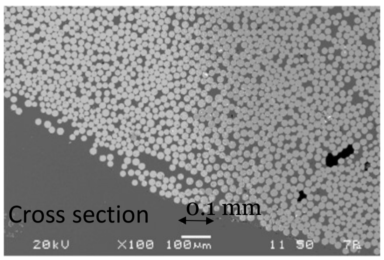
Thanks to contributions by **Sándor Sólyom (BME)**

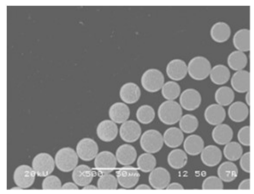
Christchurch-Wellington-Auckland, 3-4-5 July 2018

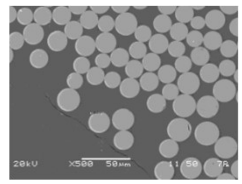
FRP = Fibre Reinforced Polymer

Fibre Reinforced Polymer

Fibres	+	Matrix
◉ diameter: 5 to 20 μm		◉ protection of fibres
◉ high strength		◉ distribution of loads
◉ linear elastic		◉ „low" strength
◉ brittle		◉ linear elastic (short term load)
◉ 60 to 75 V%		◉ viscoelastic (long term load)

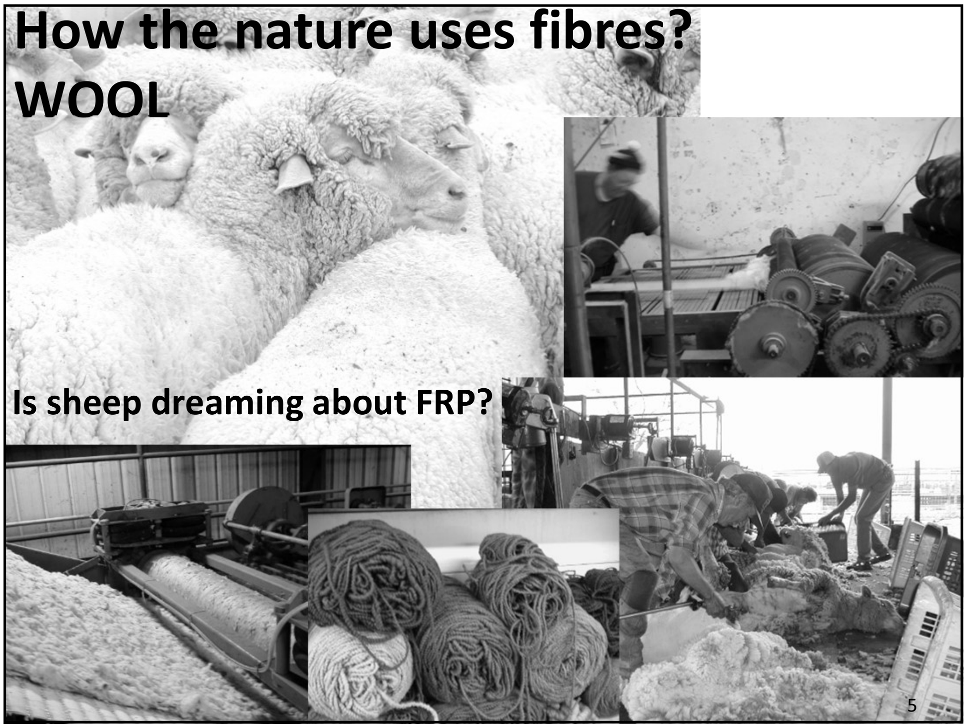






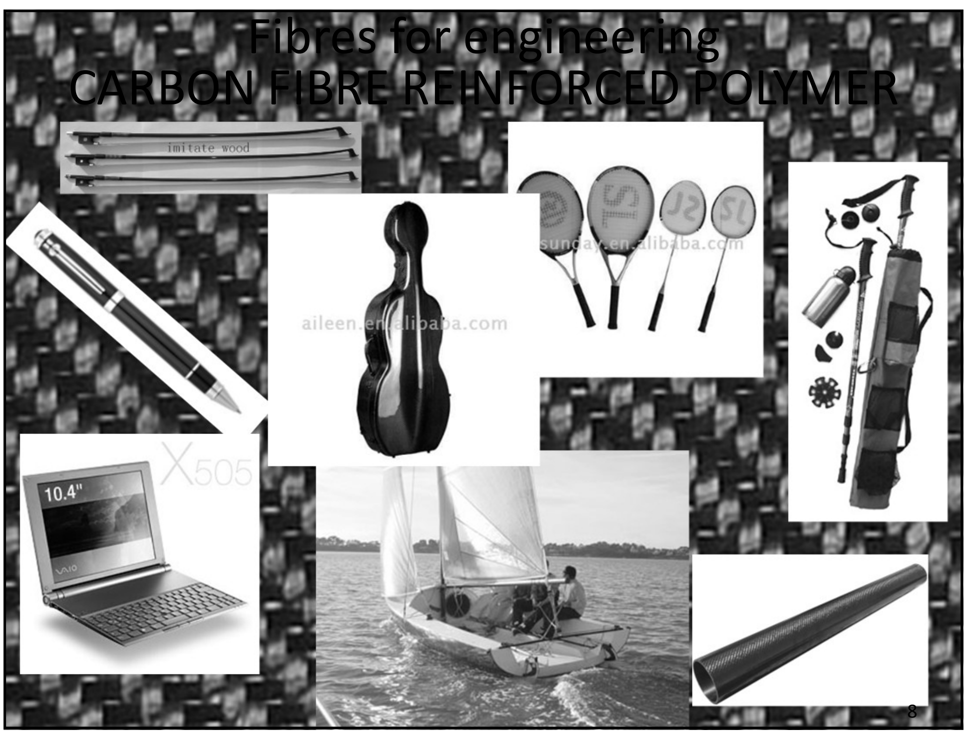
What is FRP?

How the nature uses fibres?
COTTON

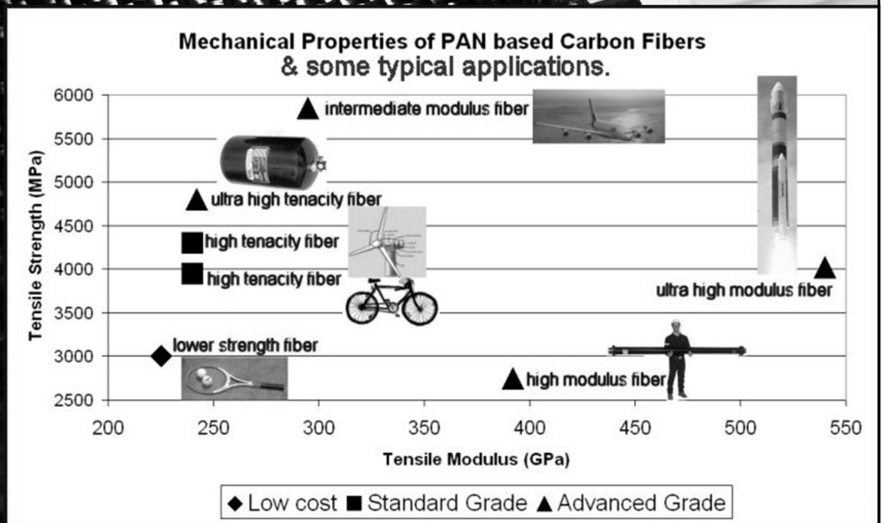

How the nature uses fibres?
WOOL
Is sheep dreaming about FRP?
5


Fibres for engineering
CARBON FIBRE REINFORCED POLYMER
Image of the new Airbus A350
The entire wing structure will be made out of carbon fiber
6


Fibres for engineering
CARBON FIBRE REINFORCED POLYMER
7/55


Fibres for engineering
CARBON FIBRE REINFORCED POLYMER
imitate wood
10.4"
X505
8

Fibres for engineering
CARBON FIBRE REINFORCED POLYMER



CONSTITUENTS OF FIBER REINFORCED POLYMER (FRP) COMPOSITES

Fiber reinforced polymer (FRP) composites are made of three essential constituents:

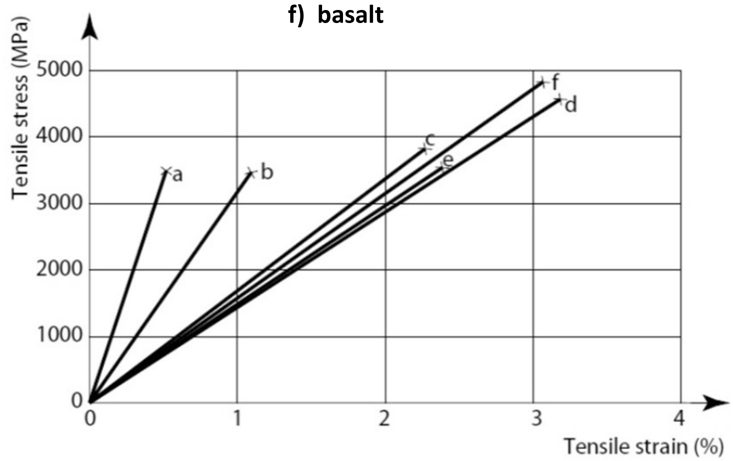
- **fibers** **typically: glass, carbon, aramid, basalt**
- **polymer matrices** **epoxy, polyester, vinyl ester**
- **additives**

The **additives** include plasticizers, impact modifiers, heat stabilizers, antioxidants, light stabilizers, flame retardants, blowing agents, antistatic agents, coupling agents, and others.

Stress-strain curves of typical reinforcing fibres

(*fib* Bulletin 40, 2007)

a) carbon (high modulus);
b) carbon (high strength);
c) aramid (Kevlar 49);
d) S-glass; e) **E-glass**;
f) basalt



GLASS FIBERS

Commercially available glass fibers:

- ***E-glass***, which has low alkali content and is the most common type of glass fiber in high-volume commercial use. It is used widely in combination with polyester and epoxy resins to form a composite. Its advantages are low susceptibility to moisture and high mechanical properties.
- ***Z-glass*** - for cement mortars and concretes due to its high resistance against alkali attack.
- ***A-glass*** - high alkali content.
- ***C-glass*** - greater corrosion resistance to acids, such as chemical applications.
- ***S-*** **or** ***R-glass*** - high strength and modulus
- Low ***K-glass*** is an experimental fiber produced to improve dielectric loss properties in electrical applications and is similar to ***D-glass*** (dielectric glass).

GLASS FIBERS

Advantages:

1. Low cost
2. High tensile strength
3. Excellent insulating properties

Drawbacks:

1. Low tensile modulus
2. Relatively high specific gravity
3. Sensitivity to abrasion from handling
4. Sensitivity to alkalies
5. Relatively low fatigue resistance

CARBON (GRAPHITE) FIBERS

Classification and Types

1. Based on precursor materials

- **PAN-based** carbon fibers
- **Pitch-based** carbon fibers
- Mesophase pitch-based carbon fibers
- Isotropic pitch-based carbon fibers
- Rayon-based carbon fibers
- Gas-phase-grown carbon

Carbon fiber > 90% carbon by weight

Graphite fiber > carbon 95% by weight

2. Based on fiber properties fibers

- Ultra-high-modulus (UHM)-type UHM (> 450 GPa)
- High-modulus (HM)-type HM (325 to 450 GPa)
- Intermediate-modulus (IM)-type IM (200 to 325 GPa)
- Low modulus and high-tensile (HT)-type HT (modulus < 100 GPa and strength > 3.0 GPa)
- Super high-tensile (SHT)-type SHT (tensile strength > 4.5 GPa)

3. Based on final heat treatment temperature

- Type I (high-heat-treatment carbon fibers): associated with high-modulus type fiber (> 2000°C)
- Type II (intermediate-heat-treatment): associated with high-strength strength type fiber (> 1500°C and < 2000°C)
- Type III (low-heat-treatment carbon fibers): associated with low modulus and low strength fibers (< 1000°C)

CARBON FIBERS

Advantages:

1. High tensile strength-to-weight ratio
2. High tensile modulus-to-weight ratio
3. Very low coefficient of linear thermal expansion
4. High fatigue strength

Drawbacks:

1. High cost
2. High brittleness
3. Electrical conductivity

(might limit their application potential)

ARAMID FIBERS

DuPont (1971) ⇒ Kevlar™

Aromatic compound of: **ARAMID: aromatic polyamid**

- carbon
- hydrogen
- oxygen
- nitrogen

Aramid fibers have:

- no melting point
- low flammability
- good fabric integrity at elevated temperatures
- para-aramid fibers, which have a slightly different molecular structure, provide outstanding strength-to-weight properties, high tenacity and high modulus.

(GangaRao, Taly, Vijay, 2007)

ARAMID FIBERS

Advantages:

1. very low thermal conductivity;
2. a very high damping coefficient;
3. **high degree of yielding under compression** (it gives superior tolerance to damage against impact and other dynamic loading)

Drawbacks:

1. **hygroscopic - they can absorb moisture up to about 10% of fiber weight**;
2. at high moisture content, they tend to crack internally at pre-existing microvoids and produce longitudinal splitting.;
3. low compressive strength and exhibit a loss of strength and modulus at elevated temperatures;
4. present difficulty in cutting and machining;
5. sensitive to UV lights.

Typical properties of fibres for FRP composites

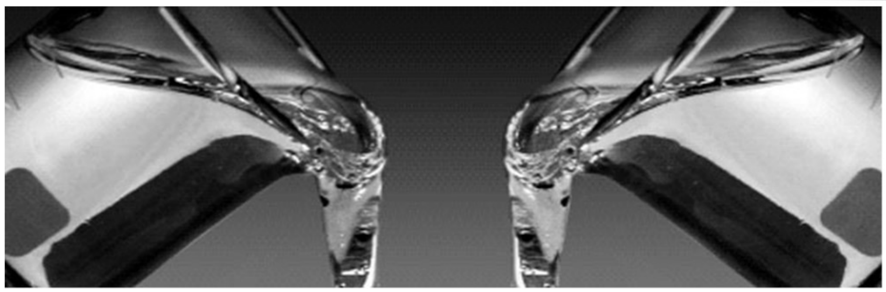
Fibre Type	Density (kg/m^3)	Tensile strength (MPa)	Young modulus (GPa)	Ultimate tensile strain (%)	Thermal expansion coefficient (10^{-6}/°C)	Poisson's coefficient
E-glass	2500	3450	72.4	2.4	5	0.22
S-glass	2500	4580	85.5	3.3	2.9	0.22
Alkali resistant glass	2270	1800-3500	70-76	2.0-3.0	-	-
ECR	2620	3500	80.5	4.6	6	0.22
Carbon (high modulus)	1950	2500-4000	350-650	0.5	-1.2...-0.1	0.20
Carbon (high strength)	1750	3500	240	1.1	-0.6...-0.2	0.20
Aramid (Kevlar 29)	1440	2760	62	4.4	-2.0 longitudinal 59 radial	0.35
Aramid (Kevlar 49)	1440	3620	124	2.2	-2.0 longitudinal 59 radial	0.35
Aramid (Kevlar 149)	1440	3450	175	1.4	-2.0 longitudinal 59 radial	0.35
Aramid (Technora H)	1390	3000	70	4.4	-6.0 longitudinal 59 radial	0.35
Aramid (SVM)	1430	3800-4200	130	3.5	-	-
Basalt (Albarrie)	2800	4840	89	3.1	8	-

(***fib*** Bulletin 40, 2007)

Polymer matrices

Thermoplastic polymers:
can be deformed easily by heating

Thermoset polymers:
cannot be softed by heating

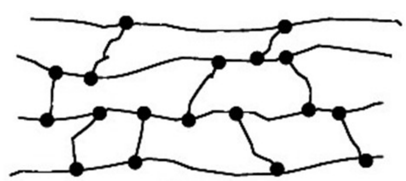


Schematic representations of

thermoplastic polymers

thermoset polymers

(a)



(b)

(GangaRao, Taly, Vijay, 2007)

Matrix

http://contentinjection.com/uses-of-epoxy-resin/

Commercially available thermoset matrixes:

- epoxy
- polyester
- vinyl ester
- others: acrylics, phenolics, polyurethanes, melamines, silocones and polymides

Most reinforced plastics are thermosetting.

There are three main reasons for using thermoset resins in producing composites:

1. Better bonding between fibers and matrix with compatible sizing
2. Ability to cure at room temperature in the presence of a catalyst
3. Good creep resistance

Typical properties of thermosetting matrices

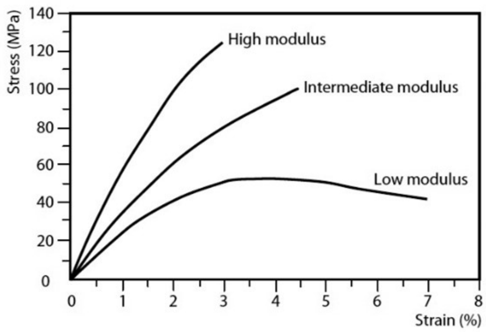
Property	*Matrix*		
	Polyester	Epoxy	Vinyl ester
Density (kg/m^3)	1200 - 1400	1200 - 1400	1150 - 1350
Tensile strength (MPa)	34.5 - 104	55 - 130	73 – 81
Longitudinal modulus (GPa)	2.1 – 3.45	2.75 – 4.10	3.0 – 3.5
Poisson's coefficient	0.35 – 0.39	0.38 – 0.40	0.36 – 0.39
Thermal expansion coefficient (10^{-6}/°C)	55 - 100	45 - 65	50 - 75
Moisture content (%)	0.15 – 0.60	0.08 – 0.15	0.14 – 0.30

Typical properties of thermoplastic matrices

Property	*Matrix*		
	PEEK	PPS	PSUL
Density (kg/m^3)	1320	1360	1240
Tensile strength (MPa)	100	82.7	70.3
Tensile modulus (GPa)	3.24	3.30	2.48
Tensile elongation (%)	50	5	75
Poisson's coefficient	0.40	0.37	0.37
Thermal expansion coefficient (10^{-6}/ °C)	47	49	56

(***fib*** Bulletin 40, 2007)

Stress-strain curves of epoxy matrix resins of different moduli
(***fib*** Bulletin 40, 2007)



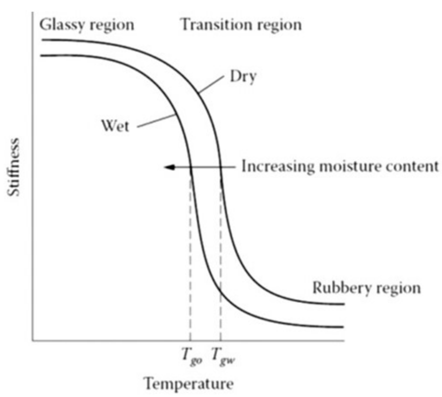
A**dvantages of epoxies** over other types of resins are as follows:

1. Wide range of properties allows a greater choice of selection
2. Absence of volatile matters during cure
3. Low shrinkage during curing
4. Excellent resistance to chemicals and solvents
5. Excellent adhesion to a wide variety of fillers, fibers, and other substrates

Major disadvantages of epoxies are:

1. relatively high cost and
2. long cure time

Glass transition temperature (T_g)



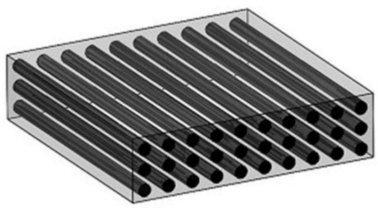
Increase in temperature causes a gradual softening of the polymer matrix material up to a certain point, indicating a transition from a glassy behavior to a rubbery behavior. The temperature at which this occurs is called the glass transition temperature: T_g

The diagram indicates the variation of stiffness with temperature for a typical polymer showing the glass transition temperature, *Tg*, and the effect of absorbed moisture in *Tg.* (GangaRao, Taly, Vijay, 2007)

MICROMECHANICS OF FRP COMPOSITES

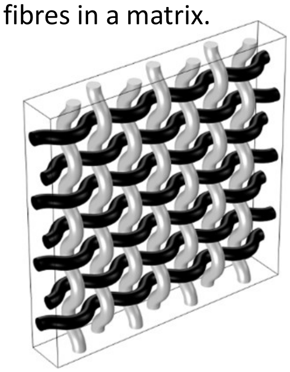
Micromechanics is a part of mechanics of materials which enables the analysis of the effective composite properties in terms of constituent materials properties.

The basic building block of a laminate is a lamina which is a flat (or curved) arrangement of unidirectional fibres or woven fibres in a matrix.



Lamina with unidirectional fibres

(https://dragonplate.com/sections/technology.asp)



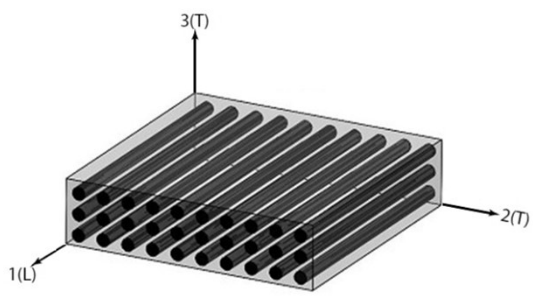
Lamina with woven fibres

(https://www.comsol.com/blogs/defining-curvilinear-coordinates-anisotropic-materials)

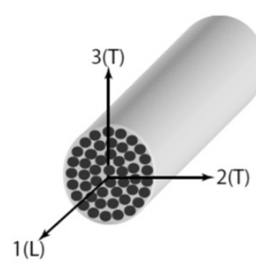
CONSTITUENT OF THE UNIDIRECTIONAL LAMINA

- The direction parallel to the fibres is called **longitudinal direction** (axis 1 or L)
- The direction perpendicular to the fibres in the 1-2 plane is called **transverse direction**
- Any direction in the 2-3 plane is also transverse direction

Because of the distribution of the fibres FRP lamina is ***orthotropic*** with respect to L(1) and T(2) and transversally ***isotropic*** with respect to T(2) and T(3)



Unidirectional fibre reinforced lamina



Unidirectional fibre reinforced round bar

Volume Fractions

Basic characteristics/parameters

- v_c - volume of composite
- v_f - volume of fiber
- v_m - volume of matrix

- V_f - fibre volume fraction
- V_m - matrix volume fraction

- ρ_c - density of composite
- ρ_f - density of fiber
- ρ_m - density of matrix

$$V_f = \frac{v_f}{v_c}$$

$$V_m = \frac{v_m}{v_c}$$

$$V_f + V_m = 1$$

$$v_f + v_m = v_c$$

Longitudinal Tensile Strength $(\boldsymbol{\sigma_1^T})_{\boldsymbol{ult}}$

Ultimate failure strains:

$$(\varepsilon_f)_{ult} = \frac{(\sigma_f)_{ult}}{E_f}$$

$$(\varepsilon_m)_{ult} = \frac{(\sigma_m)_{ult}}{E_m}$$

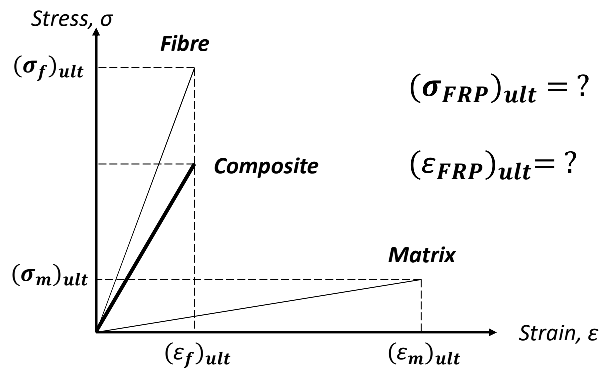
- fibers carry most of the load in polymeric matrix composites

it is assumed that,

at the strain of $(\varepsilon_f)_{ult}$, the whole composite fails.

$$(\sigma_1^T)_{ult} = (\sigma_f)_{ult} V_f + \left(\varepsilon_f\right)_{ult} E_m (1 - V_f)$$

Longitudinal Tensile Strength $(\sigma_1^T)_{ult}$



$(\sigma_{FRP})_{ult} = ?$

$(\varepsilon_{FRP})_{ult} = ?$

$(\sigma_1^T)_{ult}$ - ultimate tensile strength of fibre

E_f - Young's modulus of fibre

$(\sigma_m)_{ult}$ - ultimate tensile strength of matrix

E_m - Young's modulus of matrix

Example - Longitudinal Tensile Strength

Find the ultimate tensile strength for a glass/epoxy lamina with a *70% fiber volume fraction*. Use the following properties for glass and epoxy.

$$E_f = 85\ GPa$$
$$(\sigma_f)_{ult} = 1550\ MPa$$

$$E_m = 3.4\ GPa$$
$$(\sigma_m)_{ult} = 72\ MPa$$

$$(\varepsilon_f)_{ult} = \frac{1550 * 10^6}{85 * 10^9} = 0.0182$$

$$(\varepsilon_m)_{ult} = \frac{72 * 10^6}{3.4 * 10^9} = 0.0212$$

$$(\sigma_1^T)_{ult} = (\sigma_f)_{ult} V_f + \left(\varepsilon_f\right)_{ult} E_m (1 - V_f) \qquad V_f = 0.7$$

$$\sigma_f = \sigma_{fib} V_{fib} + \sigma_m V_m = \sigma_{fib} V_{fib} + \sigma_m (1 - V_{fib})$$

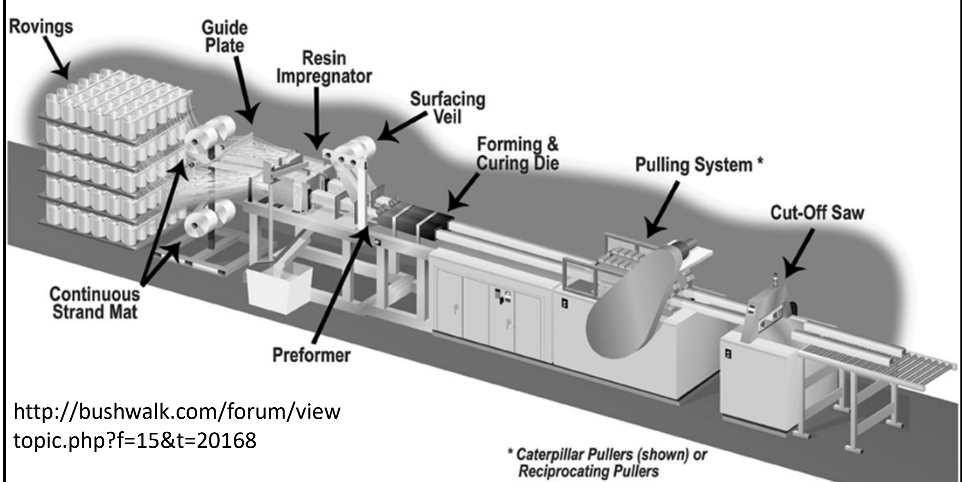
$$(\boldsymbol{\sigma_1^T})_{\boldsymbol{ult}} = (1550 * 10^6)(0.7) + (0.1823 * 10^{-1})(3.4 * 10^9)(1 - 0.7) = \mathbf{1104}\ MPa$$

$(\sigma_f)_{ult} * V_f$ =1550*0.7=1085 MPa

Manufacturing Methods

- Hand (Wet) Lay-up/Automated Lay-up
- Pultrusion
- Filament Winding
- Resin Transfer Molding (RTM)
- Sheet Molding Compound (SMC)
- Seemann Composite Resin Infusion Molding Process (SCRIMP)
- Injection Molding
- Compression Molding
- Extrusion

Pultrusion

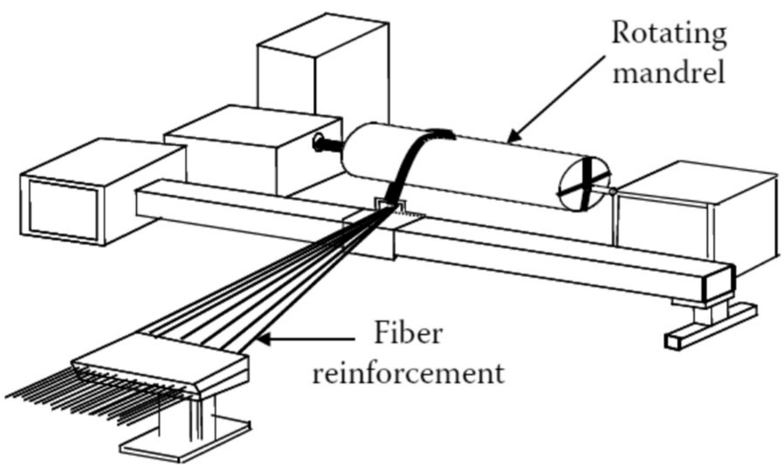


http://bushwalk.com/forum/view
topic.php?f=15&t=20168

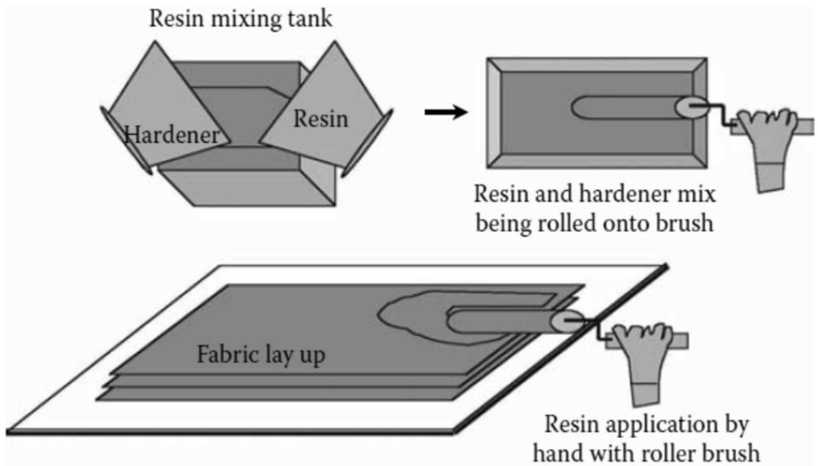
https://www.youtube.com/watch?v=SeqDm9l3yEM

Click to start

Filament winding

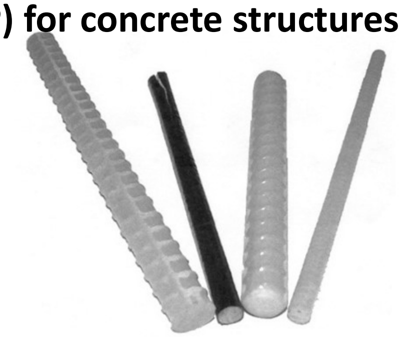


Wet lay-up

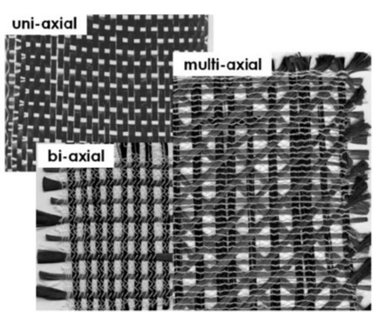


(GangaRao, Taly, Vijay, 2007)

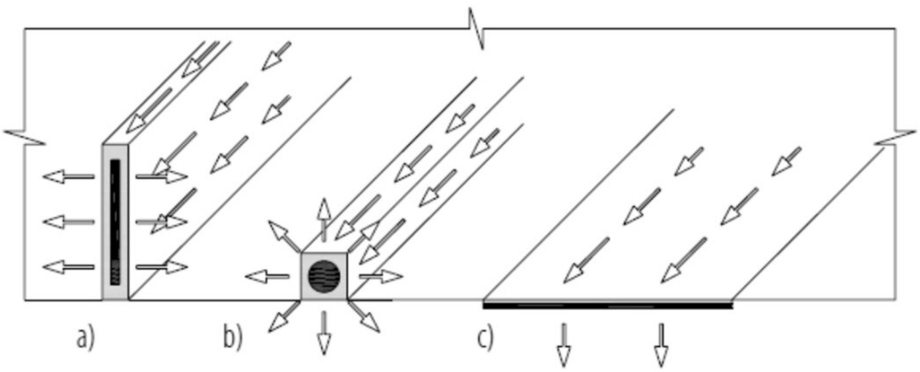
Fibre reinforced polymers (FRP) for concrete structures



- ***for reinforcement***
  1) as prestressing tendon
  2) as not prestressed reinforcement
- ***for strengthening***
  1) as FRP plates
  2) as FRP fabrics



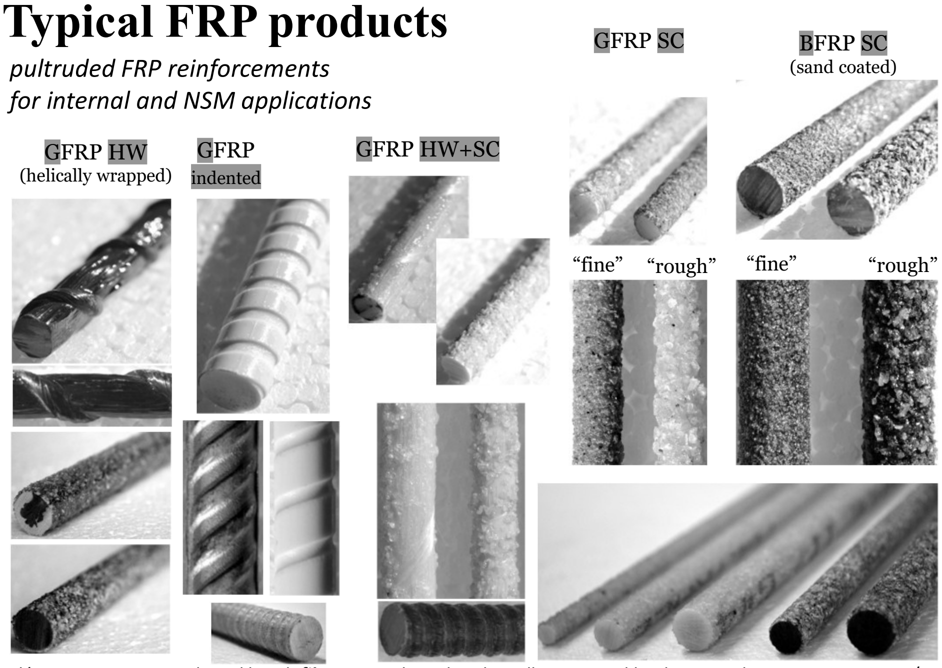
Fibre reinforced polymers (FRP) for concrete structures



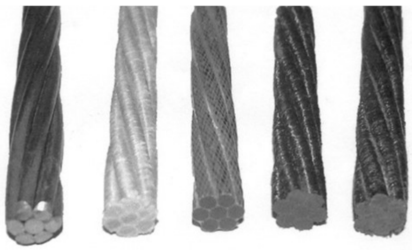
(PhD Thesis - Zsombor K. SZABO,
Supervisor G. L. Balázs)

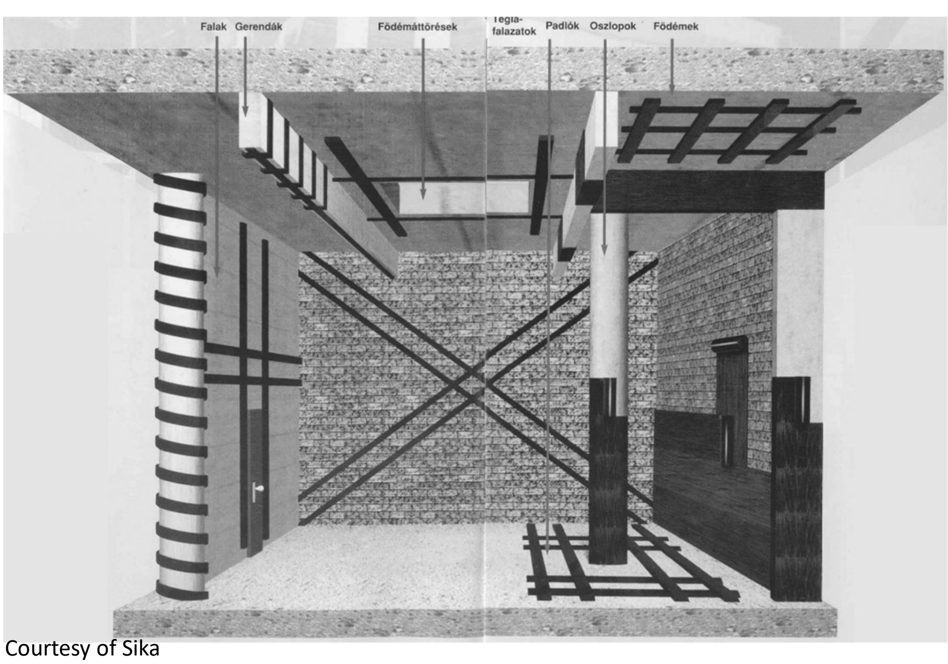
Typical FRP products

pultruded FRP reinforcements
for internal and NSM applications



Typical FRP products





Courtesy of Sika

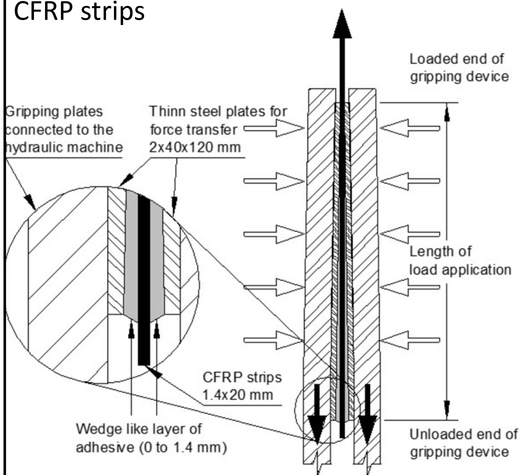

Application of micromechanics
Softeye
41

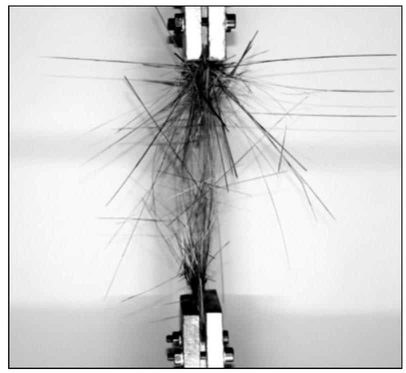

42

TESNSILE TEST OF CFRP STRIP

(Szabó, Balázs)

For propper force trasfer two thin steel plates were glued on oposite sides of the CFRP strips

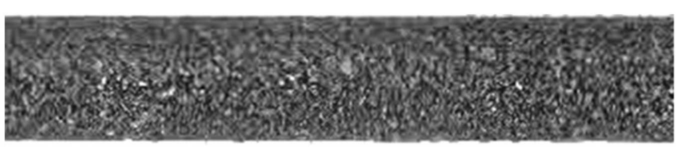




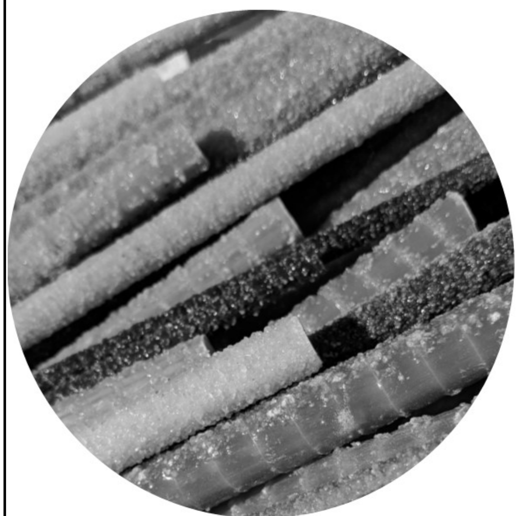
INTERFACE CHARACTERISTICS (5 mm diameter wires)

Bond of FRP reinforcement is different from that of steel reinforcement!

Steel prestressing wire



FRP prestressing wire

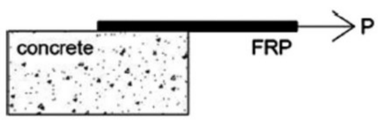


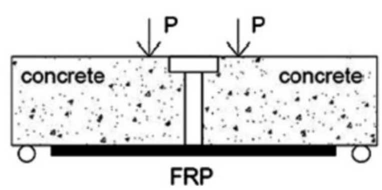
“Bond between concrete and FRP reinforcing bars is the key to develop the composite action of FRP RC”

fib Bulletin 40, p. 91 (2007)

Test types

to study the bond behavior of EBR





D’Antino, Pellegrino, 2014

Importance of bond

Bond of embedded steel reinforcement or FRP (internal or external) influences performance of concrete structures in several ways.

At the serviceability limit state, bond influences

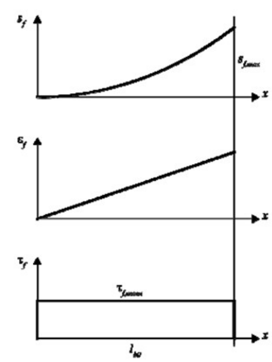
- width and spacing of transverse cracks
- tension stiffening
- curvature.

At the ultimate limit state, bond is responsible for

- strength of end anchorages (and intermediate regions for EBR FRP)
- lapped joints of reinforcement (steel rebars)
- rotation capacity of plastic hinge regions (steel rebars).

Christoph Czaderski, FRP Training Course, Gent, Belgium, 26 January, 2016

Simplified bond shear stress-slip relation



$$s_f(x) = \frac{\tau_f}{E_f t_f}\frac{x^2}{2} \qquad (1)$$

$$\varepsilon_f(x) = \frac{\tau_f}{E_f t_f} x \qquad (2)$$

$$\tau_f(x) = \tau_{f,mean} \qquad (3)$$

with $l_{b0} = \frac{F_{b0,R}}{\tau_{f,mean} \cdot b_f}$ and $s_f(x = l_{b0}) = s_{f,max}$ we get from Eq. (1) and (3): $s_{f,max} = \frac{F_{b0,R}^2}{2E_f t_f b_f^2 \tau_{f,mean}}$

and with $G_{Fb} = s_{f,max}\tau_{f,mean}$ we get $F_{b0,R} = b_f\sqrt{2G_{Fb}E_f t_f}$

Therefore, $F_{b0,R}$ is proportional to $b_f\sqrt{E_f t_f f_{ct}}$

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Active (effective, maximum) bond length

(the length which is actively involved in the force transfer from the strip to the concrete)

If we assume a constant bond shear stress:

$$l_{b0} \approx \frac{F_{b0,R}}{b_f \cdot \tau_{f,mean}}$$

with

$$F_{b0,R} = b_f \sqrt{2 \cdot G_{Fb} \cdot E_f \cdot t_f}$$

$$\longrightarrow \quad l_{b0} = \frac{\sqrt{2 G_{Fb} E_f t_f}}{\tau_{f,mean}}$$

Therefore, l_{b0} is proportional to $\sqrt{\frac{E_f t_f}{f_{ct}}}$

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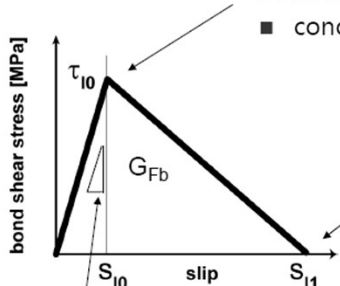
Bond behavior depends on



Bond damage
- concrete quality

maximum slip
- constant (~0.2 mm)

Additionally, the stiffness ($E_l t_l$) of the strip influences also the bond behaviour.

Elastic deformation
- shear modulus of adhesive and concrete
- thickness of adhesive plus a layer of concrete

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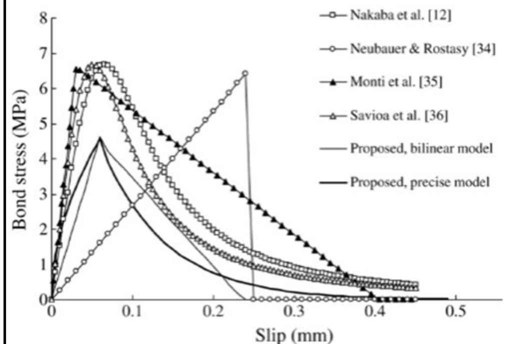
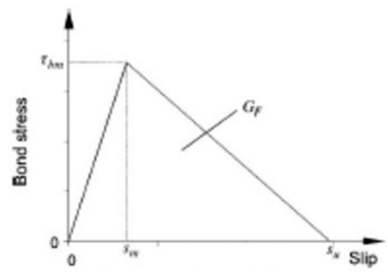
Local bond stress-slip model for EBR



Lu, Teng, Ye and Jiang, 2005



fib Model Code 2010

$$\frac{\tau_b}{\tau_{bm}} = \left(\frac{s}{s_m}\right) \quad 0 < s < s_m$$

$$\tau_b = \tau_{bm} - \tau_{bm}(s - s_m)/(s_u - s_m) \quad sm < s < su$$

Guidelines for anchorage capacity ($l_b > l_{bmax}$)

***fib* Model Code 2010** $F_{fbm} = k_b \cdot b_f \sqrt{0.125 \cdot E_f \cdot t_f \cdot f_{cm}^{2/3}}$ $\quad k_b = \sqrt{\frac{2 - b_f/b}{1 + b_f/b}} \geq 1$

SIA 166 $F_{b0,R} = b_f \sqrt{2 \cdot G_{Fb} \cdot E_f \cdot t_f} = b_f \sqrt{2 \frac{f_{ctH}}{8} E_f t_f} = 0.5 b_f \sqrt{E_f t_f f_{ctH}}$ $\quad G_{Fb}\left[\frac{N}{mm}\right] = \frac{f_{ctH}\left[\frac{N}{mm^2}\right]}{8\left[\frac{1}{mm}\right]}$

***fib* Bulletin 14** $N_{fa,max} = \alpha \cdot c_1 \cdot k_c \cdot k_b \cdot b_f \cdot \sqrt{E_f \cdot t_f \cdot f_{ctm}}$

$$k_b = 1.06 \sqrt{\frac{2 - \frac{b_f}{b}}{1 + \frac{b_f}{400}}} \geq 1$$

TR55 $T_{k,max} = 0.5 \cdot k_b \cdot b_f \cdot \sqrt{E_{fd} \cdot t_f \cdot f_{ctk}}$

Italian Code $F_{fd} = b_f \sqrt{2 \cdot E_f \cdot t_f \cdot \Gamma_{Fk}}$; $\Gamma_{Fk} = 0.03 \cdot k_b \cdot \sqrt{f_{ck} \cdot f_{ctm}}$

Take care to symbols. They depend on the reference. Please note: without safety factors!!

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Guideline-Equations for active bond length

(minimum necessary length for maximum anchor resistance $F_{b0,R}$)

***fib* Model Code 2010**	$l_{b,max} = \sqrt{\frac{E_f \cdot t_f}{2 \cdot f_{ctm}}}$
SIA 166	$l_{b0} = \frac{\pi}{2} \cdot \sqrt{2 \cdot \frac{G_{Fb} \cdot E_l \cdot t_l}{\tau_{l0}^2}} \;\Big|\; \tau_{l0} = \frac{4}{3} f_{ctH} \;\Big|\; l_{b0} = \frac{3\pi}{16} \cdot \sqrt{\frac{E_l \cdot t_l}{f_{ctH}}}$
***fib* Bulletin 14**	$l_{b,max} = \sqrt{\frac{E_f \cdot t_f}{c_2 \cdot f_{ctm}}}$
TR55	$l_{t,max} = 0.7 \cdot \sqrt{\frac{E_{fd} \cdot t_f}{f_{ctk}}}$
Italian Code	$l_e = \sqrt{\frac{E_f \cdot t_f}{2 \cdot f_{ctm}}}$

Take care to symbols. They depend on the reference. Please note: without safety factors!!

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Example

Concrete C30/37

Sika CarboDur S1012 — elastic modulus = 165GPa

tensile strength > 2800 MPa

tensile strain > 1.7%

$$G_{Fb} = 0.35 / 0.40 / 0.45 \, \frac{N}{mm}$$

$$F_{b0,R} = b_f \cdot \sqrt{2 \cdot G_{Fb} \cdot E_f \cdot t_f} =$$

$$= 100\sqrt{2 \cdot G_{Fb} \cdot 165'000 \cdot 1.2} = 37.2 / 39.8 / 42.2 \, kN$$

$$\sigma_{b0,R} = \frac{F_{b0,R}}{A_f} = 310 / 332 / 352 \, MPa \qquad \varepsilon_{b0,R} = \frac{\sigma_{b0,R}}{E_f} = 1.88 / 2.01 / 2.13 \text{‰}$$

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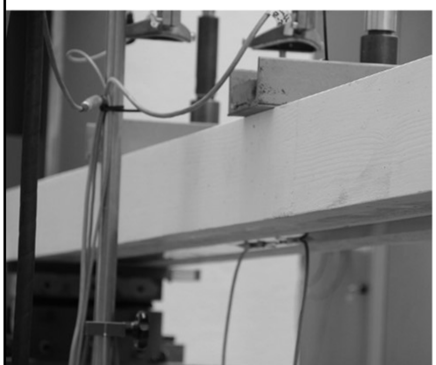
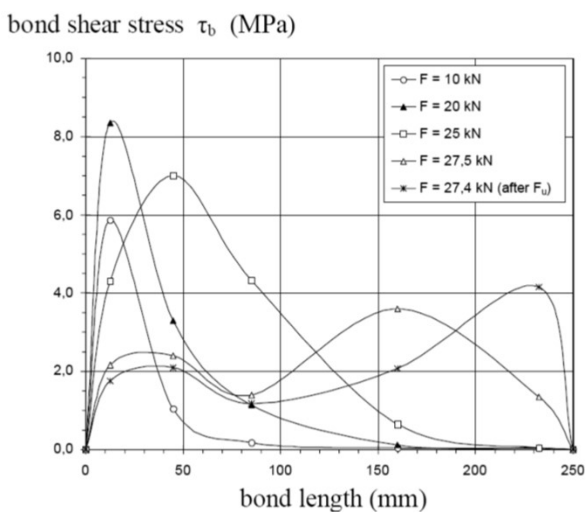
Flexural test (EBR)



55

Distribution of shear stresses along the bond length (Zilch et al., 1998)



fib Bulletin 14 - Externally bonded FRP reinforcement for RC structures

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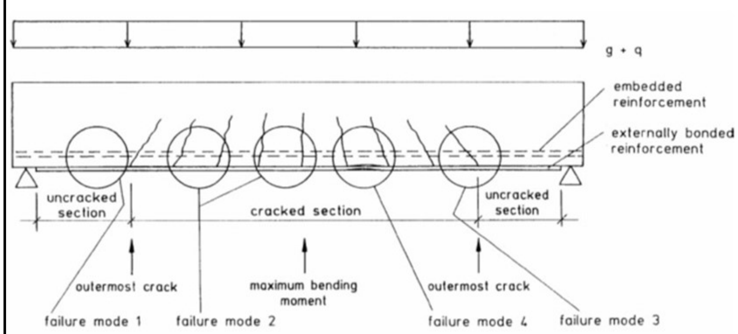

Bond failure modes of a concrete member with EBR (Blaschko et al., 1998)

Mode 1: peeling-off in an uncracked anchorage zone
The FRP may peel-off in the anchorage zone as a result of bond shear fracture through the concrete.

Mode 2: peeling-off caused at flexural cracks
Flexural (vertical) cracks in the concrete may propagate horizontally and thus cause peeling-off of the FRP in regions far from the anchorage.

Mode 3: peeling-off caused at shear cracks
Shear cracking in the concrete generally results in both horizontal and vertical opening, which may lead to FRP peeling-off. However, in elements with sufficient internal (and external) shear reinforcement (as well as in slabs) the effect of vertical crack opening on peeling-off is negligible

Mode 4: peeling-off caused by the unevenness of the concrete surface
The unevenness or roughness of the concrete surface may result in localized debonding of the FRP, which may propagate and cause peeling-off.

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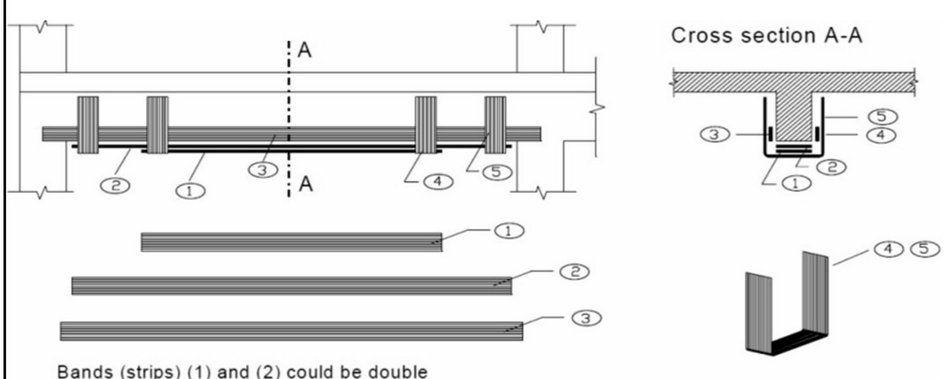

Flexural reinforcement with possible shear anchorages

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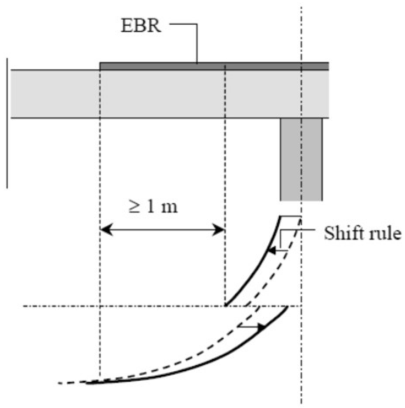

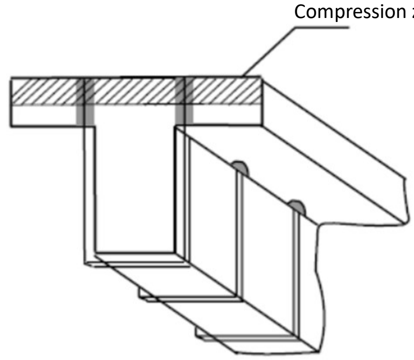

Anchorage above internal supports

Anchorage in the compression zone

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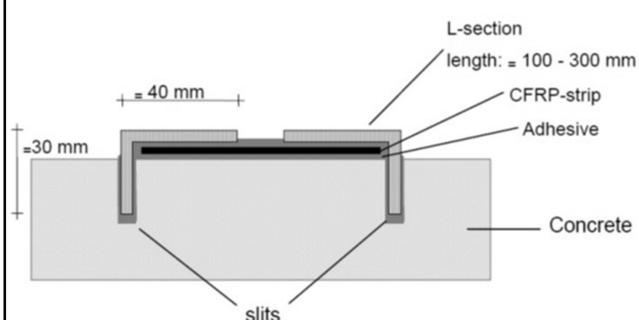
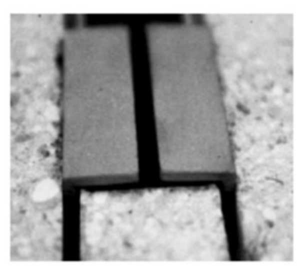
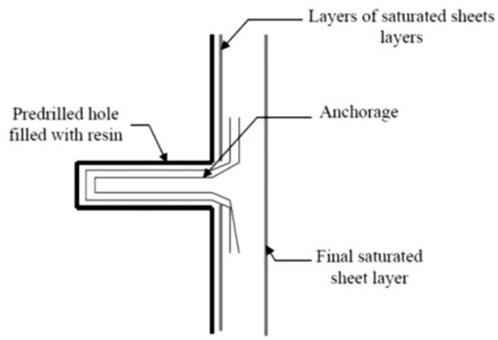


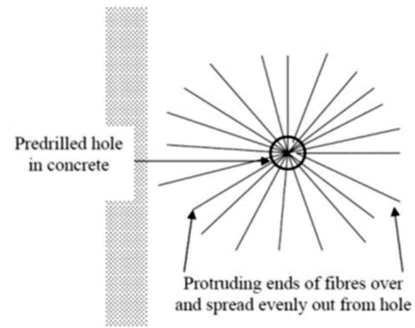


Anchorage for CFRP strips, special anchorage system
(Zehetmaier 2000)

fib Bulletin 14 - Externally bonded FRP reinforcement for RC structures



Section of the anchor system.



Top view of the anchor system

Internal FRP reinforcement

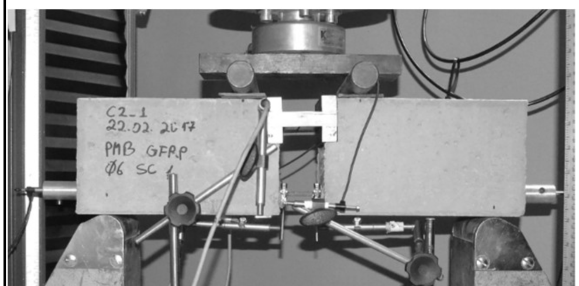
Bond of FRP bars ≠ Bond of steel RF

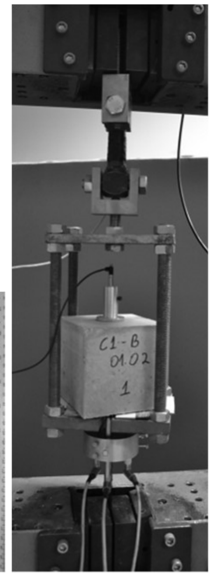
- Surface profile (and material) of FRP bars
- Modulus of elasticity of FRP bars
- Diameter of FRP bars ("shear lag"effect)
- Elevated temperature
- Concrete strength
- Environmental conditions

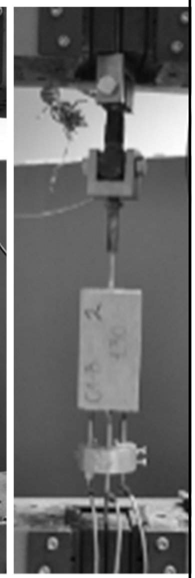
Test types

to study the bond behavior of internal bars

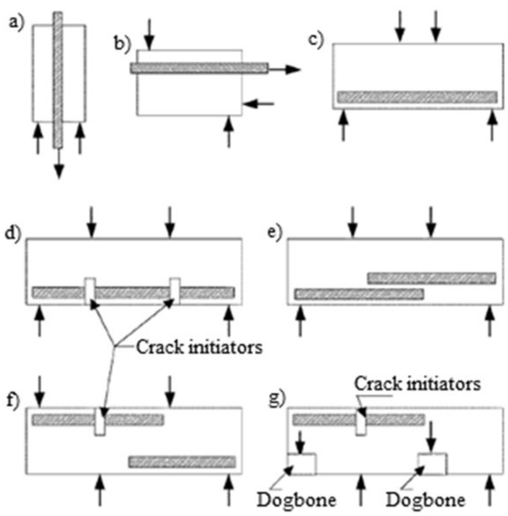
- ◉ Pull-out
- ◉ Direct tension
- ◉ Beam pull-out



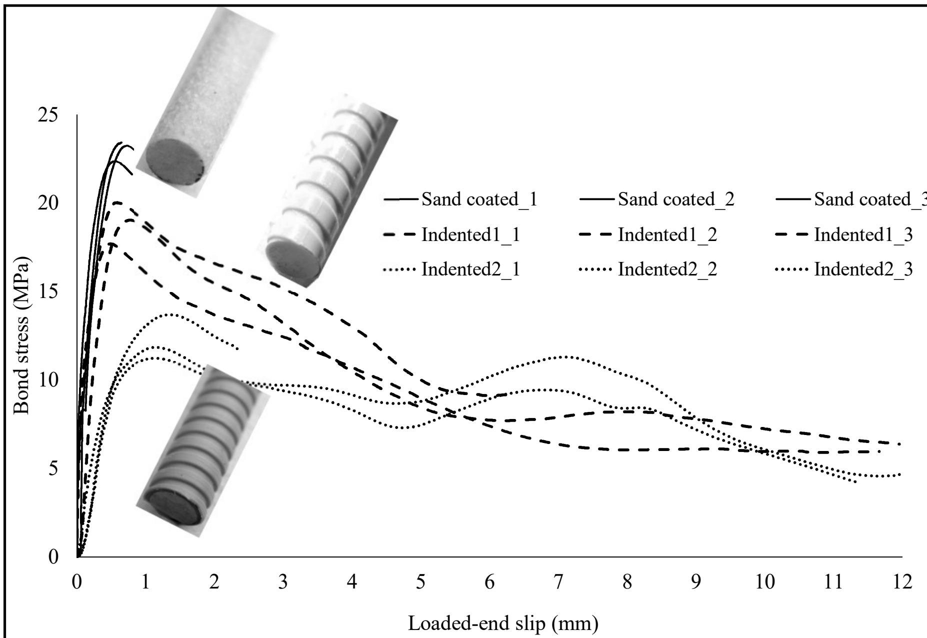




ACI 440.1R-15



Types of test methods for different bond values of FRP rebars in concrete: a) pull-out specimen; b) beam-end specimen; c) simple beam specimen; d) hinged beam-end specimen; e) splice specimen; f) cantilever beam specimen (without dogbones); and g) cantilever beam specimen (with dogbones). (ACI, 2015)



Analytical models

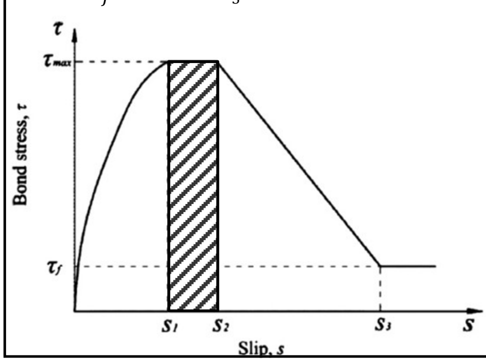
◉ BPE model (steel bars)

$$\frac{\tau}{\tau_{max}} = \left(\frac{s}{s_m}\right)^{\alpha} \qquad s < s_1$$

$$\tau = \tau_{max} \qquad s_1 < s < s_2$$

$$\frac{\tau}{\tau_{max}} = 1 - p * \left(\frac{s}{s_m} - 1\right) \qquad s_2 < s < s_3$$

$$\tau = \tau_f \qquad s > s_3$$



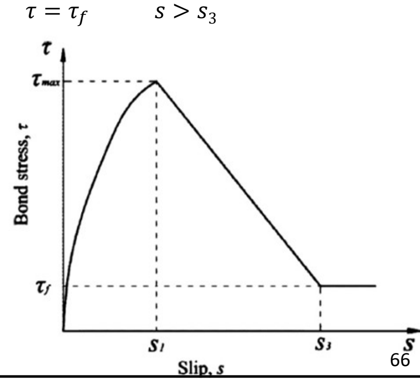
◉ modified BPE model (FRP bars)

$$\frac{\tau}{\tau_{max}} = \left(\frac{s}{s_m}\right)^{\alpha} \qquad s < s_1$$

$$\cancel{\tau = \tau_{max} \qquad s_1 < s < s_2}$$

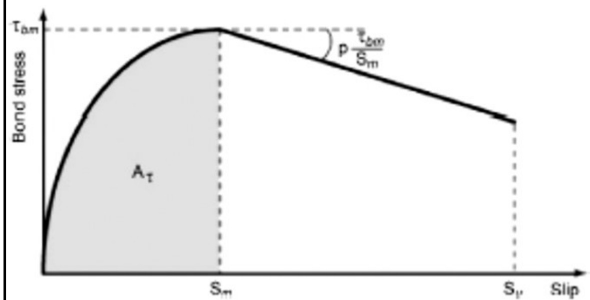
$$\frac{\tau}{\tau_{max}} = 1 - p * \left(\frac{s}{s_m} - 1\right) \qquad s_1 < s < s_3$$

$$\tau = \tau_f \qquad s > s_3$$



Local bond stress-slip model for internal FRP

fib Model Code 2010



$$\frac{\tau}{\tau_{max}} = \left(\frac{s}{s_m}\right)^{\alpha} \qquad s < s_1$$

$$\frac{\tau}{\tau_{max}} = 1 - p * \left(\frac{s}{s_m} - 1\right) \qquad s_2 < s < s_3$$

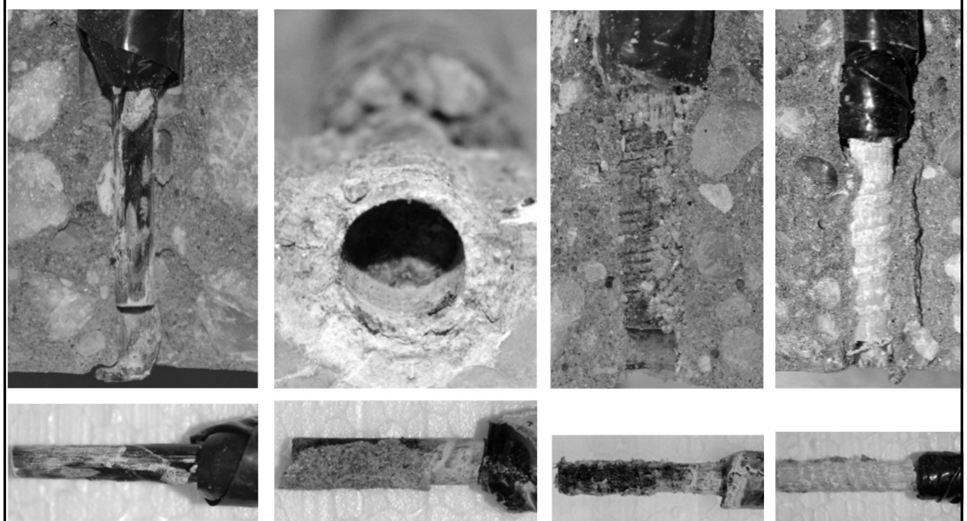
Bond strength and development length

- JSCE (1997) $l_d = \alpha_1 \kappa \frac{f_d}{4 f_{bod}} \Phi$ $f_{bod} = \frac{0.318 + 0.795\left(\frac{c}{\Phi} + \frac{15 A_t}{s\Phi}\frac{E_t}{E_s}\right)}{\frac{3.2}{\sqrt{f'_c}} - \frac{53.2}{f_y}}$

- CSA S806-02 (2002) $l_d = 1.15 \frac{K_1 K_2 K_3 K_4 K_5}{d_{cs}} \frac{f_F}{\sqrt{f'_c}} A_f$ $\tau_f = \frac{d_{cs}\sqrt{f'_c}}{1.15(K_1 K_2 K_3 K_4 K_5)\pi d_b}$

- ACI 440.1R-15 (2015) $l_d = \frac{\propto \frac{f_{fr}}{0.083\sqrt{f'_c}} - 340}{13.6 + \frac{C}{d_b}} d_b$ $\frac{u}{0.083\sqrt{f'_c}} = 4.0 + 0.3\frac{C}{d_b} + 100\frac{d_b}{l_e}$

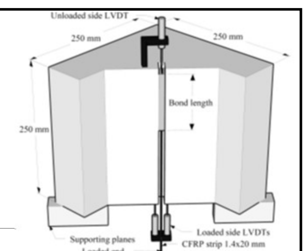
Bond failure modes

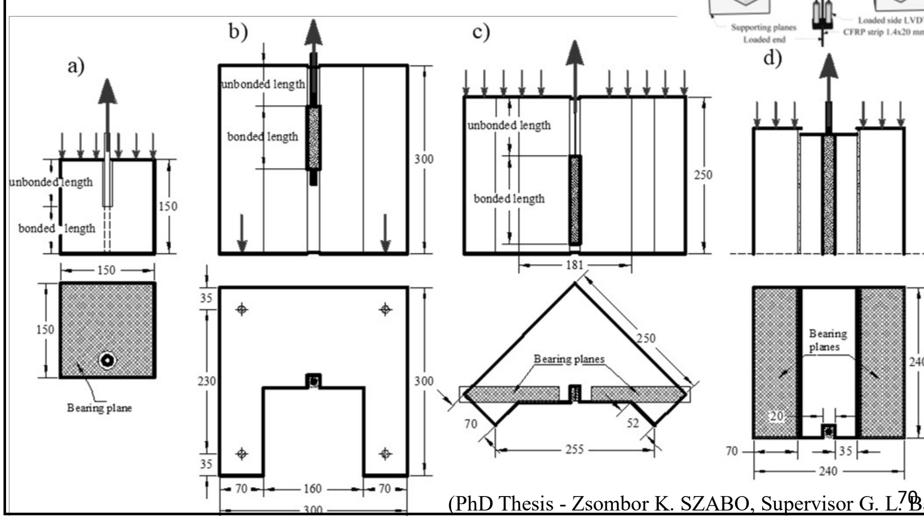


GFRP HW(Ø6) BFRP SC "fine"(Ø6) BFRP SC "rough"(Ø6) GFRP HW+SC(Ø8)

Test types

to study the bond behavior of **NSM reinforcement**





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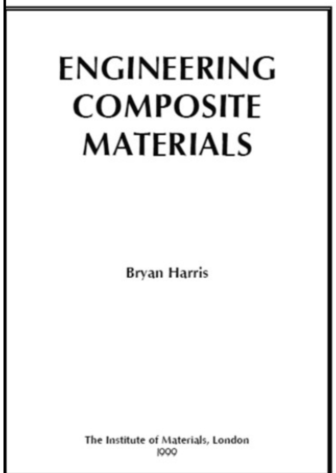

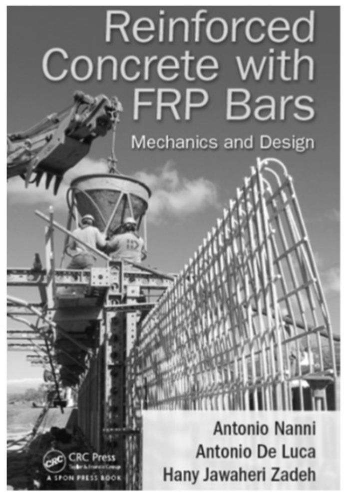

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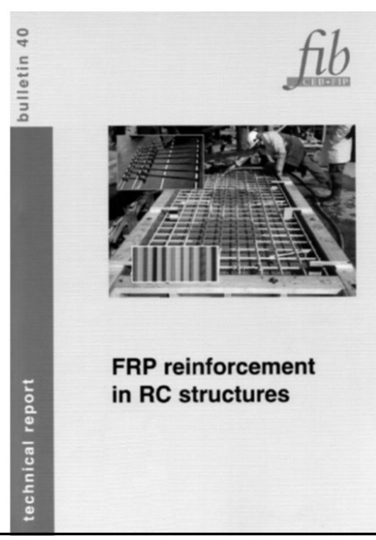

Further reading

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73

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fib-Short Course
REINFORCING & STRENGTHENING OF
STRUCTURES WITH ADVANCED COMPOSITES

Thank you for your kind attention

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Session 2

Flexural and shear strengthening and design aspects

Stijn Matthys

FACULTY OF ENGINEERING AND ARCHITECTURE

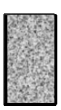
FLEXURAL AND SHEAR STRENGTHENING AND DESIGN ASPECTS

Prof. Stijn Matthys

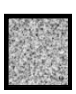
fib short course ‘Reinforcing & Strengthening of Structures with Advanced Composites

GHENT UNIVERSITY

► increase flexural capacity
increase stiffness (less deflections)

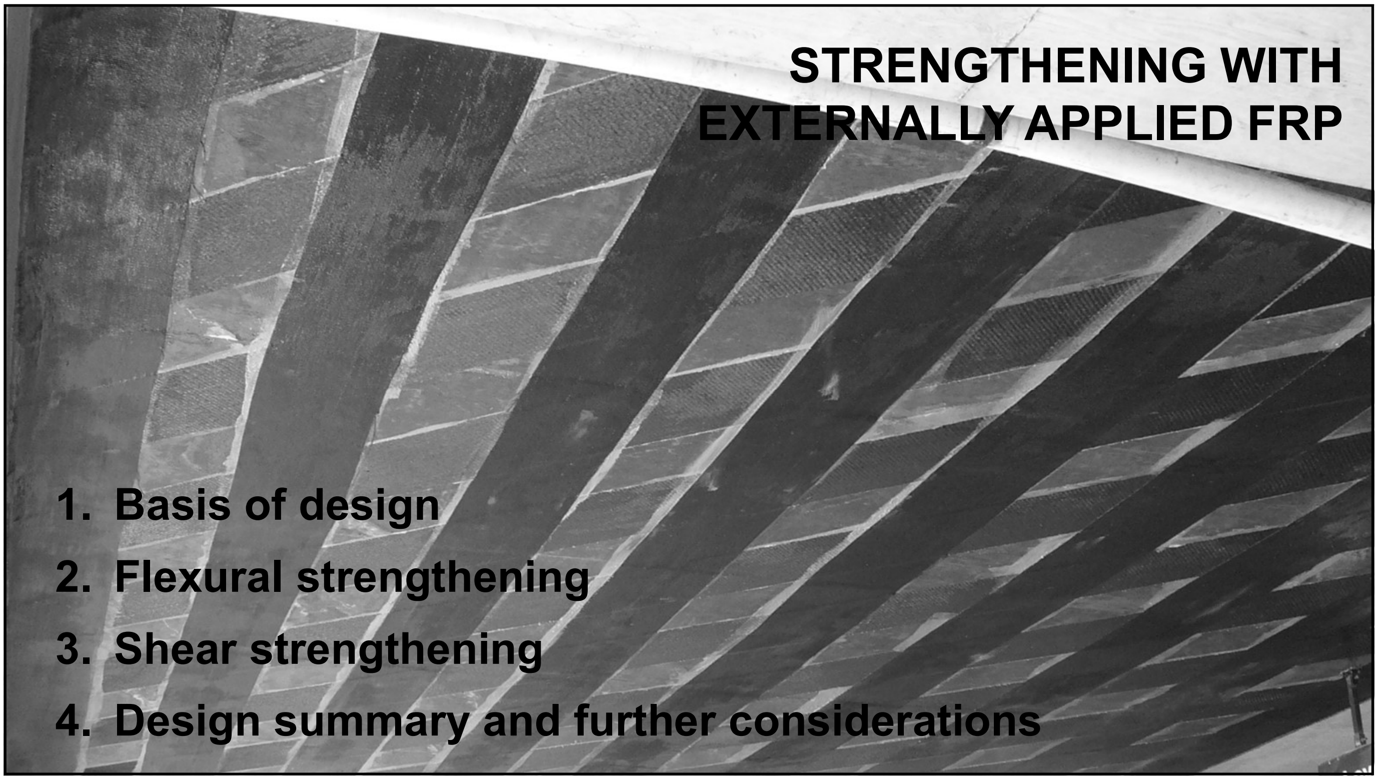


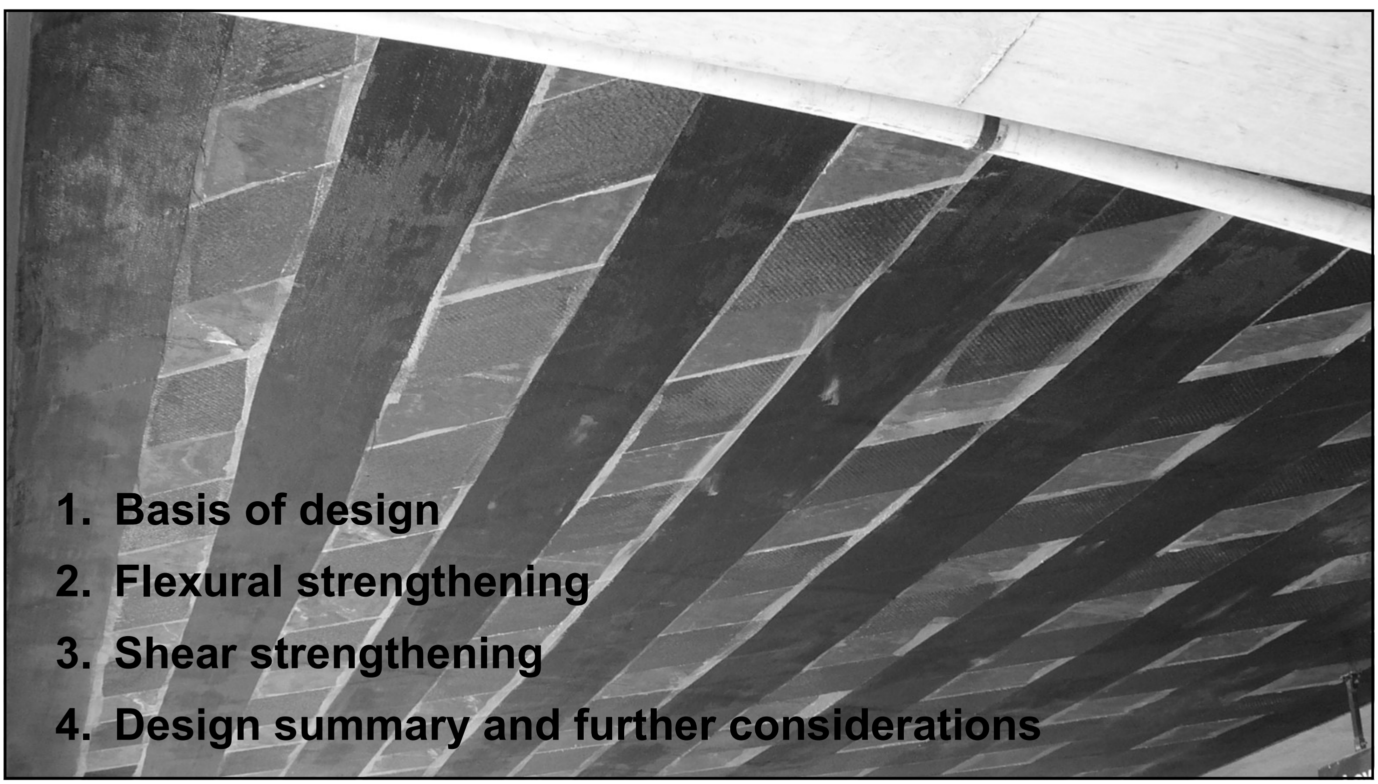
► increase shear capacity



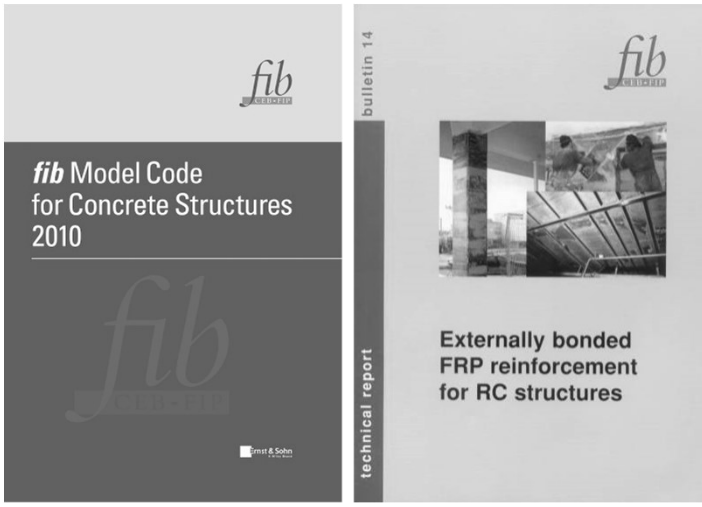
► confinement of columns

Vuurmolen Overijse (BE)


STRENGTHENING WITH
EXTERNALLY APPLIED FRP
1. Basis of design
2. Flexural strengthening
3. Shear strengthening
4. Design summary and further considerations


1. Basis of design
2. Flexural strengthening
3. Shear strengthening
4. Design summary and further considerations

Basis of design



This lecture focuses on the design aspects of externally bonded reinforcement by means of FRP. fib MC2010 and B14 (including its successor under press) are basically the reference for the design equations

5/26

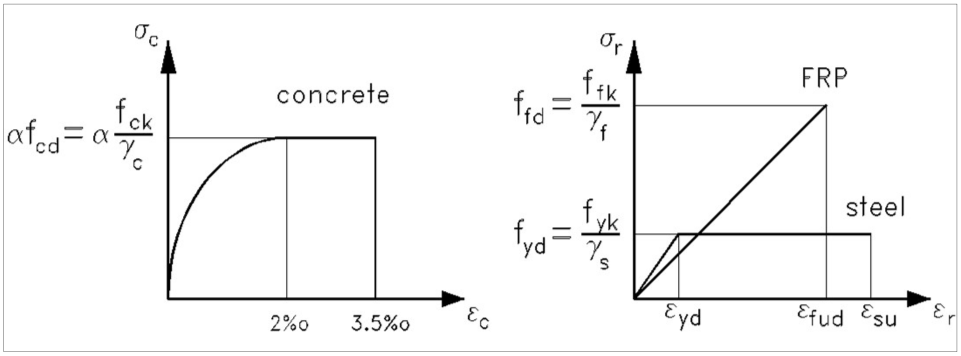
2 basic questions which cover the complete design

2 Critical Questions to Answer

1. What is the effect of the extra reinforcement, assuming that there is proper bond interaction?
2. How much force can the system transfer over the bond interface (debonding verification)?

6/26

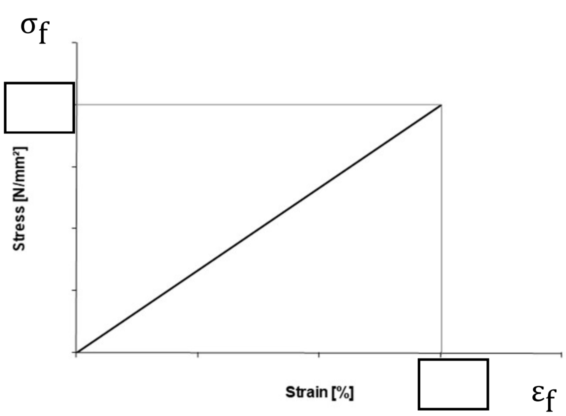
Constitutive material models to use in the design



$\gamma_c = 1.5$ - $\gamma_s = 1.15$ - $\gamma_f = 1.25$

7/26

Constitutive model for FRP in SLS



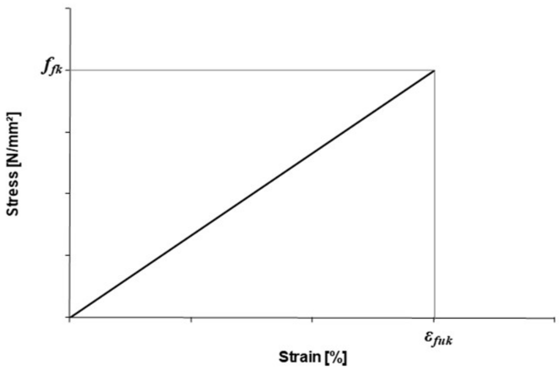
SLS (serviceability limit state)

$\sigma_f = E_f \varepsilon_f$

E_f = mean secans E-modulus of FRP

8/26

Constitutive model for FRP in ULS



ULS (ultimate limit state)

$$f_{fd} = \eta \frac{f_{fk}}{\gamma_f}$$

f_{fk} = characteristic tensile strength
Slope of the diagram → modulus f_{fk} / ε_{fuk}
ε_{fuk} = characteristic ultimate strain
η = ε_{fue} / ε_{fum} : effective ultimate strain factor (≤ 1)

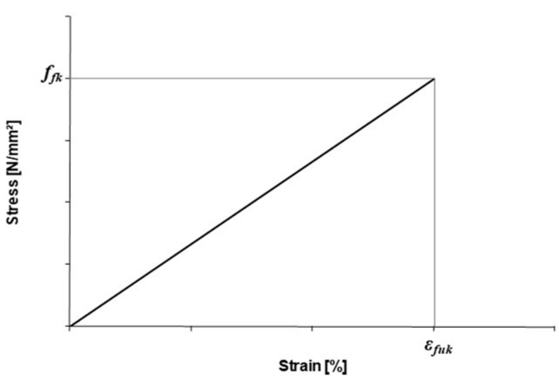
η → Can be taken 1 in most cases

Smaller than 1 to account for effective strain e.g. when wrapping sharp corners, high number of layers, multi-axial stress state, etc.

A limiting design strain can also been considered as a simplified design alternative. In this case, the ULS verification restricts excessive FRP deformations, rather than verifying the related failure mode itself

9/26

Constitutive model for FRP in ULS



ULS (ultimate limit state)

$$f_{fd} = \eta \frac{f_{fk}}{\gamma_f}$$

Partial safety factor γ_f

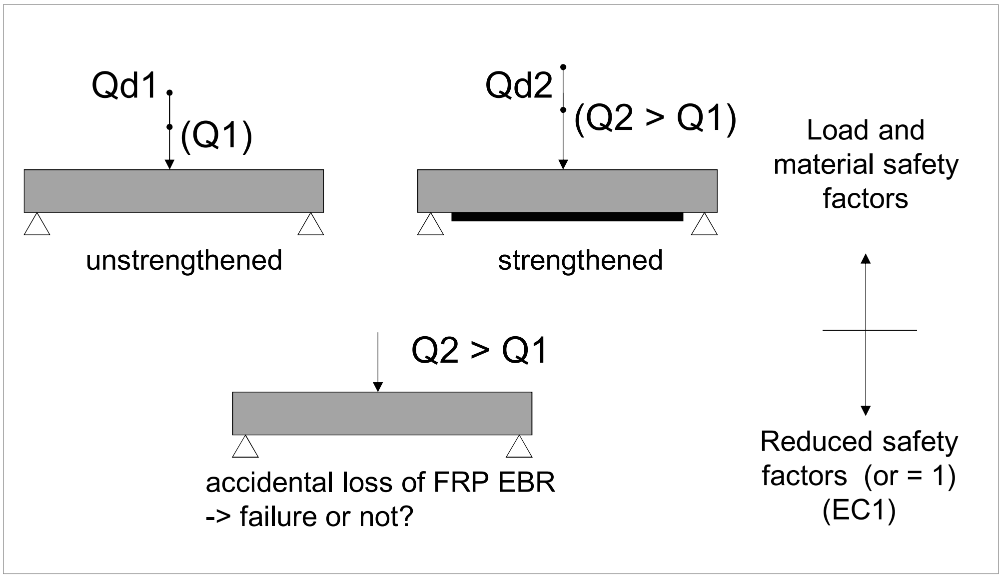
Design situation	Safety factor
Persistent/transient	1.25
Accidental	1.00

These safety factors imply that quality control provisions on the FRP materials and products, as well as their installation, are applied (fib gives specifications for that)

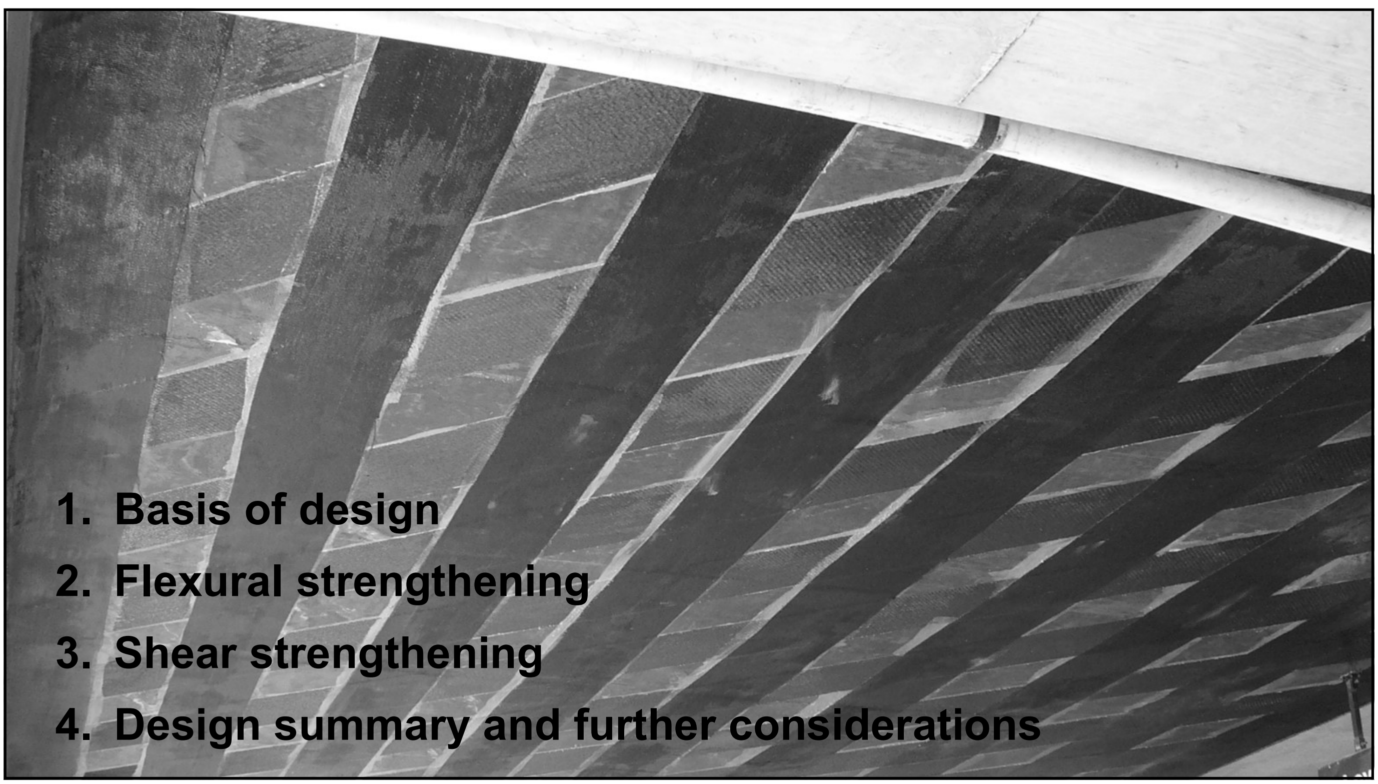
The safety factors adopted in seismic retrofitting are higher (see later teaching).

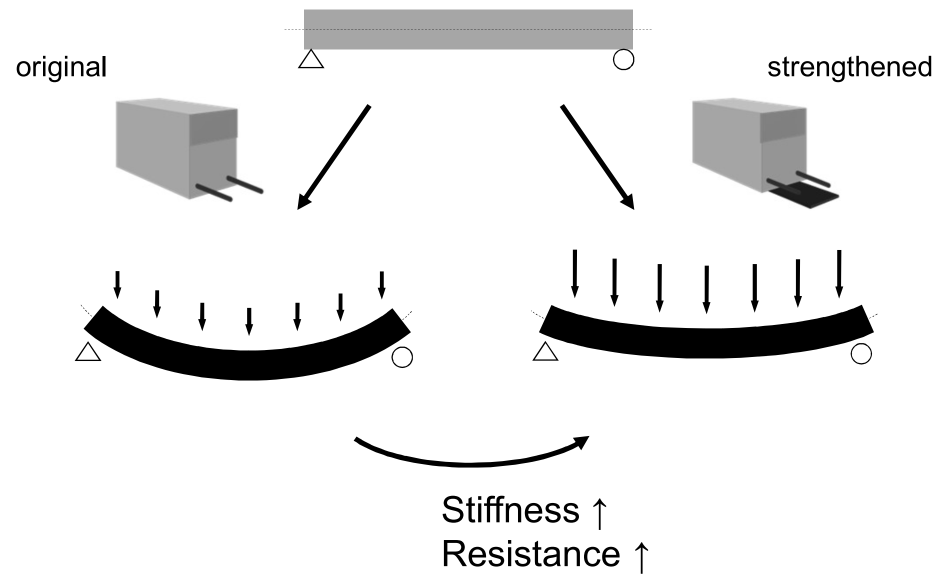
10/26

Accidental situation



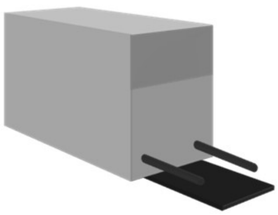
11/26

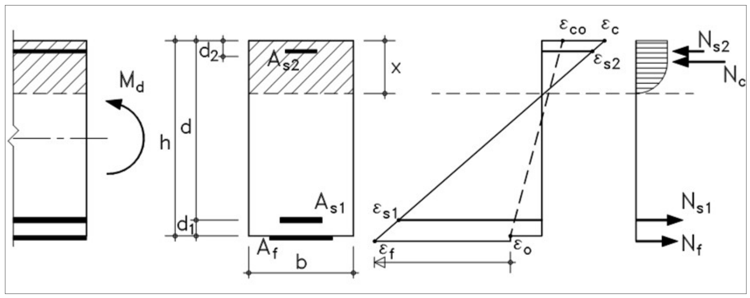




Flexural strengthening
DESIGN STEP 1: ULS full composite action

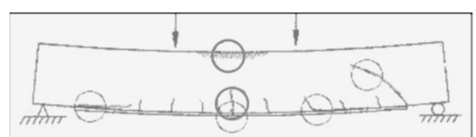
Design approach



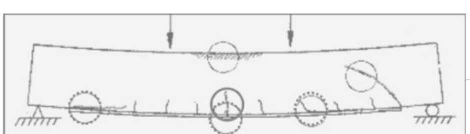


Traditional equilibrium equations at the basis

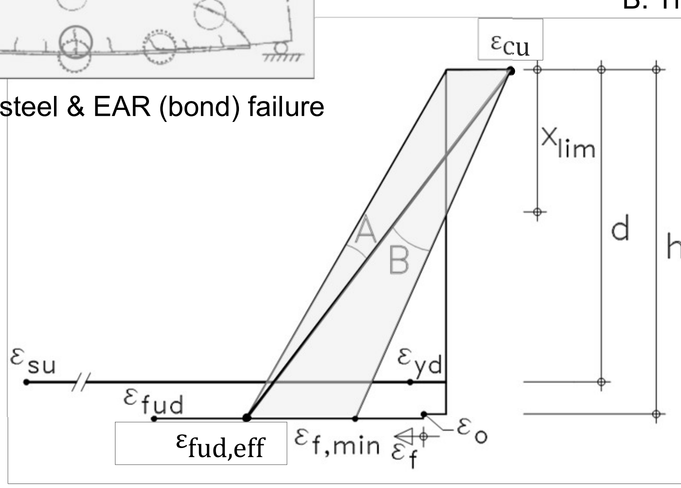
Design reflects on failure aspects



B. Yielding steel & concrete crushing



A. Yielding steel & EAR (bond) failure



Brittle failure at first cracking

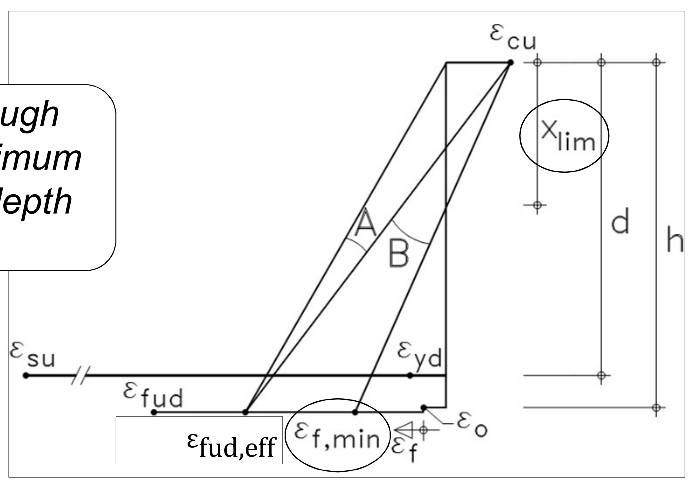
$$\rho_{s,min} = 0.095 \frac{f_{ck}^{2/3}}{f_{yk}}$$

- Going from uncracked section to cracked section: sufficient internal steel reinforcement available?
- Resisting moment > cracking moment

17/26

Deformability

Warning at ultimate through large deflections → minimum curvature (limitation of depth of compression zone)



18/26

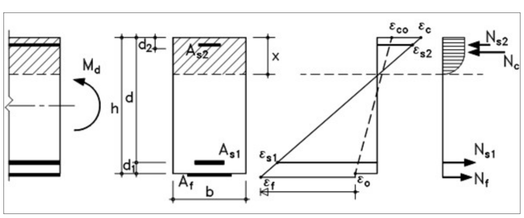
Deformability condition

	$\xi = x/d$ [-]	$\varepsilon_{f,min}$ [mm/m]	$\varepsilon_{s,min}$ [mm/m]	$\delta_{1/r,min}$ [-]
C35/45 or less	≤ 0.45	$5.0 - \varepsilon_o$	4.3	$\approx 0.0043/\varepsilon_{yk}$
Higher than C35/45	≤ 0.35	$7.5 - \varepsilon_o$	6.5	$\approx 0.0065/\varepsilon_{yk}$

Based on CEB-FIP Model Code 1990

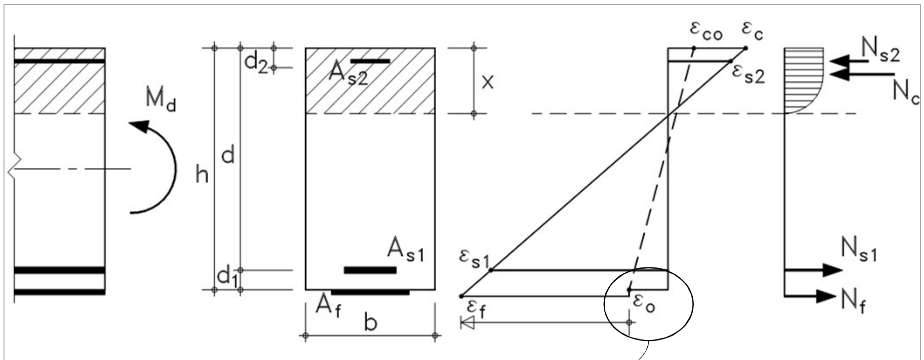
In case more FRP is required than needed for ULS → deformability condition can be ignored if the design moment is 25% over-dimensioned.

ULS design - full composite action



1. **Initial situation**
2. Cross-sectional analysis
3. (M-χ and enveloppe line)

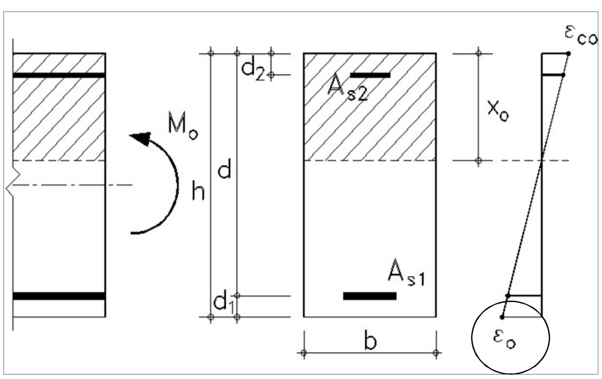
Initial situation



Initial strain in the concrete at the moment FRP is applied
(FRP takes only additional load on the member after strengthening)

21/26

Initial situation sometimes neglectable



Not cracked:
Initial deformation at strengthening can be neglected in design

Cracked:
Calculate ε_o via theory of elasticity (transformed cracked section)

22/26

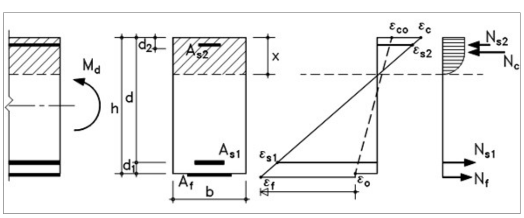
Initial situation calculation

Depth compression zone

Concrete strain at top fibre

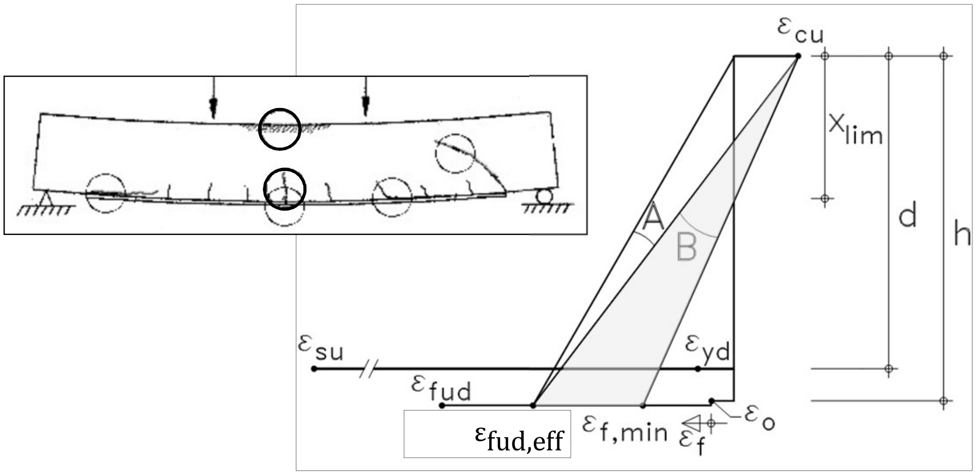
Concrete strain at extreme tension fibre

ULS design - full composite action



1. Initial situation
2. **Cross-sectional analysis**
3. (M-χ and enveloppe line)

Yielding steel & concrete crushing



25/26

Yielding steel & concrete crushing

Concrete stress block parameters

$\psi = 0.81$
$\delta_G = 0.416$

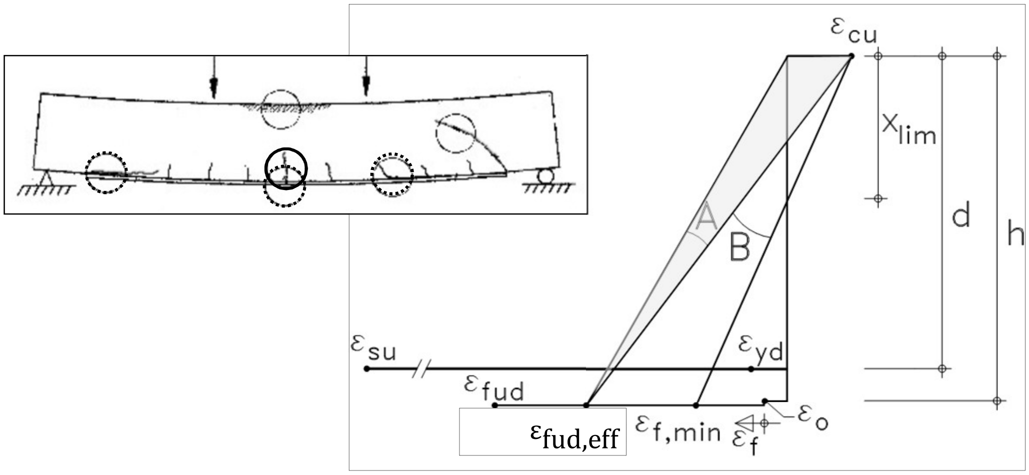
$\Sigma N = 0$

$\varepsilon_{f,min} \leq \varepsilon_f \leq \varepsilon_{fud,eff}$ → Verify that FRP failure is not governing

$\Sigma M = 0$

26/26

Yielding steel & FRP (bond) failure



27/26

Yielding steel & FRP (bond) failure

Concrete stress block parameters

$$\lambda = \frac{0.002}{\varepsilon_f + \varepsilon_o}\frac{h-x}{x}$$

$$\lambda \geq 1: \quad \psi = \frac{3\lambda - 1}{3\lambda^2} \quad \delta_G = \frac{4\lambda - 1}{4(3\lambda - 1)}$$

$$\lambda \leq 1: \quad \psi = 1 - \frac{\lambda}{3} \quad \delta_G = \frac{\lambda^2 - 4\lambda + 6}{4(3-\lambda)}$$

$\Sigma N = 0$

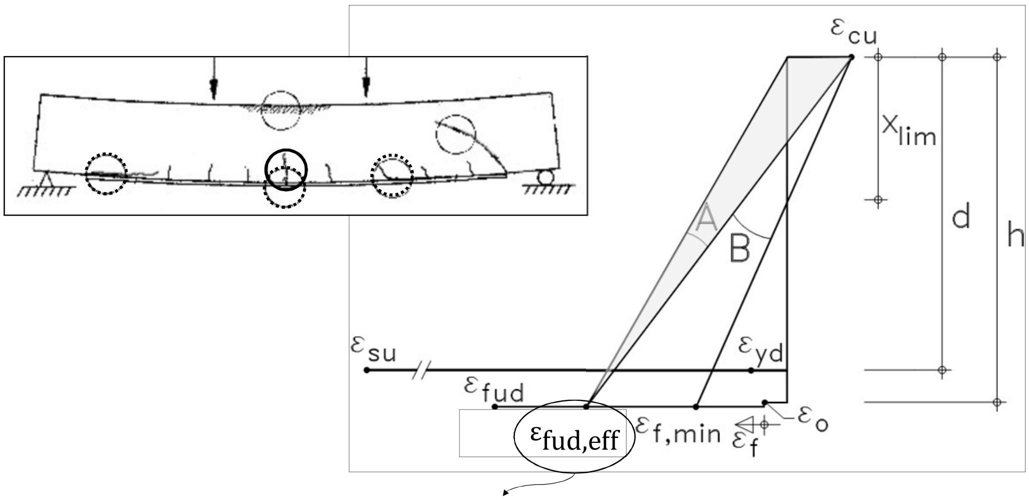
$$\varepsilon_{s2} = (\varepsilon_{fud,eff} + \varepsilon_o)\frac{x - d_2}{h - x} \leq \frac{f_{yd}}{E_s}$$

$$\varepsilon_f = \varepsilon_{fud,eff}$$

$\Sigma M = 0$

28/26

Governing FRP strain at ULS?



Which ultimate FRP strain to assume for the ULS?
Ultimate strain ε_{fud} would be logic, but debondig may occur somewhere along the FRP length. We can anticipate for this by considering a lower strain limit $\varepsilon_{fud,eff} < \varepsilon_{fud}$

29/26

Simplified FRP strain limit method

EBR (externally bonded reinforcement)

$$\varepsilon_{fud,\,eff} = \frac{k_{cr,k} k_k k_b \sqrt{\frac{2E_f}{t_f} f_{cm}^{2/3}}}{\gamma_{fb} E_f} \longrightarrow$$

Stress @ intermediate crack bond capacity

$$\leq \eta \frac{f_{fk}}{\gamma_f E_f} = \frac{f_{fd}}{E_f}$$

$\gamma_{fb} = 1.5$ (and $\eta = 1$)

$k_{cr,k}=1.8$, $k_k=0.17$, $k_b = \sqrt{(2 - b_f/b)/(1 + b_f/b)} \geq 1$

NSM (near surface mounted)

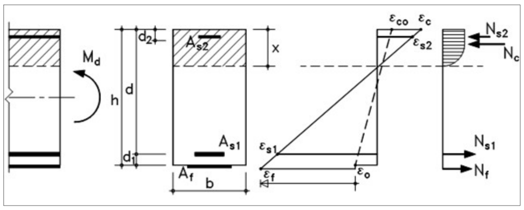
$$\varepsilon_{fud,\,eff} = \eta \frac{f_{fk}}{\gamma_f E_f} = \frac{f_{fd}}{E_f}$$

$\eta = 0.8$

Simple level of approximation: it can be considered to verify debonding more into detail in later design steps.

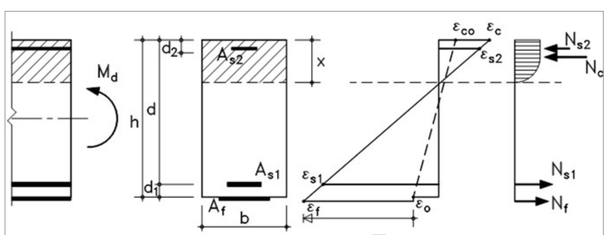
30/26

ULS design - full composite action



1. Initial situation
2. Cross-sectional analysis
3. **(M-χ and enveloppe line)**

Summary of cross-sectional analysis



Strain compatibility

Equilibrium of forces

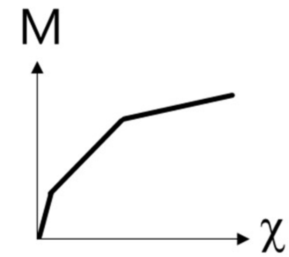
→ solve equation(s) with unknowns ε_c and x_c

→ following all strain and force components are known, and bending moment and curvature can be calculated as well

M-χ diagram

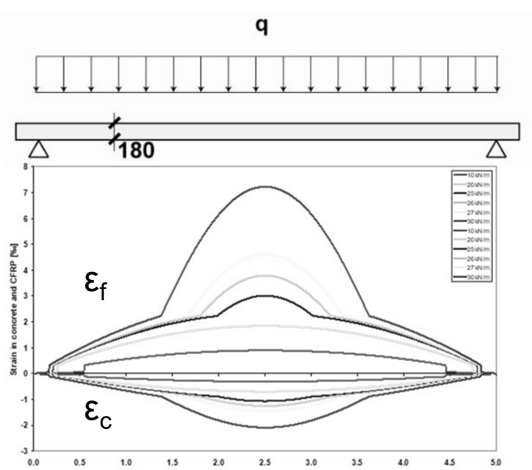
ε_c (mm/m)	x_c (mm)	ε_{s1} (mm/m)	ε_f (mm/m)	χ (1/m)	M (kNm)
0					
0.05					
0.10					
0.15					
…					
3.40					
3.45					
3.50					

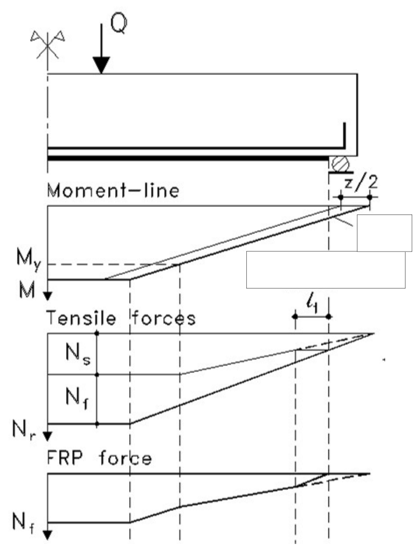
→



With increasing ε_c:
→ uncracked,
→ cracked and steel elastic
(excluding tension stiffening)
→ cracked and steel yielding

Envelope line along beam

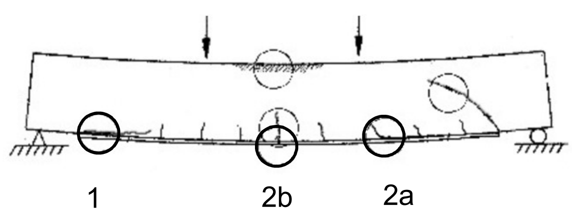




Flexural strengthening
DESIGN STEP 2: ULS bond verification

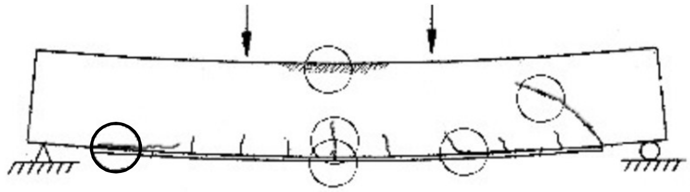
Debonding verification

1. Anchorage zone (end debonding)
2. Debonding at intermediate cracks



36/26

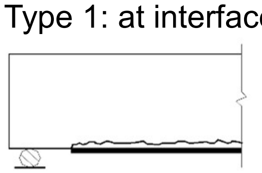
Debonding verification



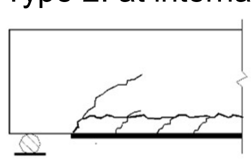
1. Anchorage zone (end debonding)
2. Debonding at intermediate cracks

Anchorage failure aspects

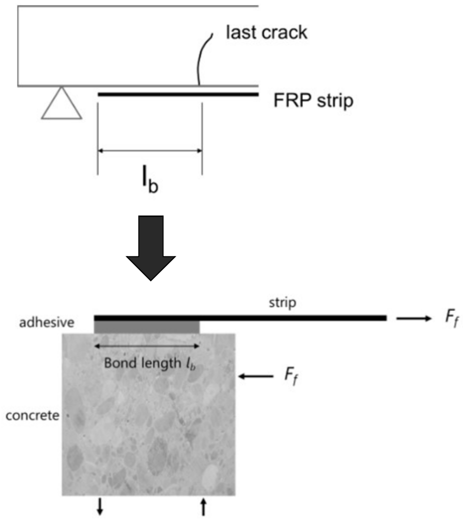
Type 1: at interface

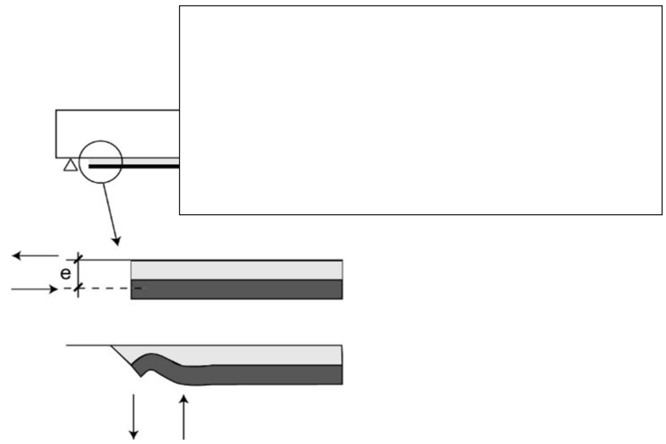


Type 2: at internal steel reinforcement level



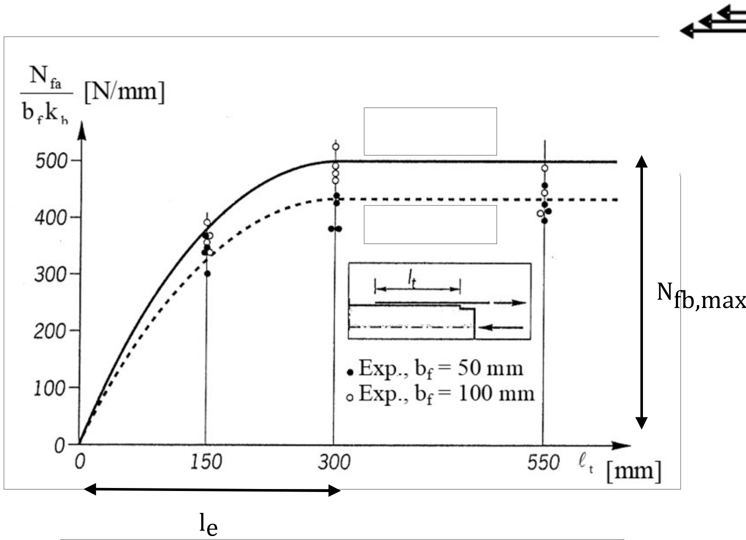
Anchorage failure type 1



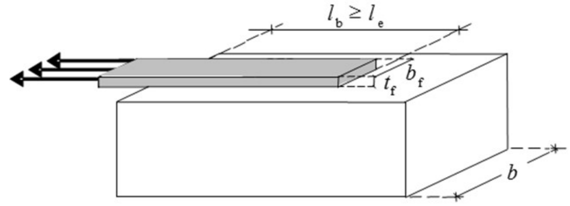


Not only shear, but also normal stresses at the end (not explicitly taken into account at ULS debonding)

Anchorage failure type 1



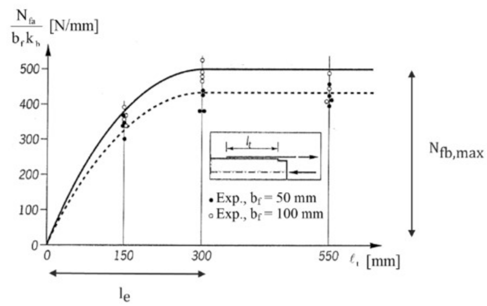
Effective bond length



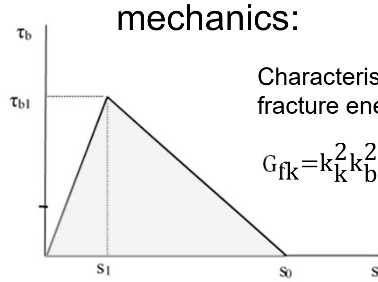
$$N_{fb}(l_b) = \beta_l(l_b) N_{fb,max}$$

$$\beta_l = \begin{cases} \frac{l_b}{l_e}\left(2 - \frac{l_b}{l_e}\right) < 1 & \text{if } l_b < l_e \\ 1 & \text{if } l_b \geq l_e \end{cases}$$

Anchorage failure type 1



Bond-slip fracture mechanics:



Characteristic value of fracture energy

$G_{fk}=k_k^2 k_b^2 f_{cm}^{2/3}$

$k_b = \sqrt{(2-b_f/b)/(1+b_f/b)} \geq 1$

$k_k = k_{0.05} = 0.17$
(surface bonded CFRP)

$\gamma_{fb} = 1.5$ (for design values)

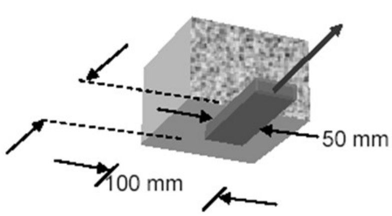
[N] & [mm]

$$N_{fb,max} = b_f\sqrt{2E_f t_f G_f} \longrightarrow f_{fbk} = k_k k_b \sqrt{\frac{2E_f}{t_f} f_{cm}^{2/3}}$$

$$l_e = \pi s_0 \sqrt{\frac{E_f t_f}{8G_f}} \longrightarrow l_{ek} = \frac{0.278}{k_k k_b}\sqrt{\frac{E_f t_f}{f_{cm}^{2/3}}}$$

41/26

Anchorage failure type 1
Example



Given: CFRP (f_{fk} = 2800 N/mm²)
E_f = 180000 N/mm²
t_f = 1.2 mm
f_{cm} = 30 N/mm²

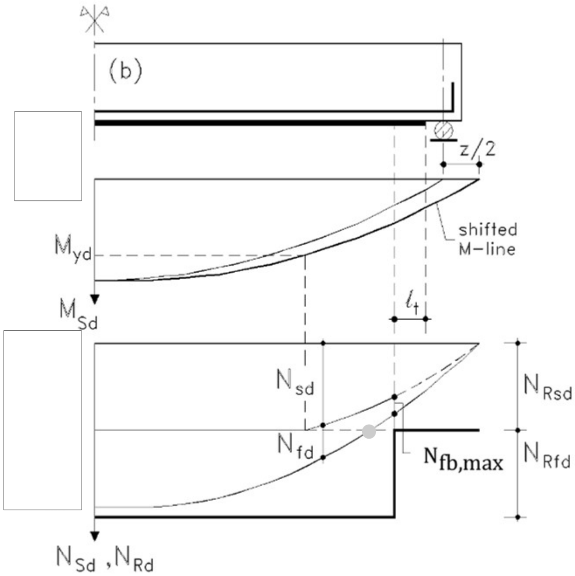
$N_{fbk,max} = 17{,}4\text{kN}$
$l_{ek} = 245$ mm

⬌

FRP rupture:
N_{fuk} = 168.0 kN

42/26

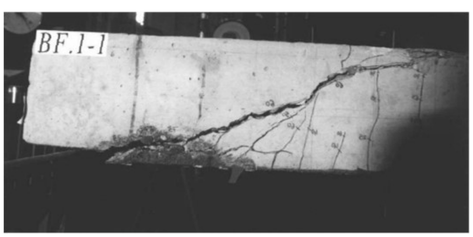
Anchorage failure type 1
Curtailment

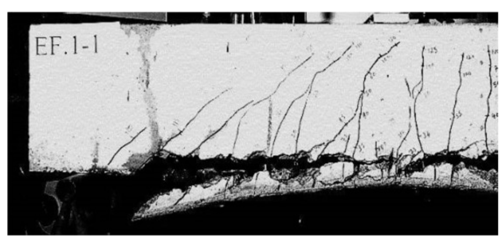


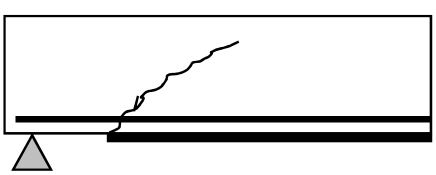
Curtailment on the basis of required tensile reinforcement in ULS. In case the FRP tensile reinforcement is also needed for e.g. deflection control, curtailment will be rather on the basis of the cracking moment (to increase the stiffness over the complete length of the cracked zone).

43/26

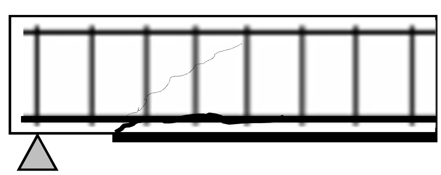
Anchorage failure type 2







No shear reinforcement:
EBR end <u>shear failure</u>

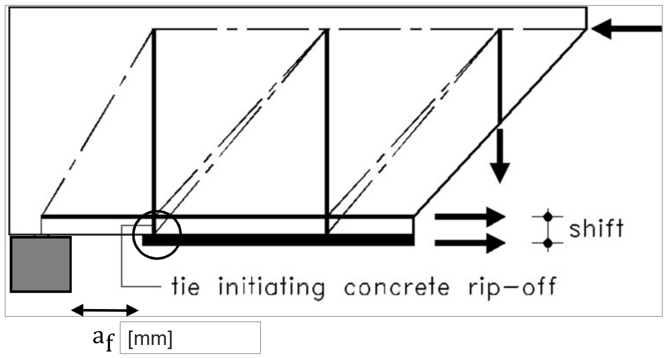


With shear reinforcement:
<u>concrete rip-off</u>

44/26

Anchorage failure type 2

Concrete rip-off mechanism



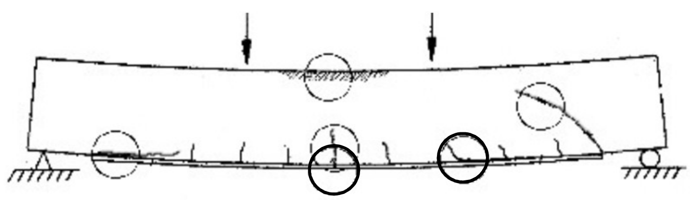
Preventing concrete ripp-off:

- $a_f \downarrow$
- Shear strap

$$V_{Rd,c,fe} = 0.75\left[1 + 19.6\frac{(100\rho_s)^{0.15}}{a_f}\right]V_{Rd,c}$$

45/26

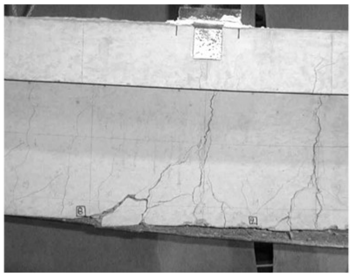
Debonding verification



1. Anchorage zone (end debonding)
2. Debonding at intermediate cracks

46/26

LoA intermediate crack (IC) debonding

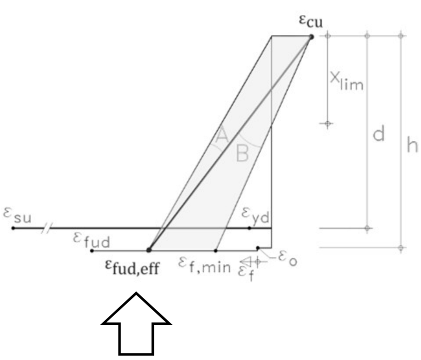


Level of Approximation (LoA)

1. Simplified method: overall strain limitation
2. Simplified models with respect to shear stress limitations
3. Detailed iterative procedure to evaluate shear stress due to crack bridging

47/26

LoA1: overall strain (or stress) limitation



$$\sigma_{fd} = E_f \varepsilon_{fud,eff}$$

$$\sigma_{fd} = \min(f_{fbd,IC}, f_{fd})$$

Design value of EAR tensile capacity

Design value of EAR related to bond shear capacity

IC bridging factor

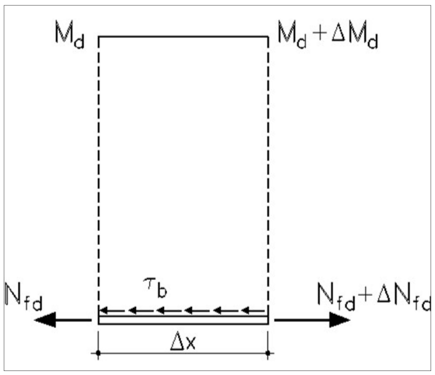
FRP anchorage capacity (see before)

$$f_{fbd,IC} = \frac{k_{cr,k} k_k k_b \sqrt{\frac{2E_f}{t_f} f_{cm}^{2/3}}}{\gamma_{fb}}$$

$k_{cr,k}$=1.8, k_k=0.17, $k_b = \sqrt{(2 - b_f/b)/(1 + b_f/b)} \geq 1$

48/26

LoA2 – force transfer

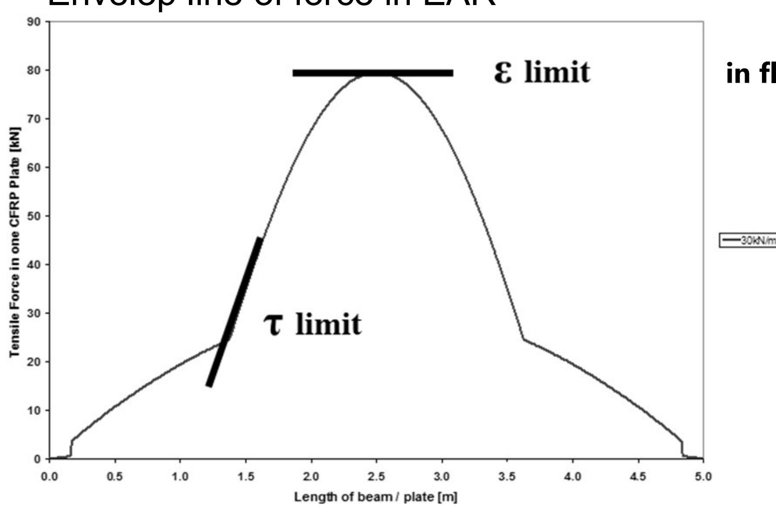


Based on cracked situation

$< \quad f_{cbd} = 1.8\frac{f_{ctk}}{\gamma_c}$

49/26

Envelop line of force in EAR

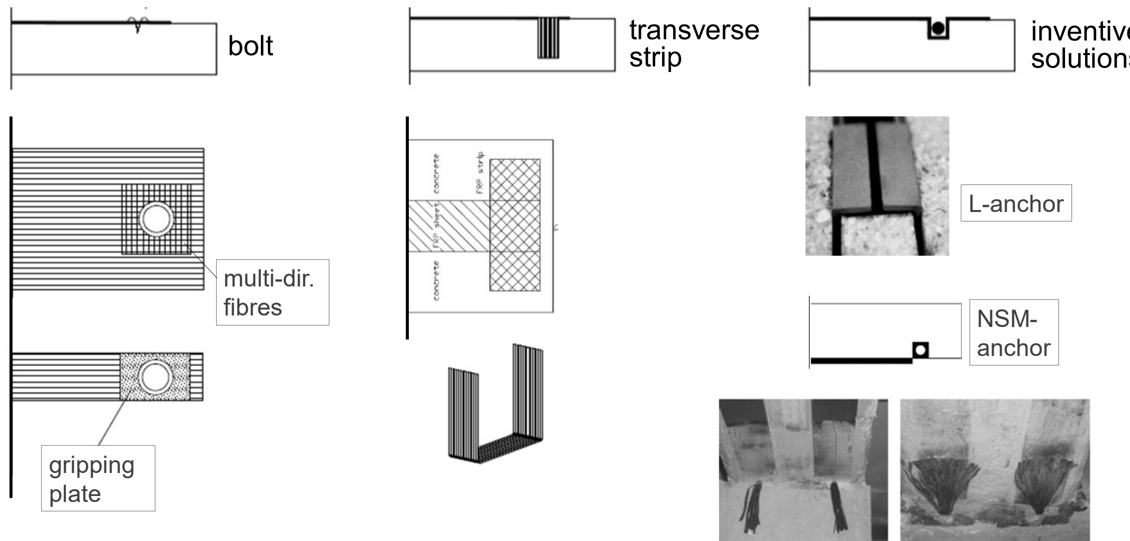


in flexural cracking zone

in zone with high stress due to force transfer (cracked state)

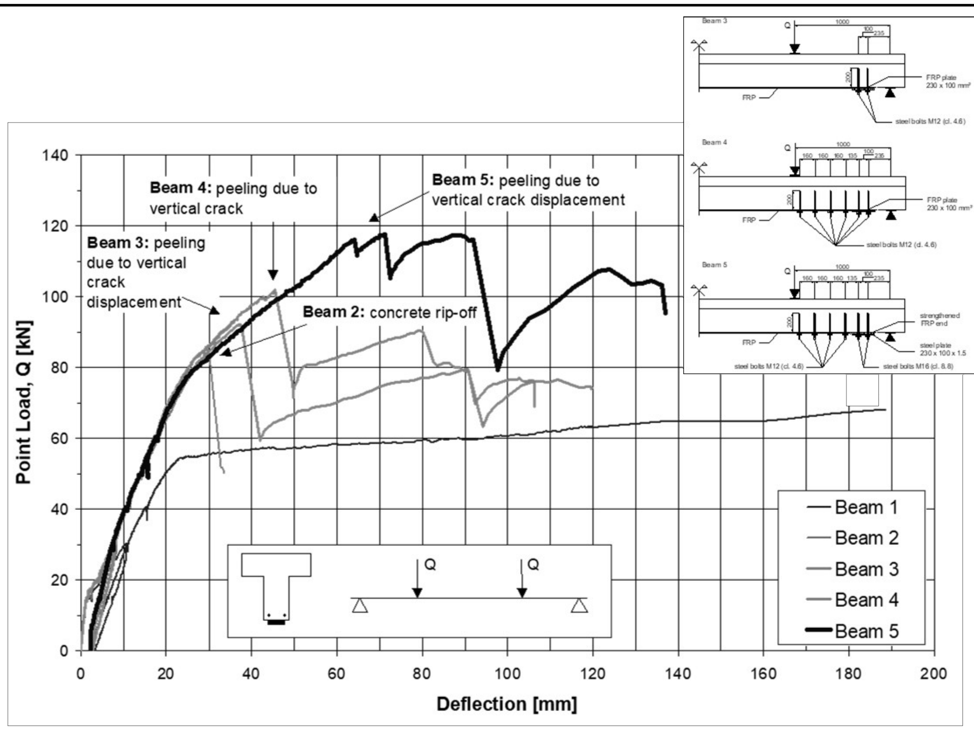
50/26

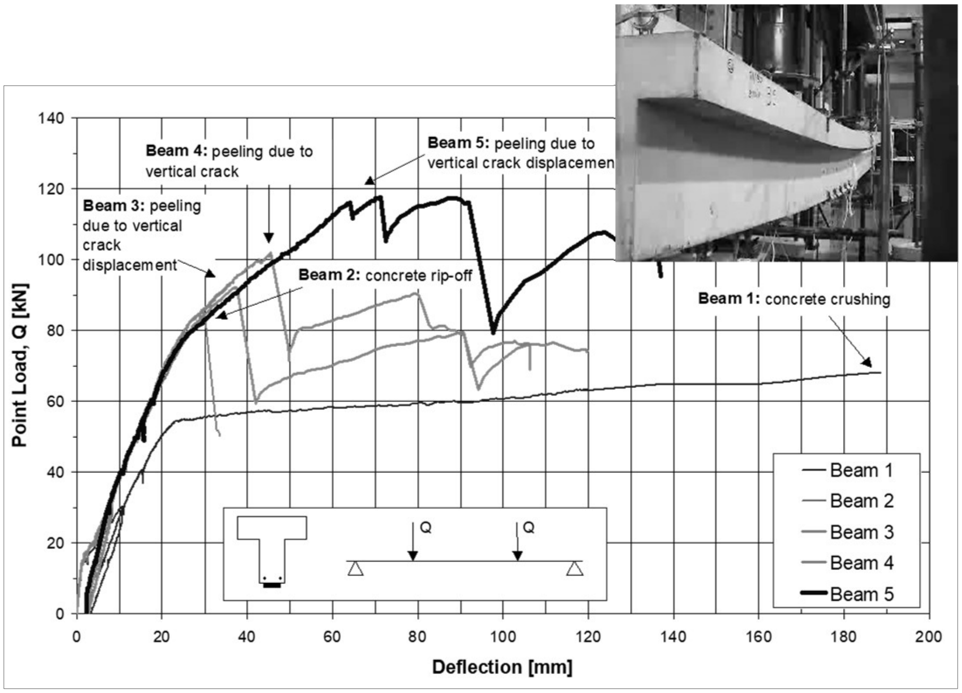
Applying mechanical anchorages



fan-anchor (also called spike-anchor)

Design by testing





53/26

Flexural strengthening
DESIGN STEP 3: SLS

Design in the SLS

SLS often governing for the design

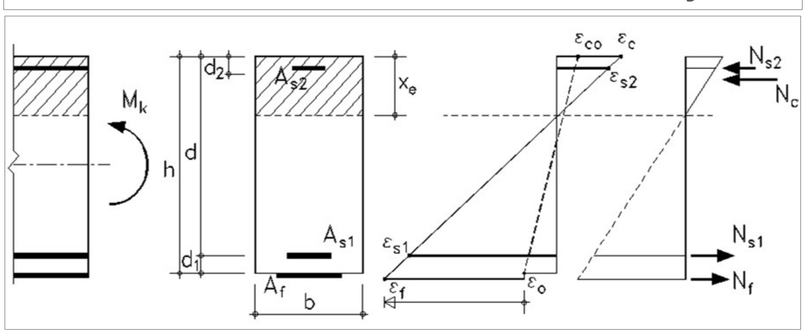
- Limited section (A) needed for ULS
- Stiffness (EA) may be insufficient for SLS

For beams strengthened in flexure:

- Limitation of concrete, steel and FRP stress
- Limitation of deflections
- Limitation of crack widths
- (Check for local debonding initiation)

Serviceability limit state

Basis of calculation: linear elastic analysis



$$\frac{1}{2}bx_e^2 + (\alpha_s - 1)A_{s2}(x_e - d_2) = \alpha_s A_{s1}(d - x_e) + \alpha_f A_f\left(h - \left(1 + \frac{\varepsilon_o}{\varepsilon_c}\right)x_e\right)$$

$$\frac{\varepsilon_o}{\varepsilon_c} \approx \frac{M_o}{M_k}\frac{h - x_o}{x_e}$$

Cracking moment

$$M_{cr} \approx f_{ctm} \frac{bh^2}{6}$$

Moment of inertia of uncracked (1) and cracked (2) section

$$I_1 \approx \frac{bh^3}{12}$$

$$I_2 = bx_e^3/3 + (\alpha_s - 1)A_{s2}(x_e - d_2)^2 + \alpha_s A_{s1}(d - x_e)^2 + \alpha_f A_f (h - x_e)^2$$

SLS – stress verification

$$\begin{aligned} &\sigma_c \le 0.60 f_{ck} \quad \text{under the rare load combination} \\ &\sigma_c \le 0.45 f_{ck} \quad \text{under the quasi - permanent load combination} \end{aligned}$$

$$E_c \varepsilon_c = \frac{M_k}{1/2\, bx_e \left(h - \frac{x_e}{3}\right) + (\alpha_s - 1)A_{s2} \frac{x_e - d_2}{x_e}(h - d_2) - \alpha_s A_{s1} \frac{d - x_e}{x_e}(h - d)}$$

$$\sigma_s = E_s \varepsilon_c \frac{d - x_e}{x_e} \quad \le 0.80 f_{yk} \quad \text{under the rare load combination}$$

$$\sigma_f = E_f \left(\varepsilon_c \frac{h - x_e}{x_e} - \varepsilon_o \right) \quad \le \eta f_{fk} \quad \text{under the quasi - permanent load combination}$$

η = 0.80 voor CFRP

SLS – verification of deflection

$$a = a_1(1-\zeta_b) + a_2\zeta_b$$

$$\zeta_b = 0 \qquad M_k < M_{cr}$$

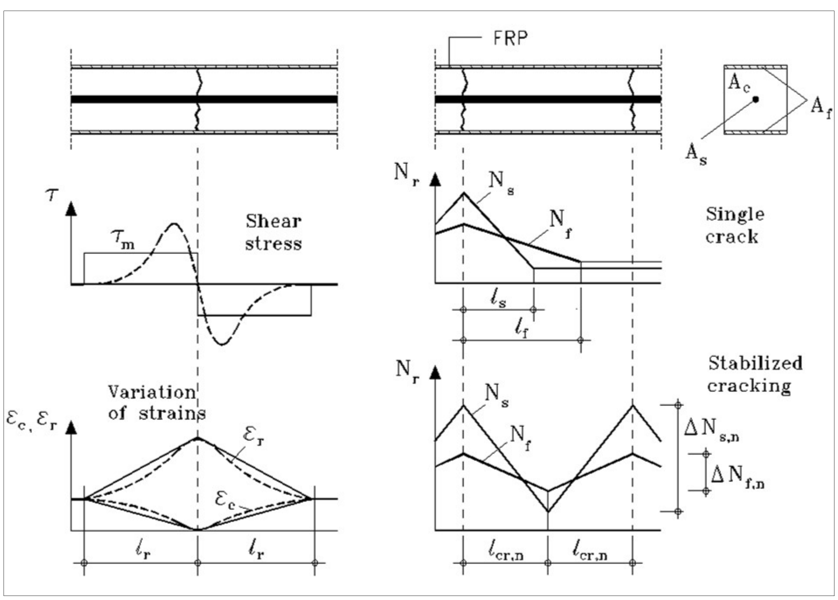
$$\zeta_b = 1 - \beta_1\beta_2\left(\frac{M_{cr}}{M_k}\right)^{n/2} \qquad M_k > M_{cr}$$

Tension stiffening factor (details see Eurocode 2)

$$a_1 = k_M \ell^2 \frac{M_k}{E_c I_1}$$

$$a_2 = k_M \ell^2 \left(\frac{M_o}{E_c I_{o2}} + \frac{M_k - M_o}{E_c I_2}\right) \quad M_k > M_o$$

SLS - verification of crack width



Calculation of crack width

Different bond behaviour of 2 reinforcing materials → modelling approach according to MC90 (combination of rebars and prestressing reinforcement).

fib Bulletin provides a detailed procedure.

These crack models can be reformulated (making some assumptions), as follows (Matthys):

$w_k \leq 0.3$ mm →

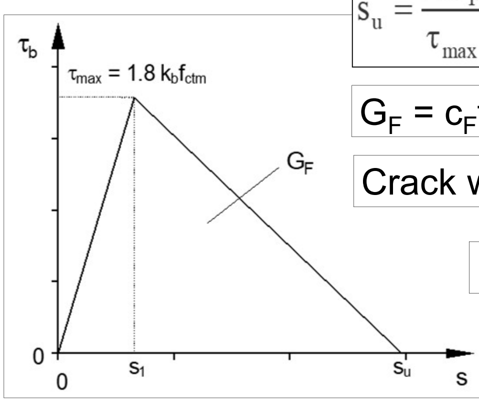
$$u_f \geq 10.1\rho_{c,eff}\frac{M_k}{E_s d\rho_{eq}} - 1.44u_s$$

$\rho_{c,eff} = A_{c,eff}/bd$ $\rho_{eq} = \rho_s + \rho_f E_f/E_s$

$u_f = b_f$ (b_f total width of the bonded FRP)

61/26

SLS - interface cracking



$$s_u = \frac{2G_F}{\tau_{max}}$$

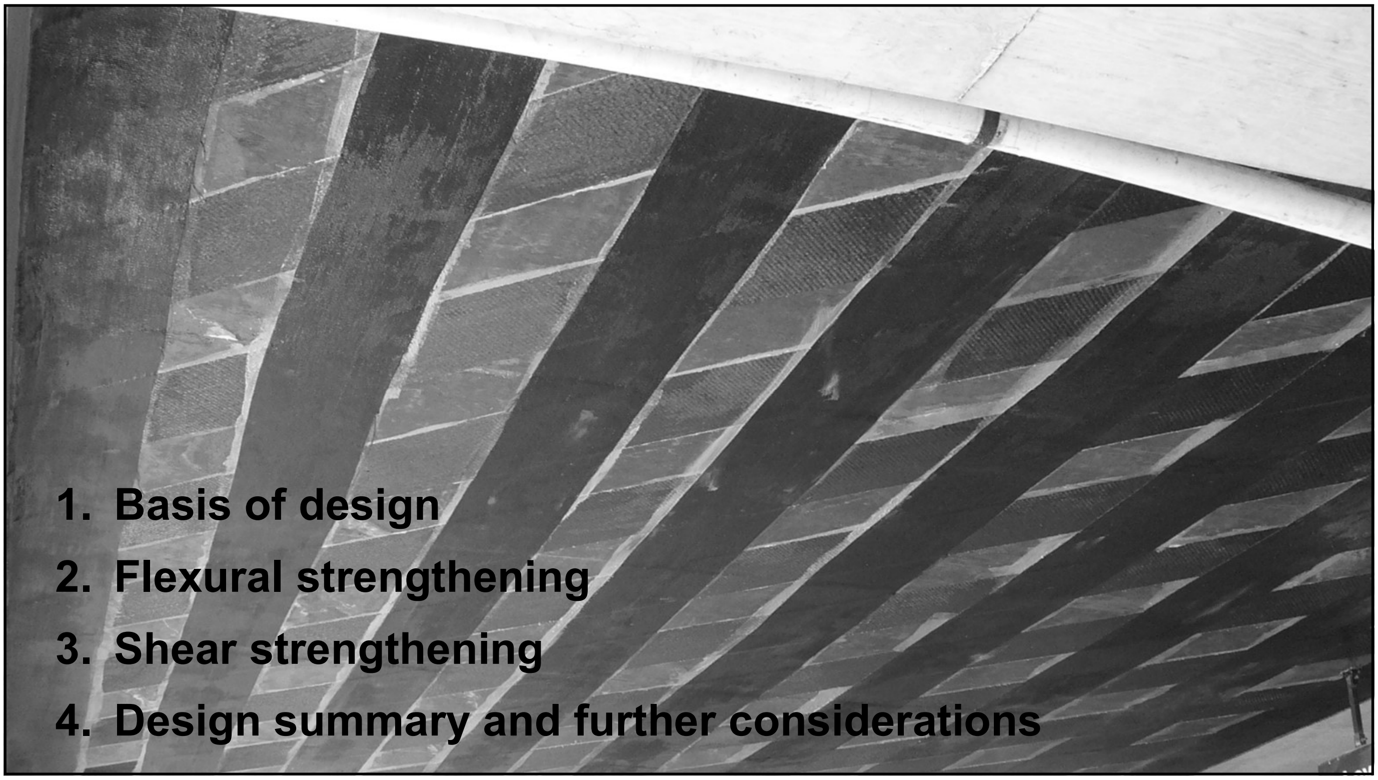
$G_F = c_F f_{ctm}$, $\tau_{max} = 1.8 f_{ctm}$ and $c_F = 0.202$ mm

Crack width $w = 2s_u = 0.45$ mm

MC90: $w_{k0.95} < 0.3$ mm → $w_m < 0.2$ mm

If crack limitations are fulfilled, local debonding at cracks is not an issue in SLS

62/26



1. Basis of design
2. Flexural strengthening
3. Shear strengthening
4. Design summary and further considerations

64/26

65/26

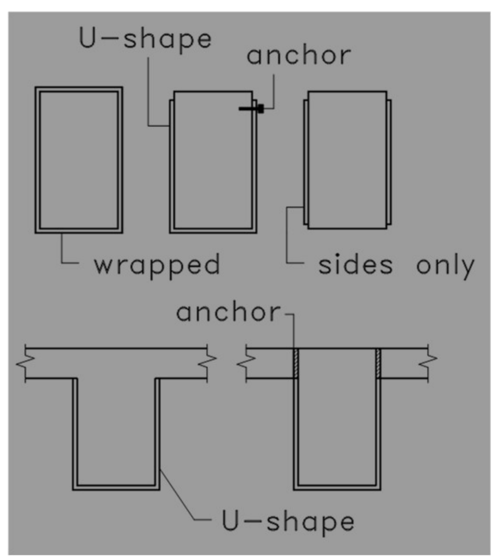
Shear strengthening configurations

- Ideal:
  - ► wrapped
  - ► anchored
- Practical:
  - ► U-shaped
  - ► sides only

more efficient

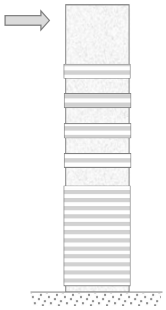
↕

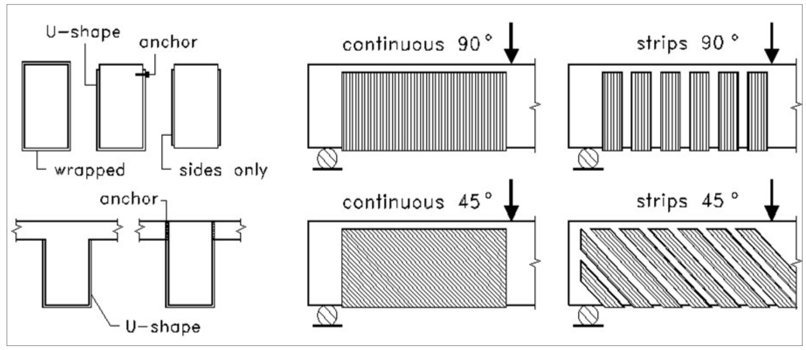
less efficient

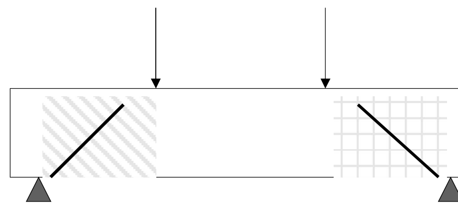


66/26

Configuration & efficiency







Modelling approach

- Superposition principle?

[redacted]

[redacted]

- ULS:

[redacted]

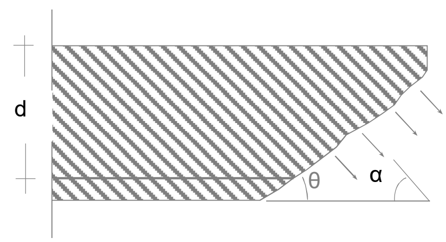
new Eurocode 2 approach

Truss analogy (similar to steel stirrups)

$$V_{ws} = \frac{A_{ws}}{s_s} 0.9 d f_{wy} \left(\cot\theta + \cot\alpha_s\right) \sin\alpha_s$$

$$V_{wf} = \frac{A_{wf}}{s_f} 0.9 d E_f \varepsilon_{f,eff} \left(\cot\theta + \cot\alpha_f\right) \sin\alpha_f$$

$f_{fw} = E_f \, \varepsilon_{fue}$



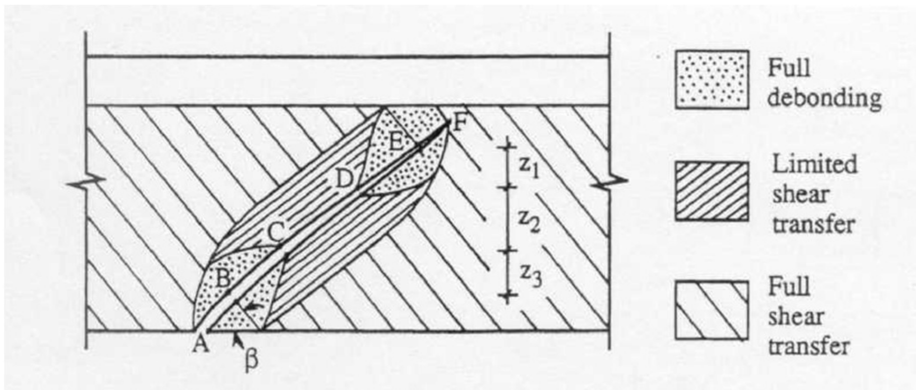
69/26

Effective ultimate FRP strain

- Strain variation along the shear crack
- Local debonding at both sides of the crack
- Possible bond failure
- Influence of fibre orientation

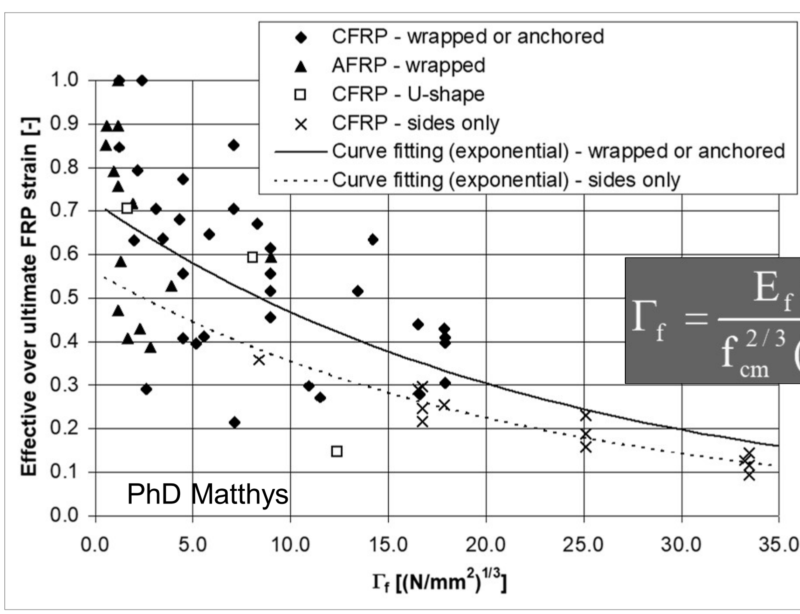
→ effective tensile strain ε_{fue}, generally lower than the ultimate FRP strain ε_{fu}

70/26



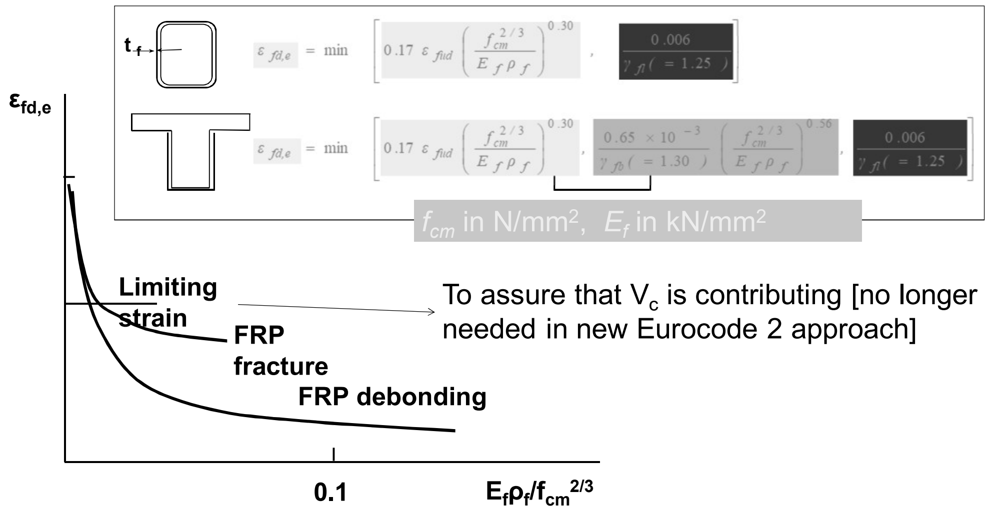
$\varepsilon_{fue} < \varepsilon_{fu}$

$\varepsilon_{fue} \rightarrow$ deterministic approach



$$\Gamma_f = \frac{E_f \rho_{wf}}{f_{cm}^{2/3}(a/d)}$$

$\varepsilon_{fue} \rightarrow$ deterministic approach (fib Bulletin 14)



$$\varepsilon_{fd,e} = \min\left[0.17\,\varepsilon_{fud}\left(\frac{f_{cm}^{2/3}}{E_f\rho_f}\right)^{0.30},\ \frac{0.006}{\gamma_{fl}(=1.25)}\right]$$

$$\varepsilon_{fd,e} = \min\left[0.17\,\varepsilon_{fud}\left(\frac{f_{cm}^{2/3}}{E_f\rho_f}\right)^{0.30},\ \frac{0.65\times10^{-3}}{\gamma_{fl}(=1.30)}\left(\frac{f_{cm}^{2/3}}{E_f\rho_f}\right)^{0.56},\ \frac{0.006}{\gamma_{fl}(=1.25)}\right]$$

To assure that V_c is contributing [no longer needed in new Eurocode 2 approach]

Alternative effective FRP strain predictions: eg. see CNR DT 200 (Italy), ACI 440.2R (USA)

$\varepsilon_{fue} \rightarrow$ semi-deterministic approach (new fib)

Closed FRP

$$\boxed{f_{fwd} = f_{fwd,c} = k_R a_t f_{fd}} \qquad (6\text{-}66)$$

where the long-term loading factor $a_t = 0.8$ and k_R = reduction factor, which accounts for the non-uniform distribution of stress in the FRP intersecting the shear crack and for the reduction of FRP strength due to bending of the fibres at the corners of the cross section. Assuming that the radius at the corners is R (mm), k_R is obtained as follows:

$$k_R = \begin{cases} 0.5\frac{R}{60}\left(2-\frac{R}{60}\right) & R < 60\text{ mm} \\ 0.5 & R \geq 60\text{ mm} \end{cases} \qquad (6\text{-}67)$$

$\varepsilon_{fue} \rightarrow$ semi-deterministic approach (new fib)

Three-sided FRP

$$\boxed{f_{fwd} = \min(f_{fbwd}, f_{fwd,c})} \qquad (6\text{-}68)$$

(a) for $h_f/\sin\alpha \geq l_e$ and $l_e \leq s_f/(\cot\theta + \cot\alpha)\sin\alpha \leq h_f/\sin\alpha$, i.e. if all strips intersected by the shear crack have bond length $\geq l_e$:

$$f_{fbwd} = \frac{f_{fbk}}{\gamma_{fb}} \qquad (6\text{-}69)$$

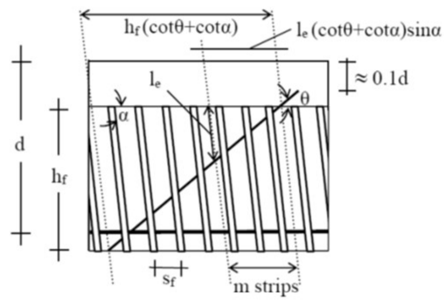
(b) for $h_f/\sin\alpha \geq l_e$ and $s_f/(\cot\theta + \cot\alpha)\sin\alpha \leq l_e$, i.e. if some of the strips intersected by the shear crack have bond length $\geq l_e$ and some others have bond length $< l_e$:

$$f_{fbwd} = \left[1 - \left(1 - \frac{2}{3}\frac{ms_f}{l_e}\right)\frac{m}{n}\right]\frac{f_{fbk}}{\gamma_{fb}} \qquad (6\text{-}70)$$

(c) for $h_f/\sin\alpha \leq l_e$ and $s_f/(\cot\theta + \cot\alpha)\sin\alpha \leq h_f/\sin\alpha$, i.e. if all strips intersected by the shear crack have bond length $< l_e$:

$$f_{fbwd} = \frac{2}{3}\frac{(ns_f)/[(\cot\theta+\cot\alpha)\sin\alpha]}{l_e}\frac{f_{fbk}}{\gamma_{fb}} \qquad (6\text{-}71)$$

where n = number of strips crossed by the shear crack = integer quotient $h_f(\cot\theta + \cot\alpha)/s_f$, m = number of strips for which the bond length is less than l_e = integer quotient $l_e(\cot\theta + \cot\alpha)\sin\alpha/s_f$, l_e= maximum bond length [Eq. (6-16)] and f_{fbk}= characteristic maximum bond strength [Eq. (6-15), bottom].



$\varepsilon_{fue} \rightarrow$ semi-deterministic approach (new fib)

Three-sided FRP

The case of **full area bond** (continuous FRP sheets) may be treated as a special case of FRP strips with $s_f = b_f/\sin\alpha$.

If $h_f/\sin\alpha \geq l_e$, Eq. (6-70) applies with $ms_f = l_e$ and $m/n = l_e/(h_f/\sin\alpha)$; and if $h_f/\sin\alpha \leq l_{b,max}$, Eq. (6-71) applies with $ns_f = h_f(\cot\theta + \cot\alpha)$. Hence,

for $h_f/\sin\alpha \geq l_e$:

$$f_{fbwd} = \left[1 - \frac{1}{3}\frac{l_e}{(h_f/\sin\alpha)}\right]\frac{f_{fbk}}{\gamma_{fb}} \qquad (6\text{-}72)$$

for $h_f/\sin\alpha \leq l_e$:

$$f_{fbwd} = \frac{2}{3}\frac{(h_f/\sin\alpha)}{l_e}\frac{f_{fbk}}{\gamma_{fb}} \qquad (6\text{-}73)$$

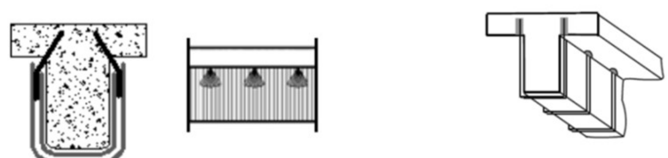
ε_{fue} → semi-deterministic approach (new fib)

Three-sided FRP with anchorage in the compression zone

If strengthening is done using three-sided FRP, the bond strength f_{fbwd} is typically lower than the strength of a closed FRP system, $f_{fwd,c}$, hence the effectiveness of FRP is low. Theoretically, f_{fbwd} can be increased up to a maximum value equal to $f_{fwd,c}$ through the use of anchors, e.g. Fig. 6-20 (Koutas and Triantafillou 2013). Assuming that the anchorage system has an effectiveness coefficient equal to k_a, to be determined through the testing approval of the system, the strength of the FRP in a three-sided system with anchors is:

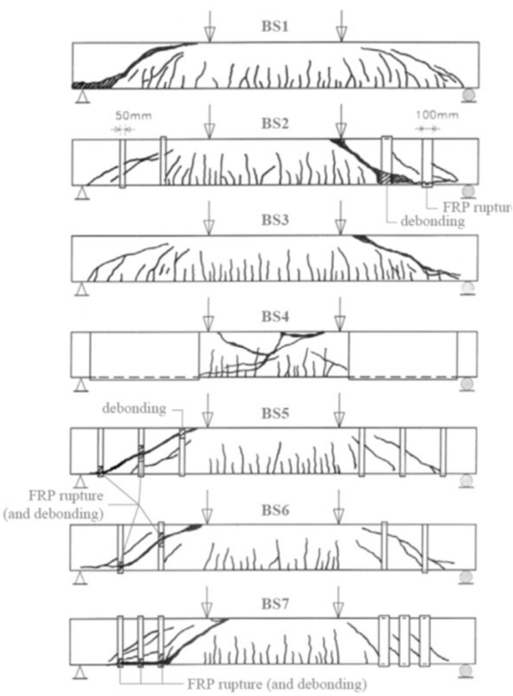
$$\boxed{f_{fwd} = k_a f_{fwd,c}} \qquad (6\text{-}74)$$

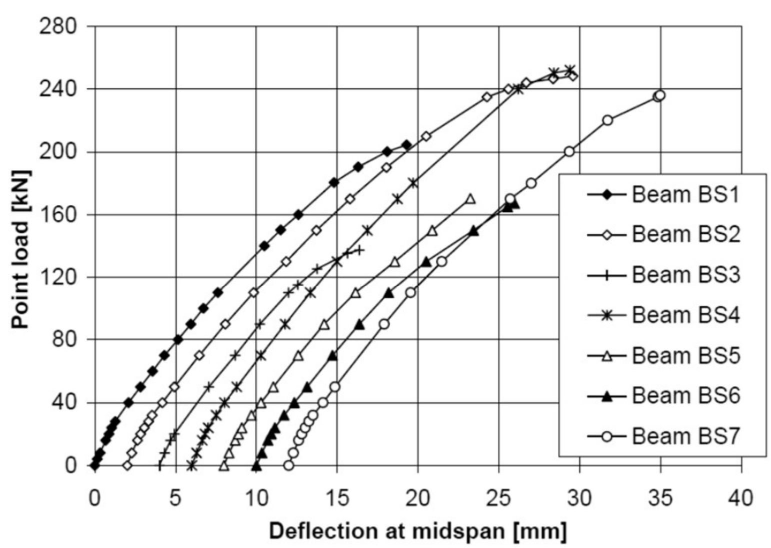
with $k_a \leq 0.9$. Note that the case when no anchors are used corresponds to $k_a = f_{fbwd}/f_{fwd,c}$.



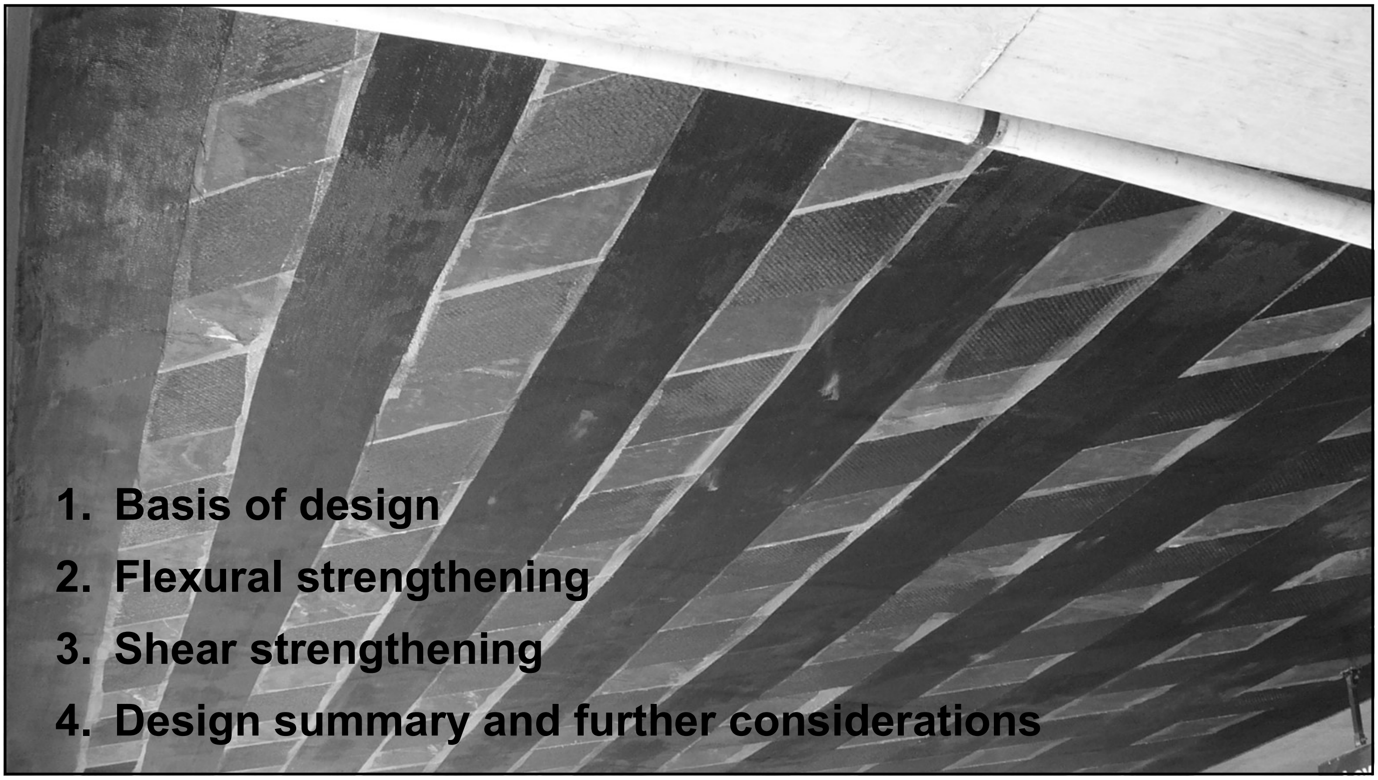
(a) (b)

77/26

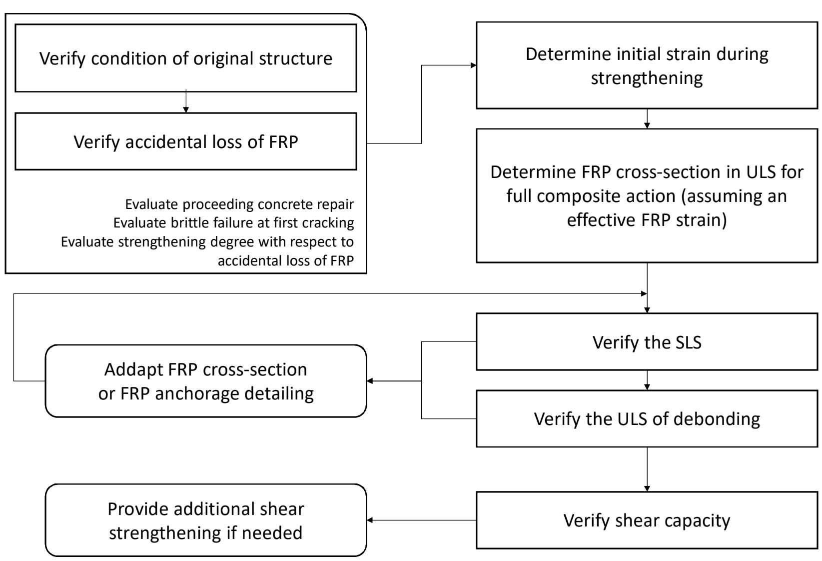




78/26

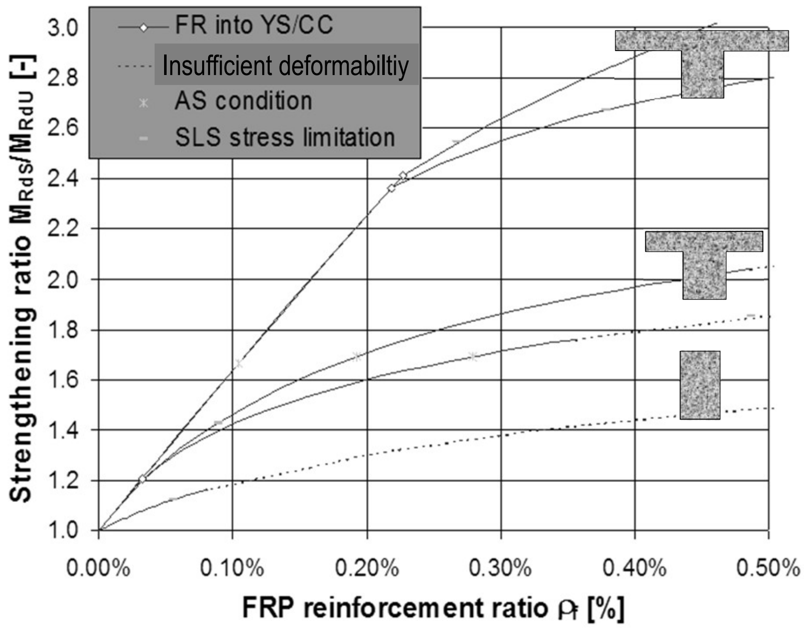


Summary: design flow chart



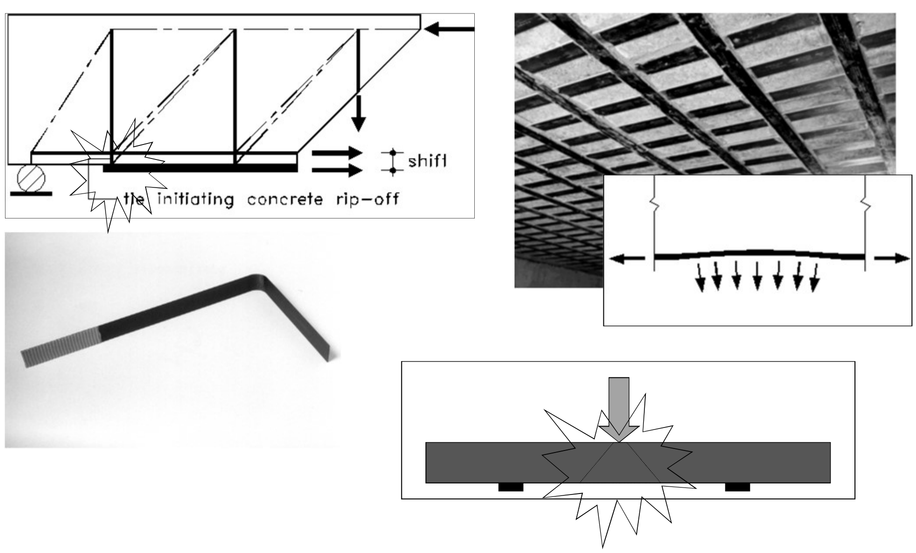
80/26

Some design observations



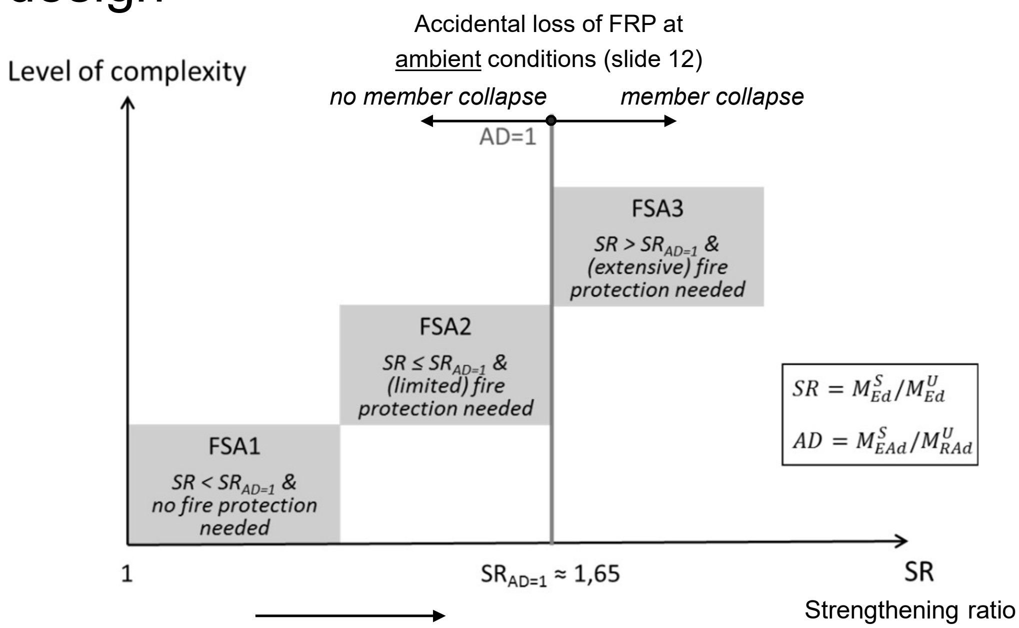
81/26

Proper detailing!



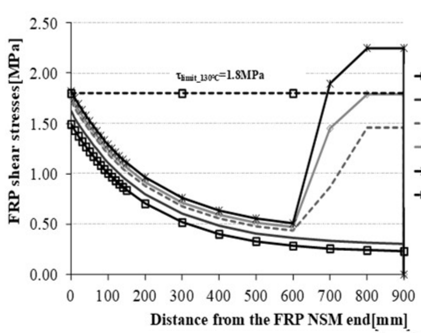
82/26

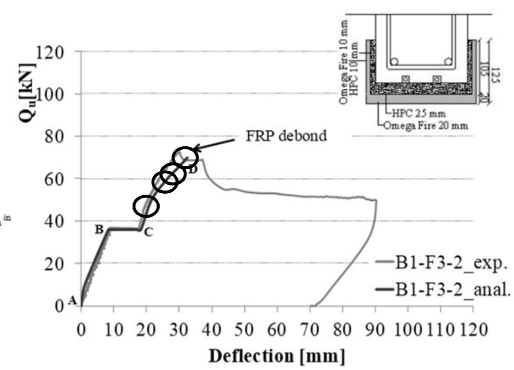
Fire design

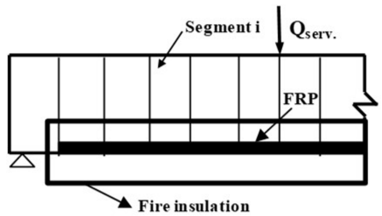


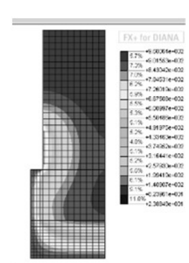
SR ↗ → Acting design load during fire ↗

Structural analysis @ fire exposure










FRPs ARE AN EFFICIENT
CONSTRUCTION MATERIAL FOR
STRENGTHENING OF STRUCTURES

Session 3

Confinement and seismic retrofitting

Thanasis Triantafillou

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REINFORCING & STRENGTHENING OF STRUCTURES WITH ADVANCED COMPOSITES

Confinement & Seismic Retrofitting

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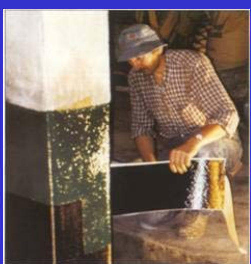
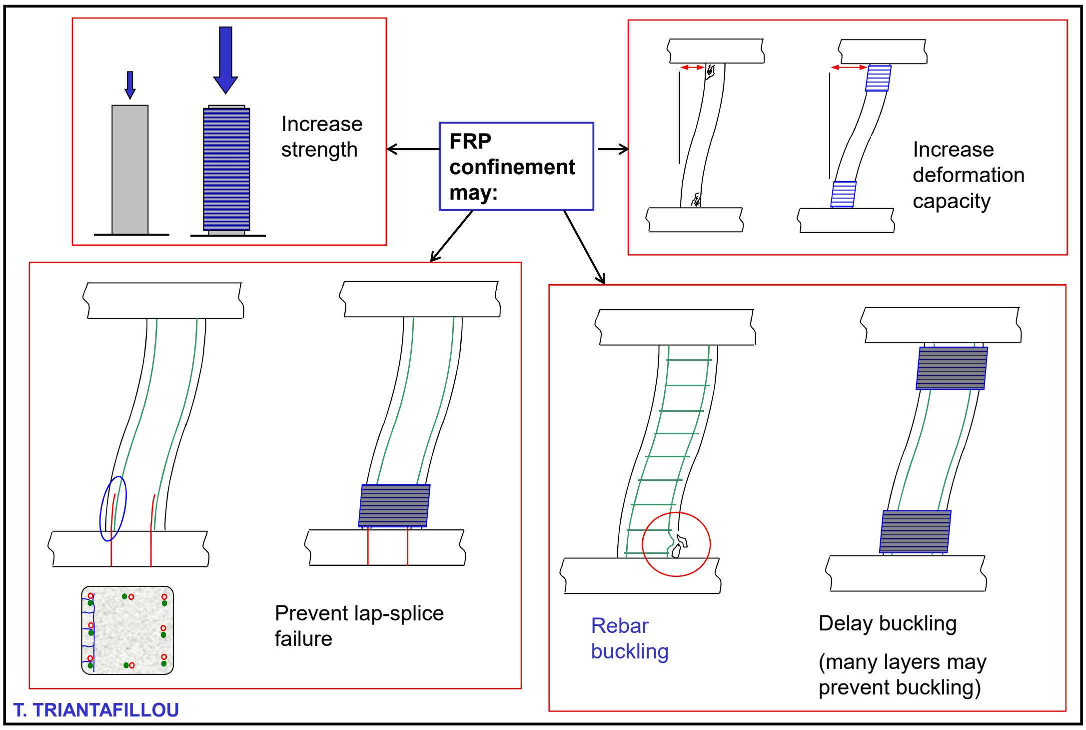
ΠΑΝΕΠΙΣΤΗΜΙΟ ΠΑΤΡΩΝ
UNIVERSITY OF PATRAS

CONFINEMENT WITH FRP



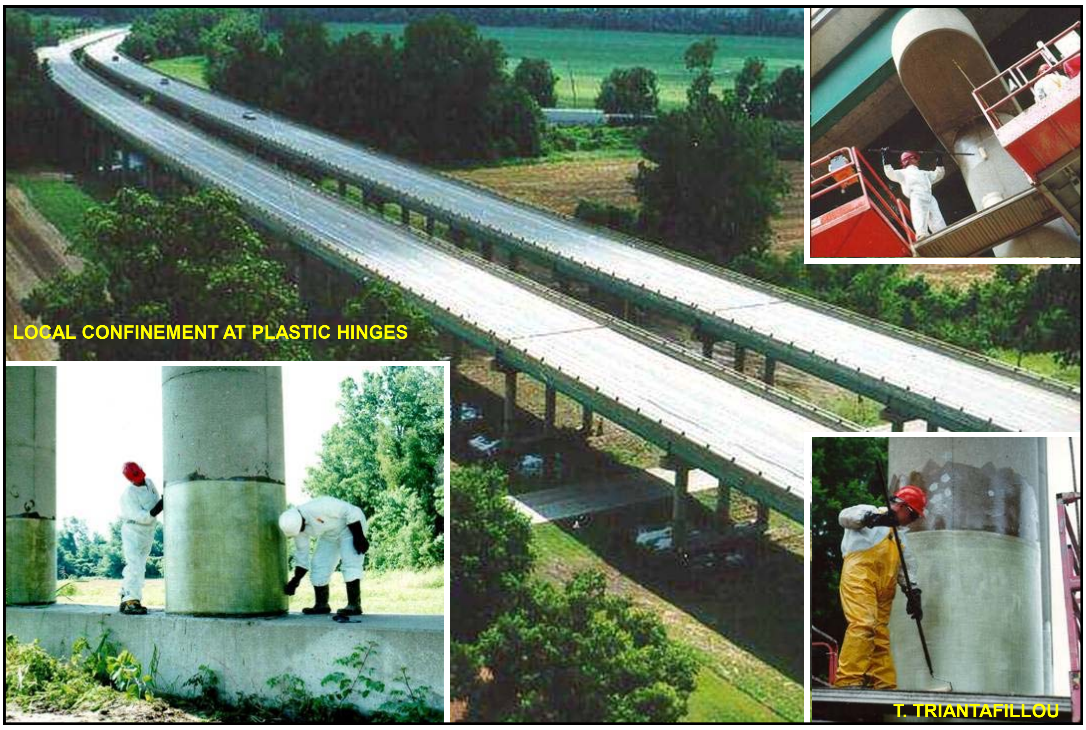
T. TRIANTAFILLOU



T. TRIANTAFILLOU


T. TRIANTAFILLOU


It's easy !
T. TRIANTAFILLOU


LOCAL CONFINEMENT AT PLASTIC HINGES
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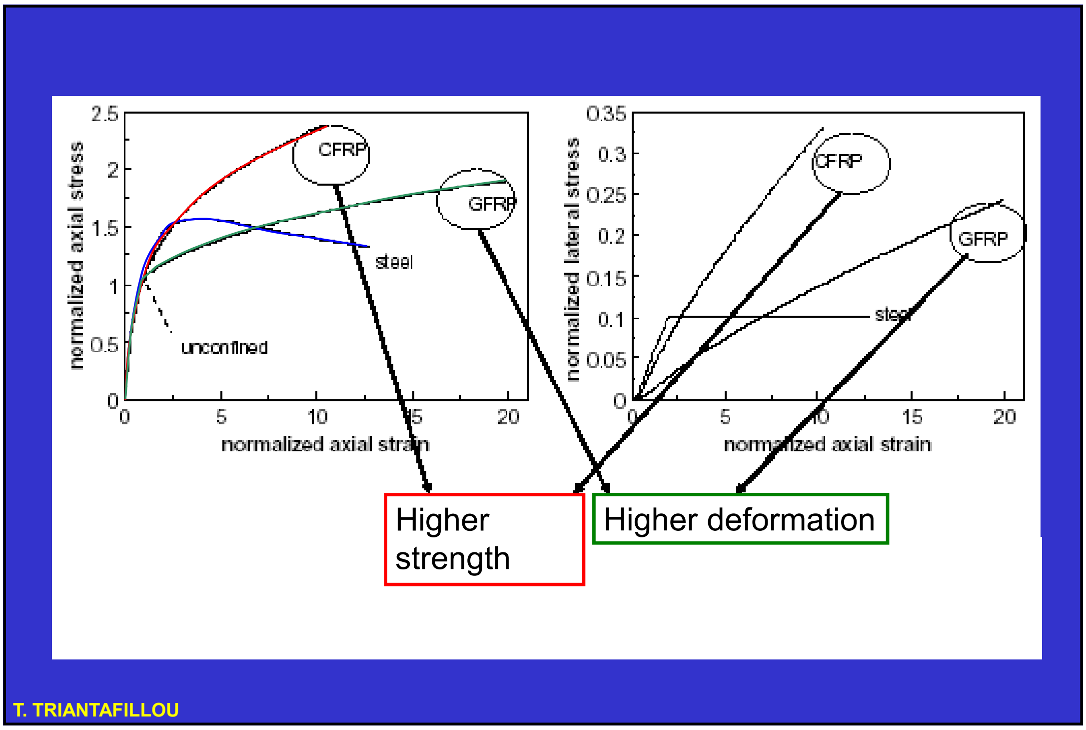
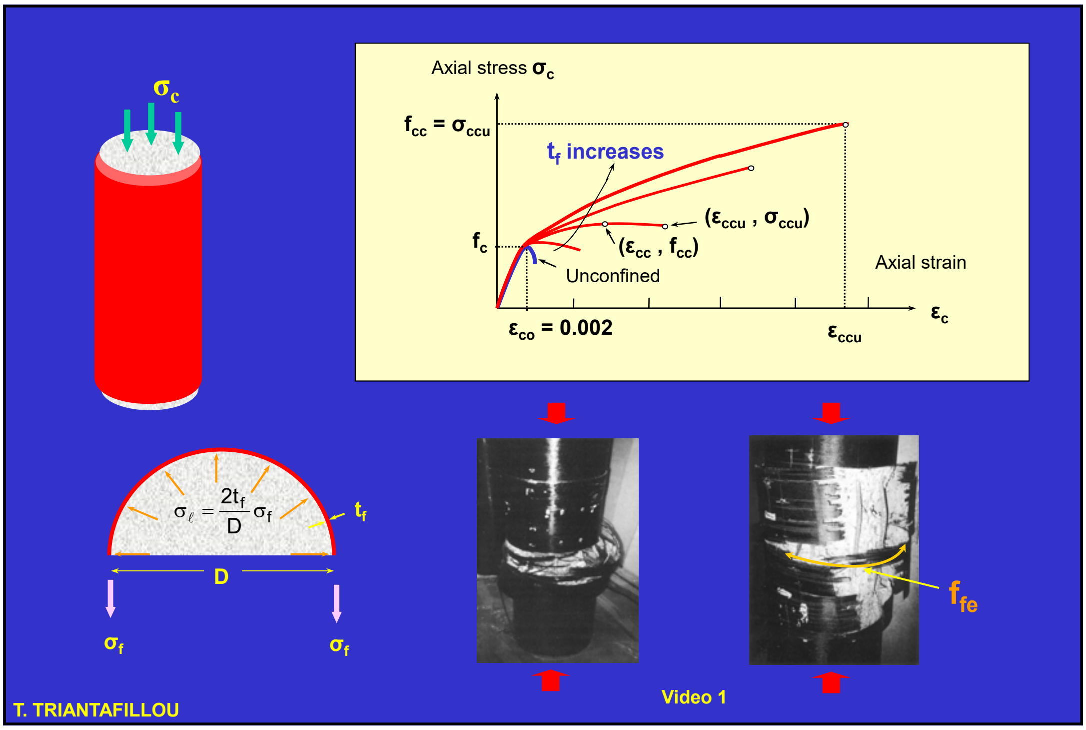
Automated wrapping
T. TRIANTAFILLOU


Impact
(aramid fibers)
T. TRIANTAFILLOU


2 m
Confinement of leaking water-pipe
T. TRIANTAFILLOU



normalized axial stress
normalized axial strain
CFRP
GFRP
steel
unconfined
normalized lateral stress
normalized axial strain
CFRP
GFRP
steel
Higher strength
Higher deformation
T. TRIANTAFILLOU



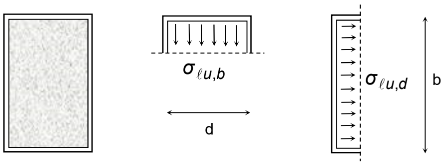
σc
Axial stress σc
fcc = σccu
tf increases
fc
(εccu , σccu)
(εcc , fcc)
Unconfined
Axial strain
εc
εco = 0.002
εccu
σℓ = 2tf/D σf
tf
D
σf
σf
ffe
Video 1
T. TRIANTAFILLOU

Confinement models

$$\frac{f_{cc}}{f_c} = 1 + k_1\left(\frac{\sigma_{\ell u}}{f_c}\right)^m$$

$$\frac{\varepsilon_{ccu}}{\varepsilon_{cu}} = 1 + k_2\left(\frac{\sigma_{\ell u}}{f_c}\right)^n$$

One possible simple approach …

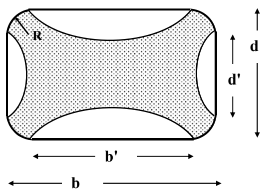


Jacket characteristics

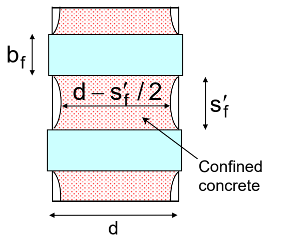
$$\sigma_{\ell u} \approx \frac{1}{2}\left(\sigma_{\ell u,b} + \sigma_{\ell u,d}\right) = \frac{1}{2}\alpha_f\left(\frac{2t_f}{d}f_{fe} + \frac{2t_f}{b}f_{fe}\right) = \alpha_f\frac{(b+d)t_f}{bd}f_{fe}$$

Effectiveness coefficient

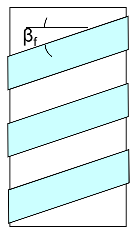
$$\alpha_f = \alpha_n \times \alpha_s \times \alpha_a \le 1$$



$$\alpha_n = \frac{A_e}{A_g} = 1 - \frac{b'^2 + d'^2}{3A_g\left(1 - \frac{A_s}{A_g}\right)}$$



$$\alpha_s = \frac{\left(1 - \frac{s_f'}{2d}\right)^2}{1 - \frac{A_s}{A_g}}$$



$$\alpha_a = \frac{1}{1 + (\tan\alpha_f)^2}$$

fib Bulletin 14 ("old" model !)

$$f_{ccd} = E_{sec,ud}\varepsilon_{ccu} \geq f_{cd}$$

$$\varepsilon_{ccu} = 0.002\left[1+5\left(\alpha_{1d}\alpha_{2d}-1\right)\right]\left[\frac{E_{sec,Md}\left(E_c - E_{sec,ud}\right)}{E_{sec,ud}\left(E_c - E_{sec,Md}\right)}\right]^{1-\frac{E_{sec,Md}}{E_c}}$$

$$E_{sec,ud} = \frac{E_c}{1+2\left(\frac{E_c}{f_{cd}} - \frac{1}{0.002}\right)\frac{f_{fde}}{E_f}} \qquad E_{sec,Md} = \frac{\alpha_{1d}\alpha_{2d}f_{cd}}{0.002\left[1+5\left(\alpha_{1d}\alpha_{2d}-1\right)\right]}$$

$$\alpha_{1d} = 2.254\sqrt{1+7.94\frac{\sigma_{\ell ud,b}}{f_{cd}}} - 2\frac{\sigma_{\ell ud,b}}{f_{cd}} - 1.254$$

$$\alpha_{2d} = 1 - \left[0.6\left(\frac{d}{b}\right)^2 - 1.4\frac{d}{b} + 0.8\right]\sqrt{\frac{\sigma_{\ell ud,b}}{f_{cd}}} \qquad \sigma_{\ell ud,b} = \boxed{\frac{t_f}{d}}\boxed{\alpha_f}\boxed{f_{fde}}$$

EXAMPLES

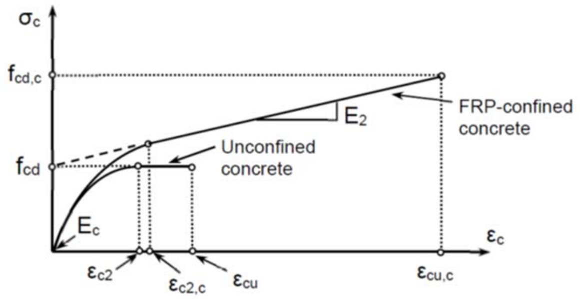
f_{cd} = 20 MPa E_c = 33.5 GPa

Target : Increase strength to 35 MPa, ultimate strain to ε_{ccu} = 0.025

Carbon fibers, E_f = 230 GPa, f_{fde} = 2460 MPa. Glass fibers, E_f = 70 GPa, f_{fde} = 1330 MPa

Section	R (mm)	A_g (cm^2)	α_f (effectiveness)	Required jacket thickness t_f (mm)			
				Carbon fibers		Glass fibers	
				for f_{ccd} = 35 MPa	*for ε_{ccu} = 0.025*	for f_{ccd} = 35 MPa	*for ε_{ccu} = 0.025*
 300 300	20	896.5	*0.50*	0.39	*0.31*	0.82	*0.12*
 250 500	20	1246.5	*0.32*	0.74	*0.56*	1.56	*0.22*
 300 300	40	886.2	*0.64*	0.31	*0.24*	0.64	*0.10*

Confinement model in the upcoming fib bulletin (modified Lam & Teng 2003)



$$\sigma_c = E_c \varepsilon_c - \frac{(E_c - E_2)^2}{4 f_{cd}} \varepsilon_c^2 \quad \text{for } 0 \le \varepsilon_c \le \varepsilon_{c2,c} \qquad \varepsilon_{c2,c} = \frac{2 f_{cd}}{(E_c - E_2)}$$

$$\sigma_c = f_{cd} + E_2 \varepsilon_c \quad \text{for } \varepsilon_{c2,c} \le \varepsilon_c \le \varepsilon_{cu,c} \qquad E_2 = \frac{f_{cd,c} - f_{cd}}{\varepsilon_{cu,c}}$$

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$$\frac{f_{cd,c}}{f_{cd}} = 1 + 3.3 \left(\frac{b}{h}\right)^2 \alpha_f \frac{2 t_f}{D^*} \frac{f_{fd,h}}{f_{cd}} \quad \text{for } \left(\frac{b}{h}\right)^2 \alpha_f \frac{2 t_f}{D^*} \frac{f_{fd,h}}{f_{cd}} \ge 0.07$$

$$\frac{f_{cd,c}}{f_{cd}} = 1 \quad \text{for } \left(\frac{b}{h}\right)^2 \alpha_f \frac{2 t_f}{D^*} \frac{f_{fd,h}}{f_{cd}} < 0.07$$

$$\frac{\varepsilon_{cu,c}}{\varepsilon_{c2}} = 1.75 + 12 \sqrt{\frac{h}{b}} \alpha_f \frac{2 t_f}{D^*} \frac{f_{fd,h}}{f_{cd}} \left(\frac{\varepsilon_{fu,h}}{\varepsilon_{c2}}\right)^{0.45}$$

$t_f = n t_o$ for $n = 1, 2$ or 3, and as $t_f = n^{0.85} t_o$ for $n \ge 4$ $\qquad D^* = \frac{2bh}{b+h}$

$$f_{fd,h} = E_f \varepsilon_{fu,h}$$

$$\varepsilon_{fu,h} = \eta_h \varepsilon_{fu} = \eta_h \frac{f_{fd}}{E_f}$$

$$\eta_h = \begin{cases} 0.5 \frac{R}{50}\left(2 - \frac{R}{50}\right) & R < 50 \text{ mm} \\ 0.5 & R \ge 50 \text{ mm} \end{cases}$$

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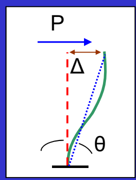
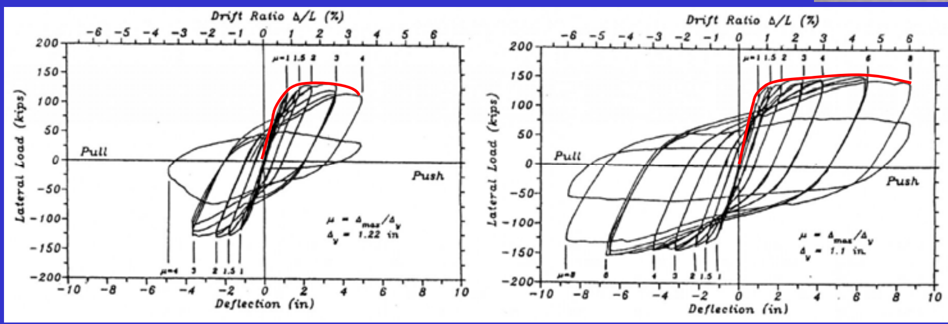
INCREASE OF DEFORMATION CAPACITY



- CHORD ROTATION

- CHORD ROTATION (OR DISPLACEMENT) DUCTILITY FACTOR
- CURVATURE DUCTILITY FACTOR



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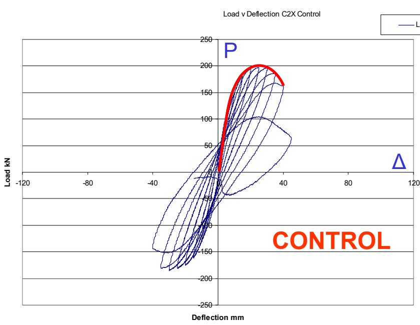

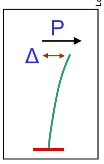

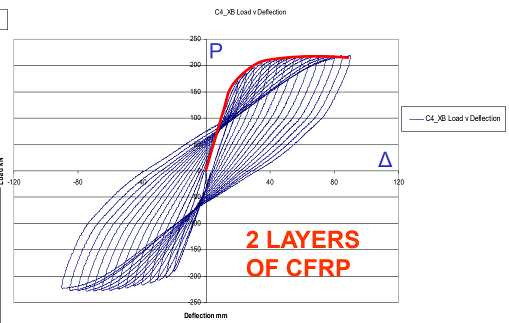

Video 2

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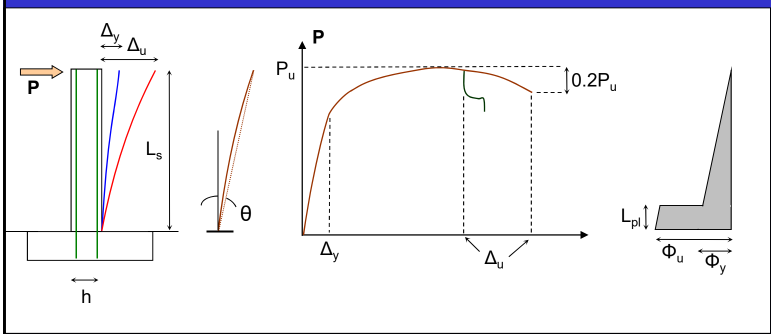
DESIGN OF JACKET FOR A TARGET CHORD ROTATION – Basic mechanics approach



Chord rotation at ultimate θ_u

$$\theta_u \approx \theta_y + (\phi_u - \phi_y)L_{pl}\left(1 - 0.5\frac{L_{pl}}{L_s}\right)$$

$$L_{pl} = 0.1L_s + 0.17h + \frac{0.24 f_y}{\sqrt{f_c}} d_b$$

$$\phi_u = \varepsilon_{ccu} / x_u$$

Expressed in terms of jacket thickness through the confinement model

$$\theta_y \approx \phi_y \frac{L_s}{3} + 0.0013\left(1 + 1.5\frac{h}{L_s}\right) + 0.13\phi_y \frac{f_y}{\sqrt{f_c}} d_b$$

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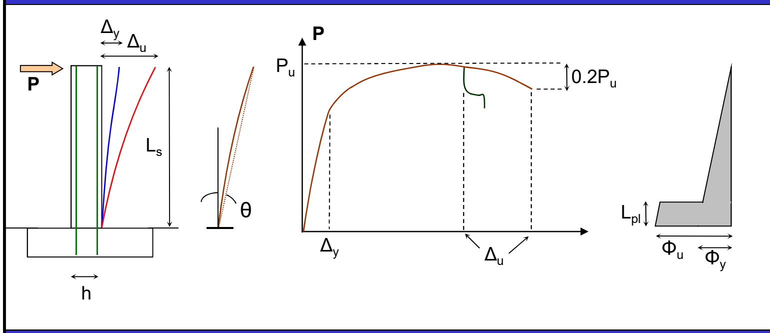
DESIGN OF JACKET FOR A TARGET CHORD ROTATION - EC8 empirical equation



Chord rotation at ultimate θ_u

$$\theta_u = 0.016\left(0.3^{v}\right)\left[\frac{\max(0.01,\omega')}{\max(0.01,\omega)} f_c\right]^{0.225}\left(\frac{L_s}{h}\right)^{0.35} 25^{\left(\alpha\rho_{sx}\frac{f_{yw}}{f_c}+\alpha_f\rho_{fx}\frac{f_{fe}}{f_c}\right)}\left(1.25^{100\rho_d}\right)$$

$$f_{fe} = \min(f_f, \varepsilon_{fu}E_f)\left[1-0.7\min(f_f, \varepsilon_{fu}E_f)\frac{\rho_{fx}}{f_c}\right]$$

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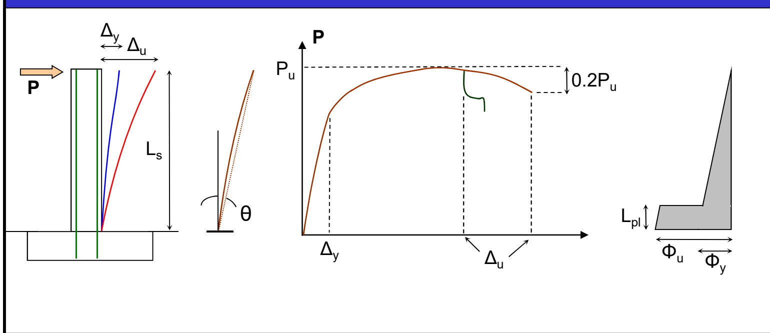
DESIGN OF JACKET FOR DUCTILITY



Chord rotation (or displacement) ductility factor

$$\mu_\Delta = \frac{\Delta_u}{\Delta_y} = \mu_\theta = \frac{\theta_u}{\theta_y}$$

Tastani & Pantazopoulou (2002)

Alternative but very conservative approach

$$\boxed{\mu_\Delta} = \mu_\theta = 1.3 + 12.4\left(\frac{\sigma_{\ell u,b}}{f_c} - 0.1\right) \geq 1.3$$

$$\sigma_{\ell u,b} = \alpha_f \frac{2\boxed{t_f}}{d} f_{fe}$$

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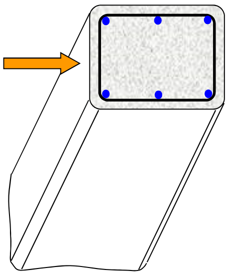
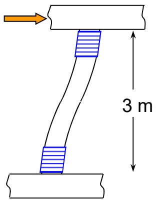
EXAMPLE

Corner radius R = 25 mm

0.30x0.40 m

f_c = 11 MPa, longitudinal reinforcment 6Φ18, f_y = 350 MPa





Carbon fibres,
E_f = 230 GPa ,
f_{fe} = 3150 MPa ,
t_{fib} = 0.12 mm

Effectiveness coefficient

$$\alpha_n = 1 - \frac{35^2 + 25^2}{3 \times 1195 \times \left(1 - \frac{15.25}{1195}\right)} = 0.48$$

Required jacket thickness t_f for μ_Δ = 4

$$4 = 1.3 + 12.4\left(\frac{0.48 \times \frac{2t_f}{300} \times 3150}{11} - 0.1\right)$$

t_f = 0.35 mm → 3 layers → μ_Δ = 4.15

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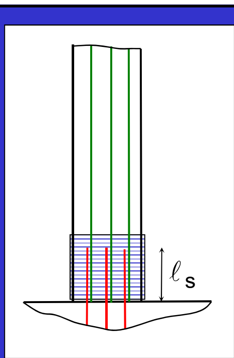
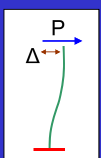
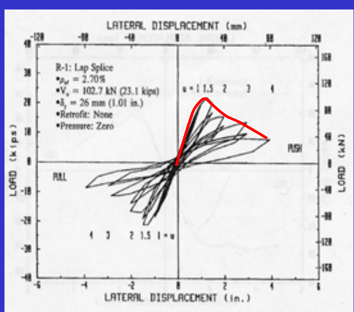
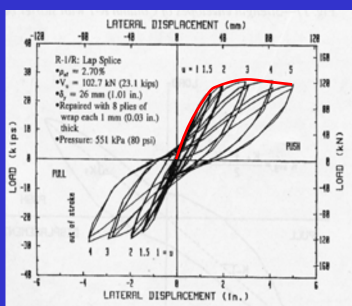
CONFINEMENT AT LAP SPLICES









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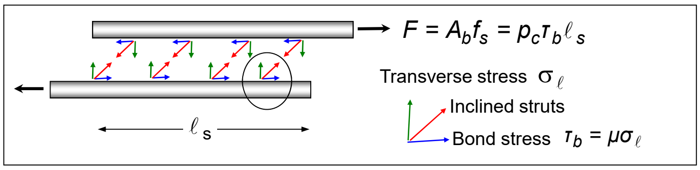
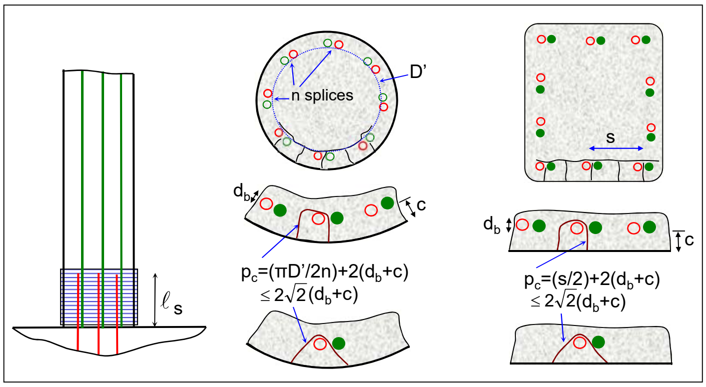
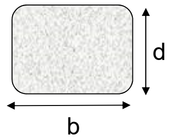
DESIGN OF JACKET TO PREVENT LAP-SPLICE FAILURE



$f_s = f_y$ if $\ell_s > \ell_{s,min}$





$$t_f = \gamma_{Rd}\frac{\left(1-\frac{\ell_s}{\ell_{s,min}}\right) b d A_b f_y}{\alpha_f (b+d) p_c \mu f_{fe} \ell_s}$$

$f_{fe} \le 0.001 \times E_f$

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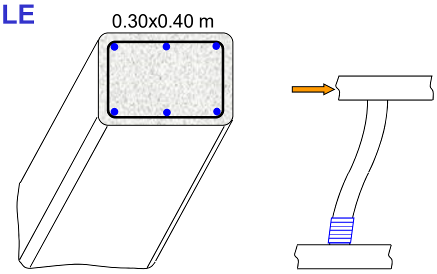
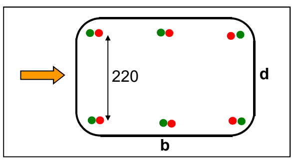
EXAMPLE



Radius at corners R = 25 mm , concrete cover c = 30 mm
f_c = 11 MPa, longitudinal steel 6Φ16, f_y = 230 MPa
Lap splice 0.25 m, $\ell_{s,min}$= 0.90 m

Carbon fibers,
E_f = 230 GPa ,
f_{fe} = 2600 MPa ,
t_{fib} = 0.12 mm

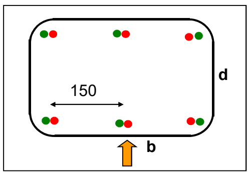


$$p_c = \min\left(\left[(220/2)+2(18+30)\right], 2\sqrt{2}(18+30)\right) = 136 \text{ mm}$$

$$A_b = \left(\pi \times 16^2\right)/4 = 200 \text{ mm}^2$$

$$f_{fe} = \min(2600, 0.001\times 230000) = \min(2600, 240) = 240 \text{ MPa}$$

$$t_f = \frac{(\gamma_{Rd}=1.5)\times\left(1-\frac{0.25}{0.35}\right)\times 300\times 400\times 200\times 230}{0.48\times(300+400)\times 136\times 1.4\times 240\times 500} = 0.31 \text{ mm} \rightarrow 3 \text{ layers}$$



$$p_c = \min\left(\left[(150/2)+2(18+30)\right], 2\sqrt{2}(18+30)\right) = 136 \text{ mm}$$

$$t_f = 0.33 \text{ mm} \rightarrow 3 \text{ layers}$$

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DELAY OF REBAR BUCKLING

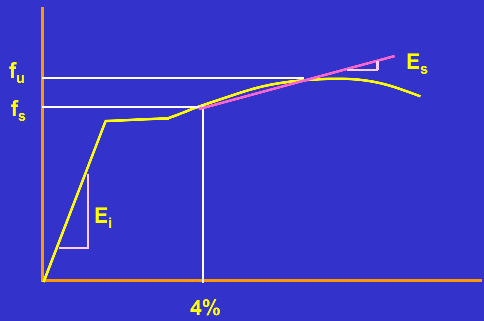
Video 3

$$t_f = \gamma_{Rd} \frac{0.45 n f_s^2 d}{4 E_{ds} E_f \alpha_f} \approx \gamma_{Rd} \frac{10 n d}{E_f \alpha_f}$$

n = Total number of longitudinal rebars

d = Depth of cross section parallel to the plane of bending

$$E_{ds} = \frac{4 E_s E_i}{\left(\sqrt{E_s} + \sqrt{E_i}\right)^2}$$ "Double" modulus



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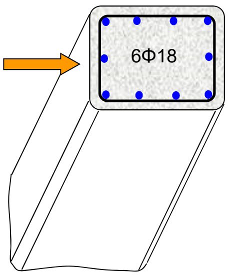
EXAMPLE

0.30x0.40 m

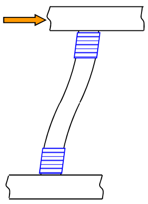


Carbon fibers,
E_f = 220 GPa ,
t_{fib} = 0.12 mm

Required jacket thickness

$$t_f = \frac{(\gamma_{Rd} = 1.5) \times 10 \times 10 \times 400}{230000 \times 0.48} = 0.54 \text{ mm}$$

5 layers



Glass fibres,
E_f = 70 GPa ,
t_{fib} = 0.17 mm

Required jacket thickness

$$t_f = \frac{(\gamma_{Rd} = 1.5) \times 10 \times 10 \times 400}{70000 \times 0.48} = 1.77 \text{ mm}$$

11 layers

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SUMMARY OF DESIGN PROCEDURE FOR FRP JACKETING

GIVEN THAT "STIFFENING" IS NOT A REQUIREMENT, THE JACKET SHOULD BE DESIGNED FOR:

(a) the target **deformation capacity** (chord rotation or ductility factor),

(b) the target **shear resistance** (such that flexural failure preceedes shear failure).

The jacket thickness in step (a) should be checked (and modified, if needed) for **rebar buckling** and **lap splice failure**.

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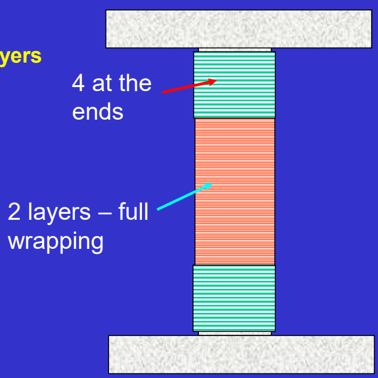
EXAMPLE OF JACKET DESIGN

	Required number of layers
Deformation capacity	4
Lap splices	2
Rebar buckling	3
Shear	2



NO IMPROVEMENTS REGARDING :

- Stiffening
- Second order (P-Δ) effects
- Flexural strengthening

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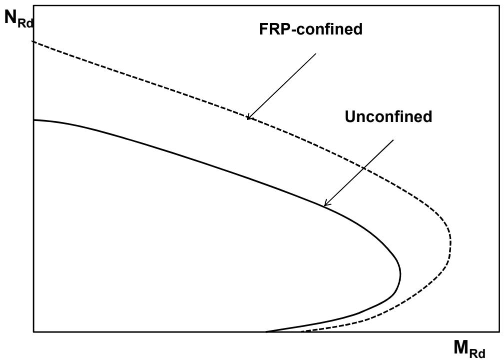
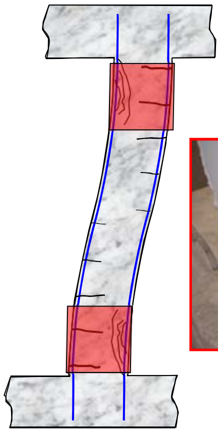
We don’t do FRP confinement to increase flexural resistance …



… but it is possible to increase flexural resistance (relocation of plastic hinges!) via longitudinal reinforcement (e.g. NSM)



Bournas and Triantafillou (2009) ACI Str. J.

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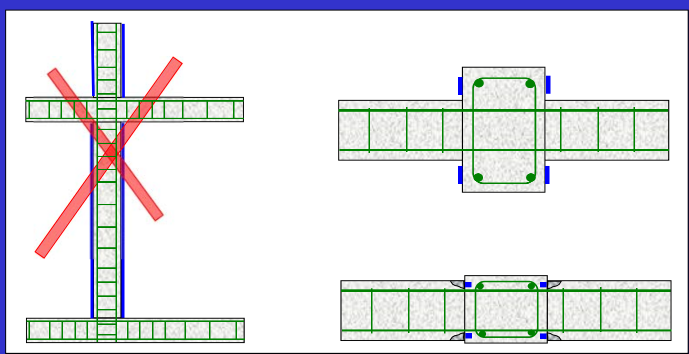
Flexural Strengthening of Columns

This is wrong !!



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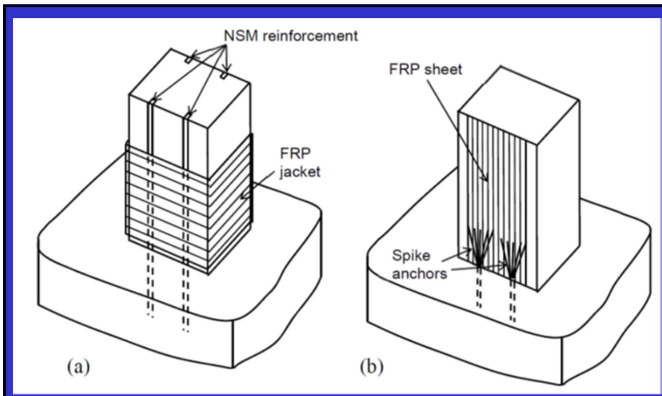
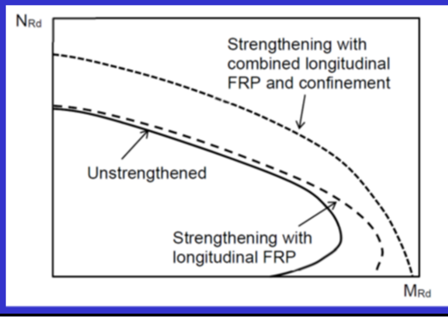


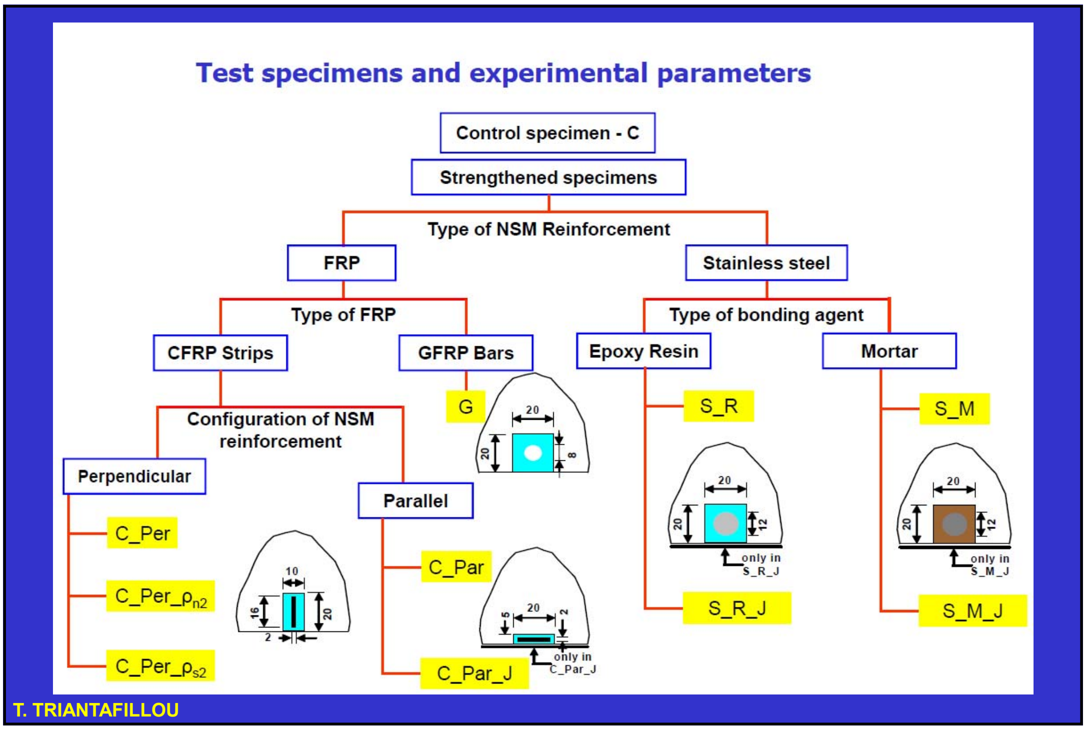


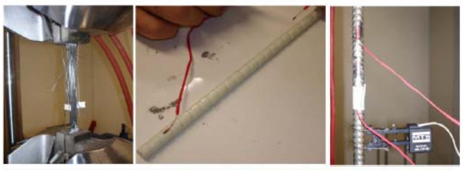
Strengthening procedures

Grooves and holes were filled by injecting the bonding agent

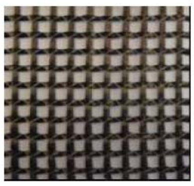
The NSM reinforcement was placed into position and the bonding material in excess was removed

4 layers of a TRM jacket (only for confined specimens)

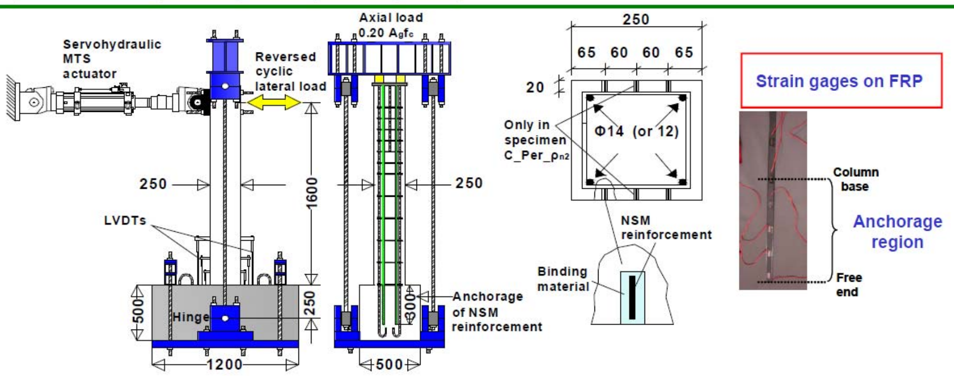




Type of NSM Reinforcement	Elastic Modulus E, (GPa)	Tensile strength, f_u (GPa)	Ultimate strain, ε_u (%)	Tensile Force (kN)
CFRP strips	144.9	2173	1.83	70
GFRP bars	65.2	1491	3.18	75
Stainless steel bars	200	761	19.15	76



ρ = 348 g/m^2
t = 0.095 mm
f_{fe}= 3800 MPa
E_f= 230 GPa



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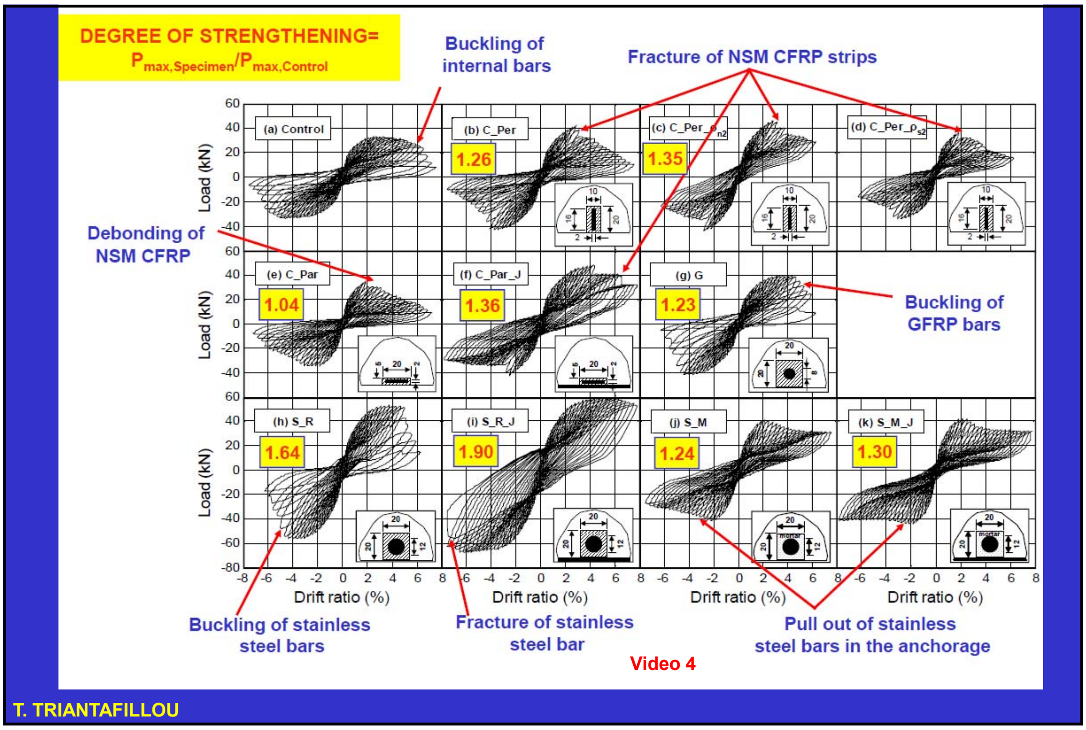
DEGREE OF STRENGTHENING=
$P_{max,Specimen}/P_{max,Control}$
Buckling of internal bars
Fracture of NSM CFRP strips
Debonding of NSM CFRP
Buckling of GFRP bars
(a) Control
(b) C_Per
1.26
(c) C_Per_ρn2
1.35
(d) C_Per_ρs2
(e) C_Par
1.04
(f) C_Par_J
1.36
(g) G
1.23
(h) S_R
1.64
(i) S_R_J
1.90
(j) S_M
1.24
(k) S_M_J
1.30
Load (kN)
Drift ratio (%)
Buckling of stainless steel bars
Fracture of stainless steel bar
Pull out of stainless steel bars in the anchorage
Video 4
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FRP sheets + spike anchors
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CASE STUDY - SEISMIC RETROFITTING OF RC BUILDING (Chania, Greece)

- 800 m^2 CFRP
- Job done in about one month
- No modification of geometry

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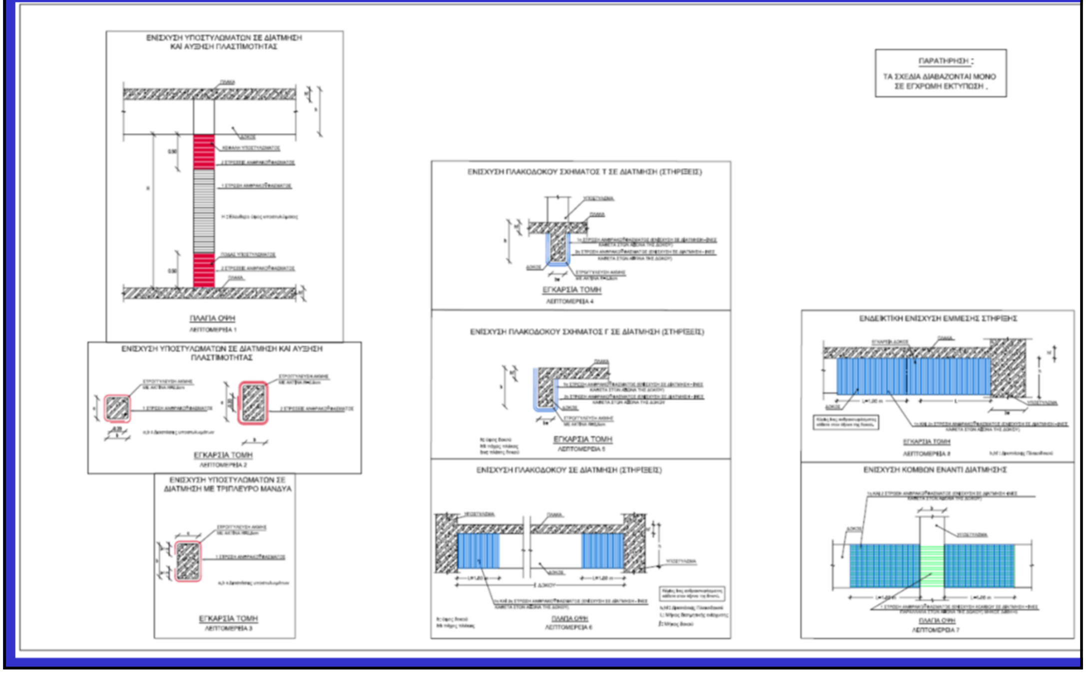
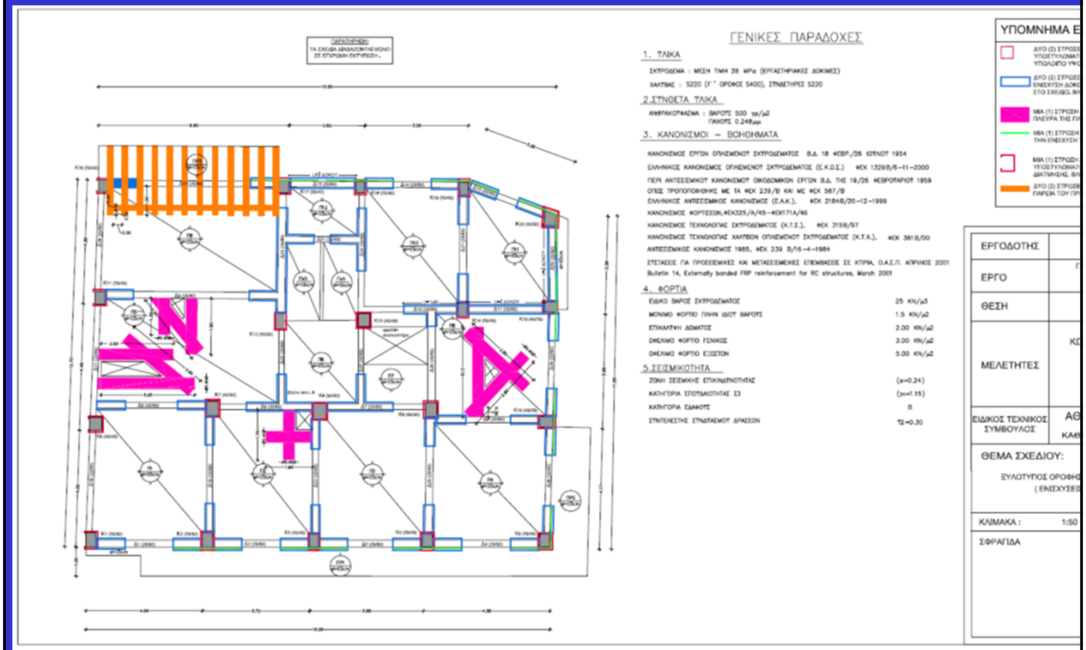
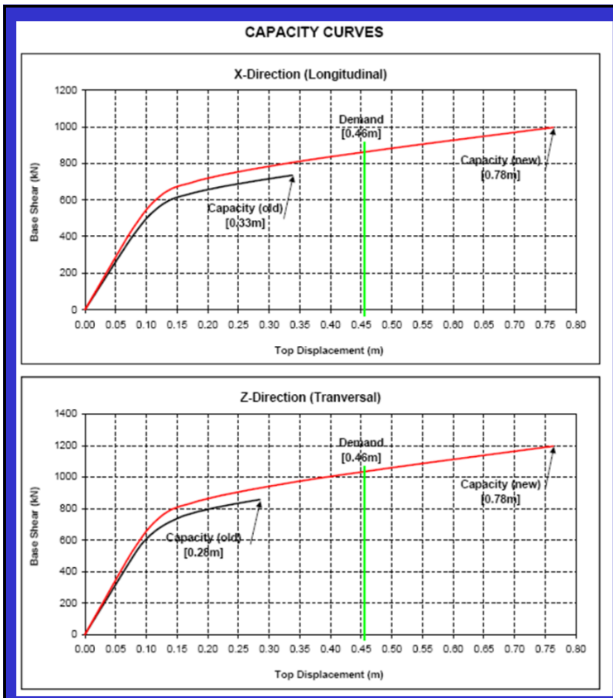
Drawings … sorry, in Greek

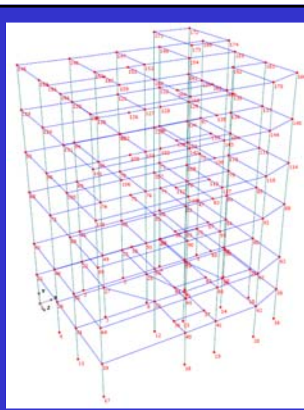


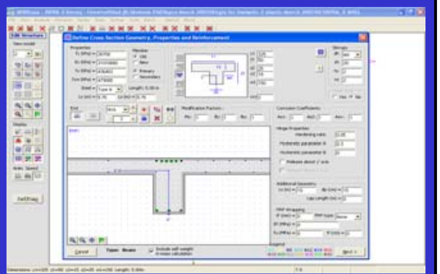
Typical floor





Nonlinear analyis using ANSRuop





T. TRIANTAFILLOU

A "FEELING" OF FRP JACKET SIZES …

Shear strengthneing: 2 layers of "standard" CFRP fabric (~0.13 mm thick) is "equivalent" to S500 Φ8/100 stirrups

Confinement for seismic retrofitting: 3 layers will provide a chord rotation ductility factor $\mu_\theta = \mu_\Delta > 4\text{-}5$ and will prevent lap splice failures in many "common" cases

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A word about … BEAM-COLUMN JOINTS

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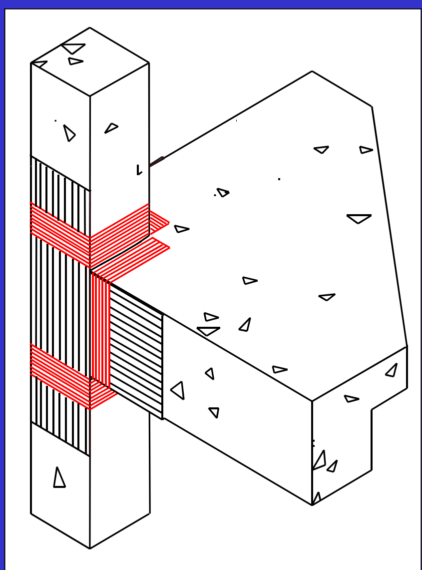

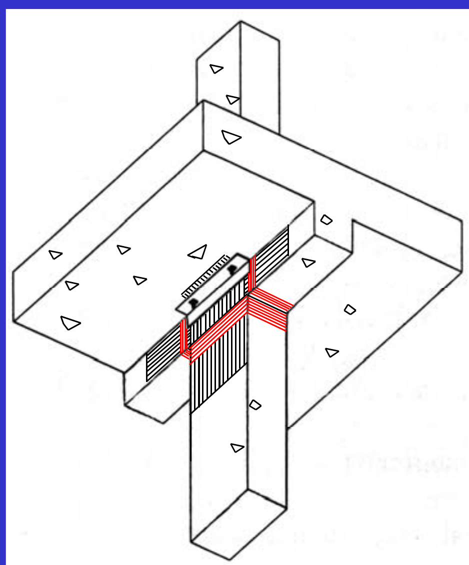

FRP thickness per direction: $t_{fh}=V_{jh}/h_{beam}f_{fe}$ $t_{fv}=V_{jv}/h_{col}f_{fe}$

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TRM (Textile Reinforced Mortar) vs. FRP: WHY ?

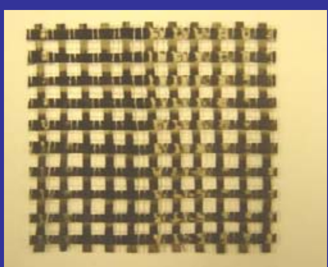
Potential problems with FRP (not so important in the case of RC, more important in masonry)

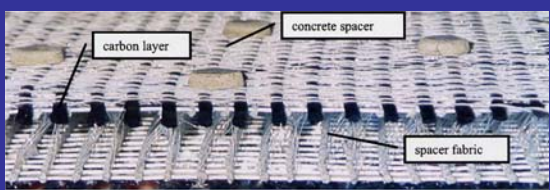
- Poor behaviour of resins above T_g
- High cost of epoxies
- Inability to apply on wet surfaces or at low temperatures
- Lack of vapour permeability
- Incompatibility with substrate materials
- Difficulty in contacting post-earthquake assessment of damage suffered by the concrete behind the FRP

Solution

Inorganic binders and fibres in the form of TEXTILES:

fiber meshes made of long woven, knitted or even unwoven tows or yarns in at least two (typically orthogonal) directions





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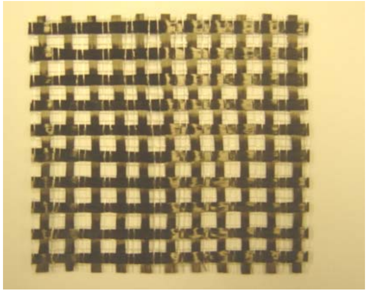
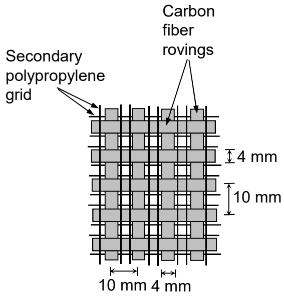
MATERIALS



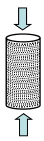


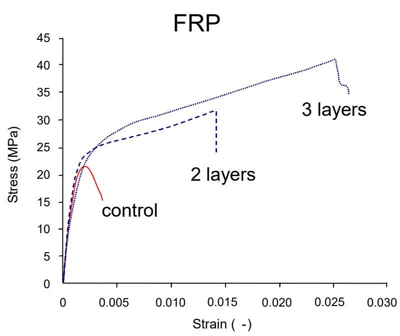
Carbon fibers , 160 g/m^2 , E = 225 GPa , f_t = 3350 GPa , tex 800

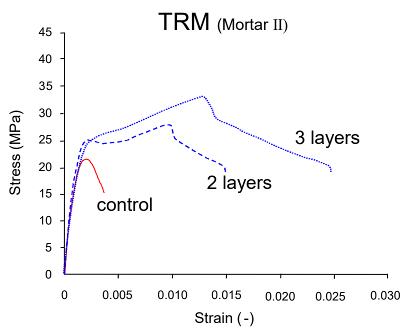
Nominal thickness (based on equivalent smeared distribution of fibers) = 0.047 mm

Mortar	Flexural Strength (MPa)	Compressive Strength (MPa)
Mortar I		
7 days	2.68	7.59
28 days	3.28	8.56
Mortar II		
7 days	3.02	27.45
28 days	4.24	30.61

Resin vs. Mortar (cylinders)







Strength increase

53% (2 layers) 92% (3 layers)

Strain increase

x8.3 (2 layers) x12.7 (3 layers)

Strength increase

25% (2 layers) 49% (3 layers)

Strain increase

x4.9 (2 layers) x5.4 (3 layers)

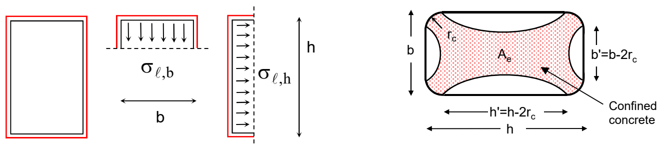
SIMPLE ANALYTICAL MODEL

$$\frac{f_{cc}}{f_{co}} = 1 + k_1\left(\frac{\sigma_{\ell u}}{f_{co}}\right)^m \qquad k_1 = \alpha_1 k_{1,R}$$

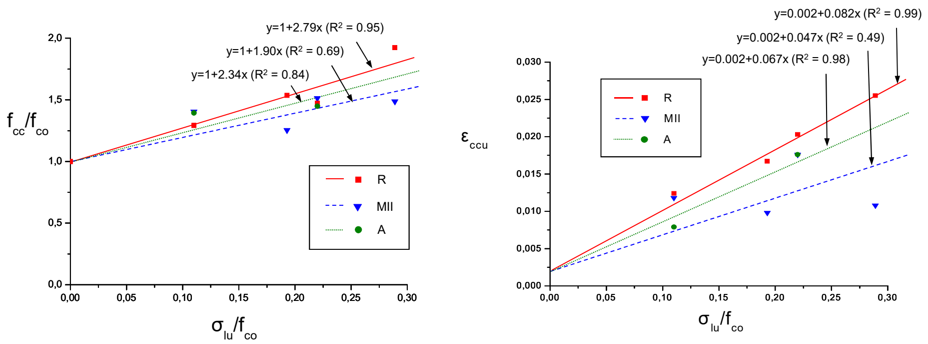
$$\varepsilon_{ccu} = \varepsilon_{co} + k_2\left(\frac{\sigma_{\ell u}}{f_{co}}\right)^n \qquad k_2 = \alpha_2 k_{2,R}$$

α_1 : effectiveness coeff. for strength

α_2 : effectiveness coeff. for strain



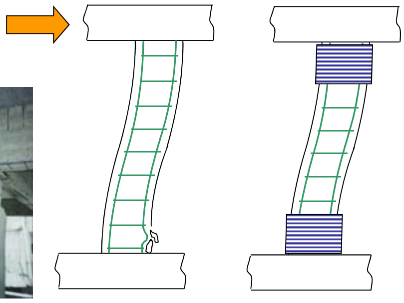
$$\sigma_\ell = \frac{\sigma_{\ell,h}+\sigma_{\ell,b}}{2} = \frac{1}{2}\alpha\left(\frac{2t_j}{h}E_j\varepsilon_j + \frac{2t_j}{b}E_j\varepsilon_j\right) = \alpha\frac{(b+h)}{bh}t_jE_j\varepsilon_j \Rightarrow \boxed{\sigma_{\ell u} = \alpha\frac{(b+h)}{bh}t_jf_j}$$



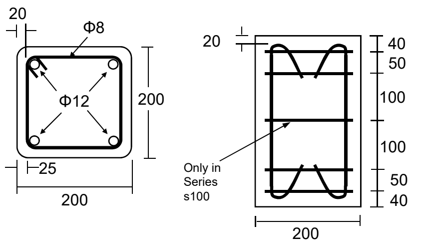
TRM vs. FRP : α_1 = 0.68 , α_2 = 0.57

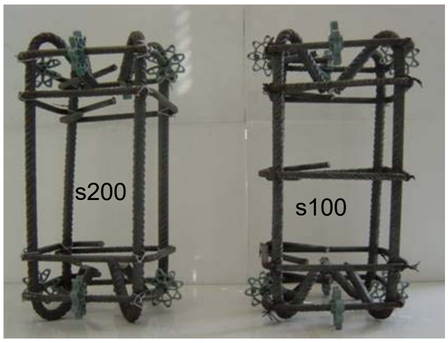
Unbonded & end-anchored vs. FRP : α_1 = 0.84, α_2 = 0.82

CONFINEMENT TO DELAY REBAR BUCKLING IN COLUMNS



Series U (no steel) , s200 , s100





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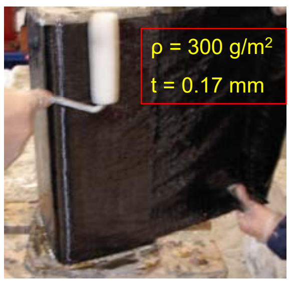
ρ = 348 g/m²

t = 0.095 mm

tex 1600

4 or 6 layers

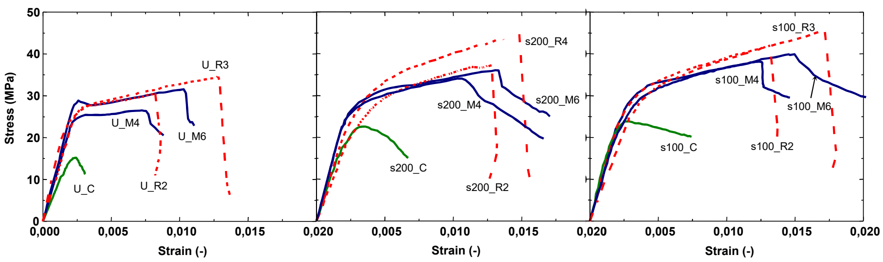


2 or 3 layers

THE TWO SYSTEMS ARE “EQUIVALENT”

T. TRIANTAFILLOU

CONTROL **FRP** **TRM**



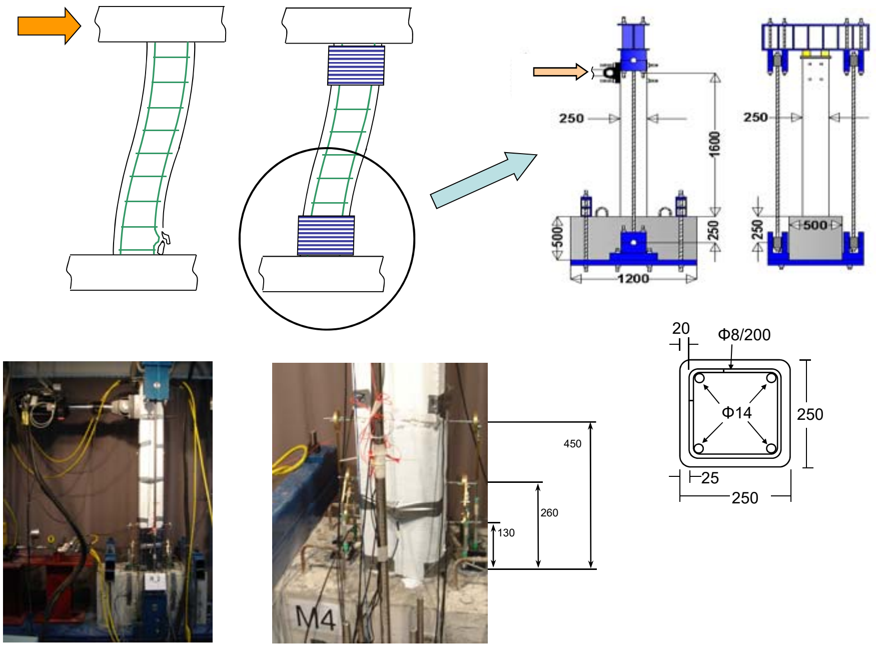
Jacket effectiveness

$$K_\sigma = \frac{\sigma_{max,c} - \frac{A_s}{A_g} f_y}{\sigma_{max,o} - \frac{A_s}{A_g} f_y} \qquad K_\varepsilon = \frac{\varepsilon_{u,c}}{\varepsilon_{u,o}}$$

TRM vs. FRP: $\frac{K_{\sigma,TRM}}{K_{\sigma,FRP}}$ $\frac{K_{\varepsilon,TRM}}{K_{\varepsilon,FRP}}$

0.85-0.90 0.87-0.93

Cyclic Testing of Poorly Detailed Columns



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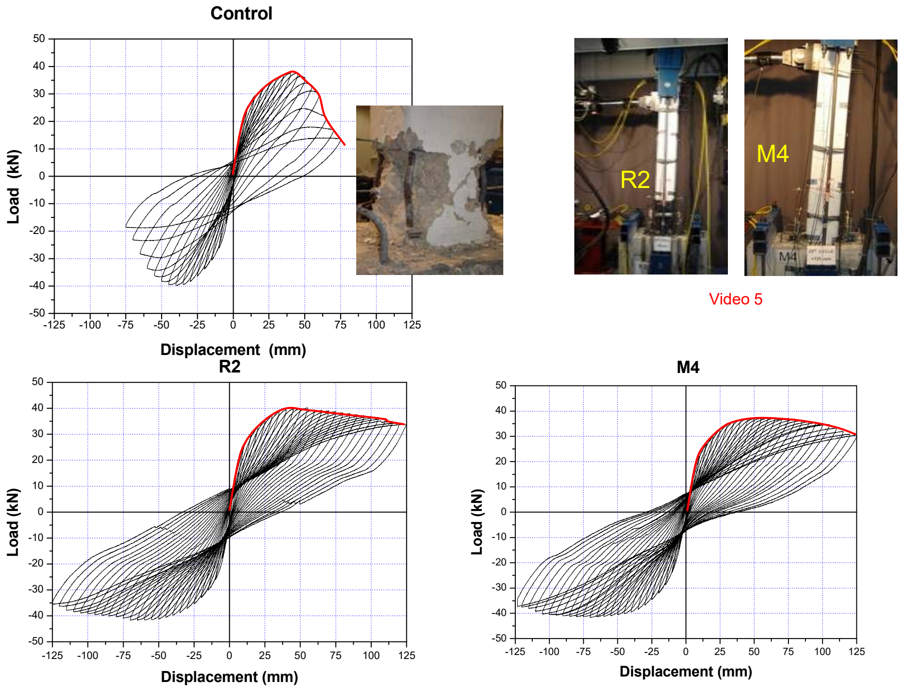

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2/3 SCALE TESTING OF 2-STOREY BUILDING

Tested at 0.3g before retrofitting Video 6

Tested at 0.45g after retrofitting

TRM jackets

TRM jackets have performed extremely well !!

T. TRIANTAFILLOU

Ultimate chord rotation (EC8)

$$\theta_u = k0.016\left(0.3^{\nu}\right)\left[\frac{max(0.01,\omega')}{max(0.01,\omega)} f_c\right]^{0.225}\left(\frac{L_V}{h}\right)^{0.35} 25^{c}\left(1.25^{100\rho_d}\right)$$

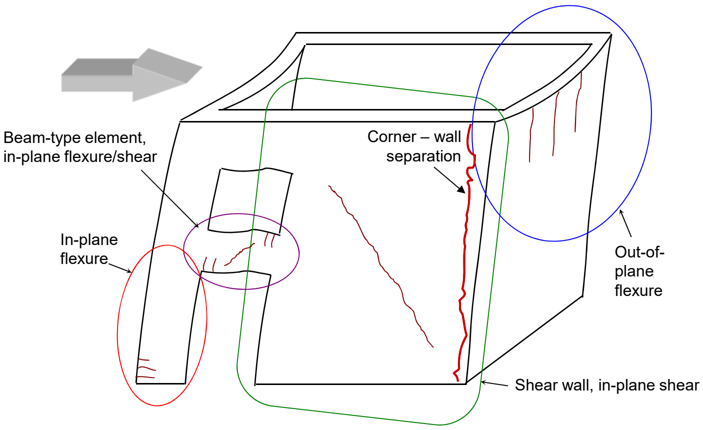
$$c = a\rho_{sx}\frac{f_{yw}}{f_c} + a_f \rho_{fx}\frac{f_{fe}}{f_c}$$

$$\alpha_f = \beta\left[1 - \frac{(b-2R)^2 + (h-2R)^2}{3bh}\right]$$

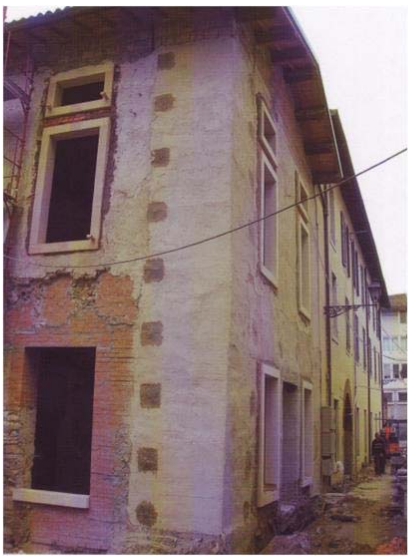
$$\beta = \frac{K_{\varepsilon,TRM}}{K_{\varepsilon,FRP}}$$

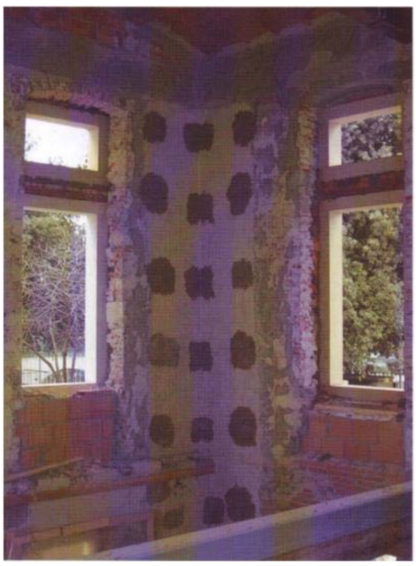
T. TRIANTAFILLOU

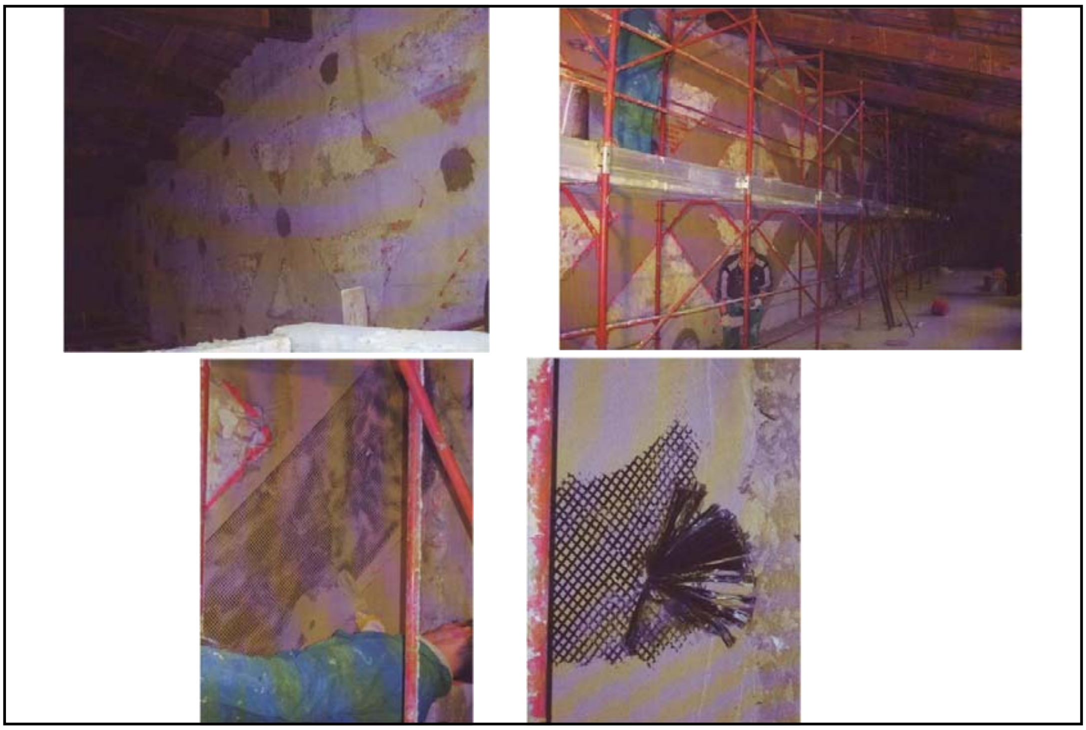
MOST PROMISING APPLICATIONS OF TRM: SEISMIC RETROFITTING OF MASONRY

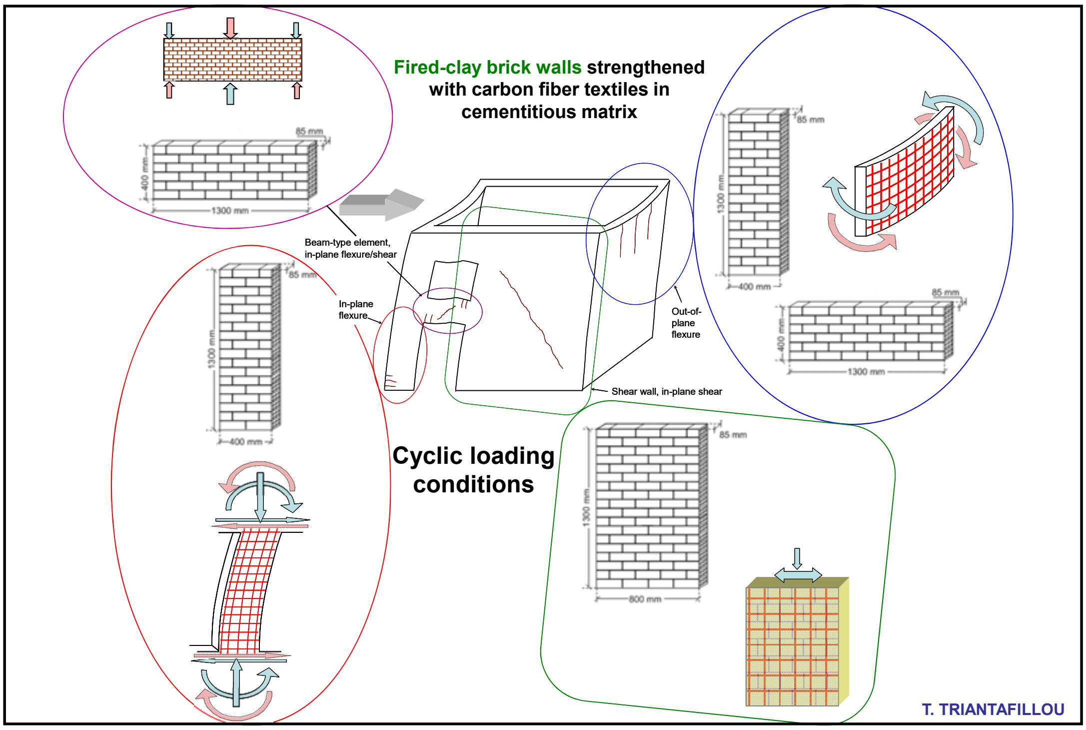


T. TRIANTAFILLOU









Fired-clay brick walls strengthened with carbon fiber textiles in cementitious matrix
85 mm
400 mm
1300 mm
Beam-type element, in-plane flexure/shear
In-plane flexure
Out-of-plane flexure
Shear wall, in-plane shear
Cyclic loading conditions
800 mm
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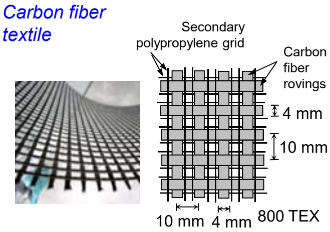
Materials used

Carbon fiber textile



Weight = 16 g/m^2

Nominal thickness of layer: 0.047 mm

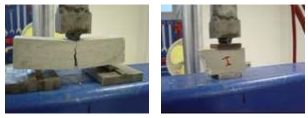
Tensile strength: 3350 MPa

Modulus of elasticity: 225 GPa

Masonry mortar

Compressive strength = 3.91 MPa

Modulus of rupture = 1.17 MPa

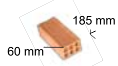


Inorganic binder (cementitious mortar)

Compressive strength = 31.36 MPa

Modulus of rupture = 5.77 MPa

6-hole clay bricks



Compressive strength:

// to perforations = 8.9 MPa

⊥ to perforations = 3.7 MPa

Masonry

⊥ to perforations
Compressive strength : 2 MPa
Modulus of elasticity: 1.7 GPa
Ultimate strain: 0.12%

// to perforations
Compressive strength: 4.3 MPa
Modulus of elasticity: 1.94 GPa
Ultimate strain: 0.22%

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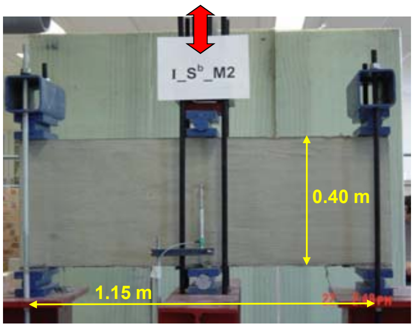

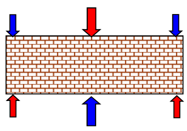

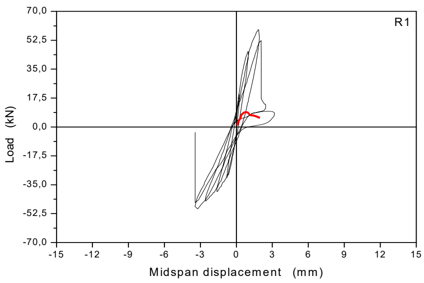

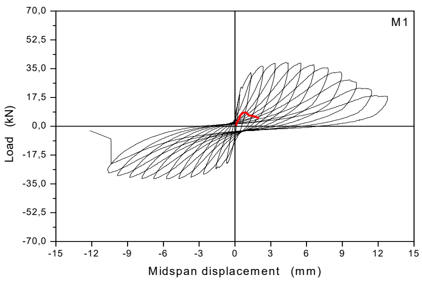

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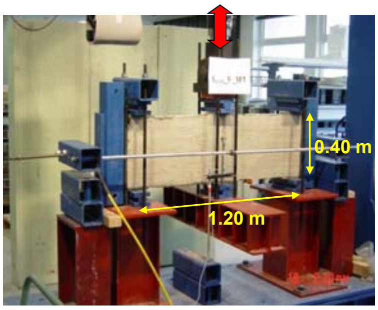

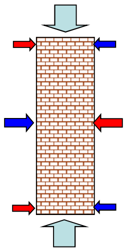

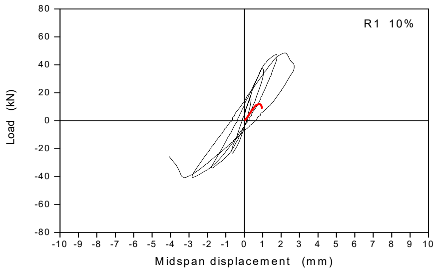

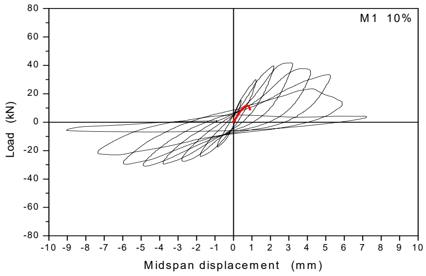

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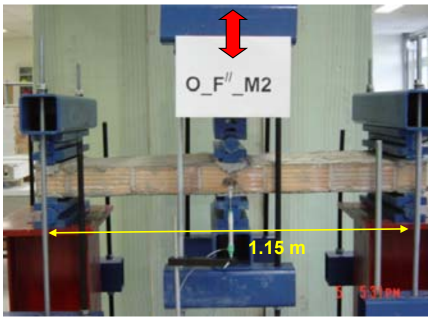

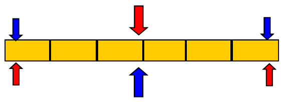

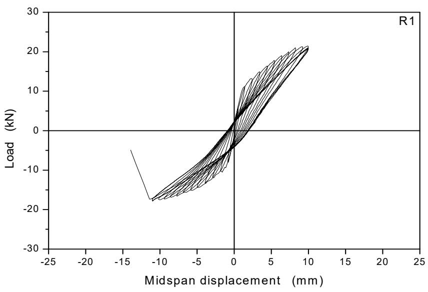

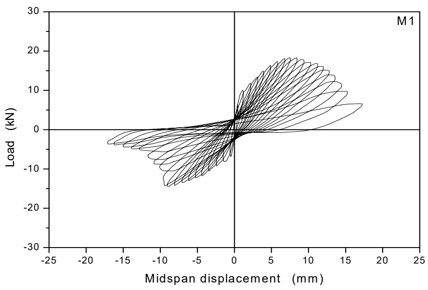

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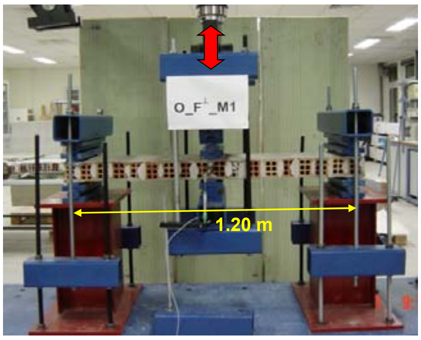

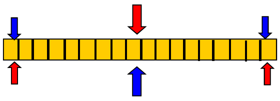

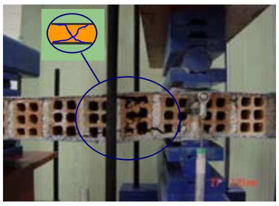

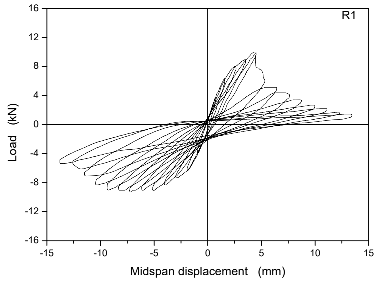

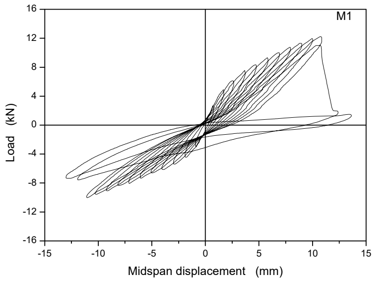

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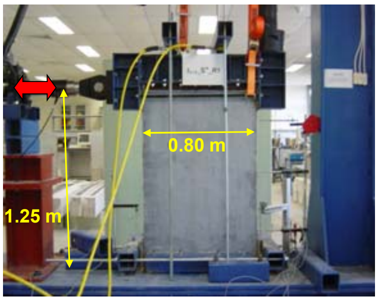

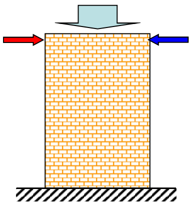

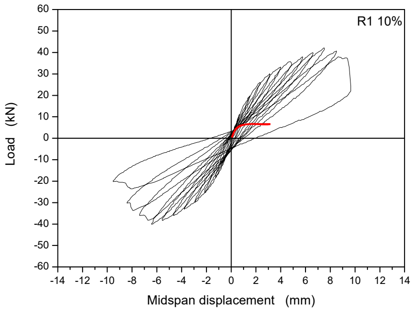

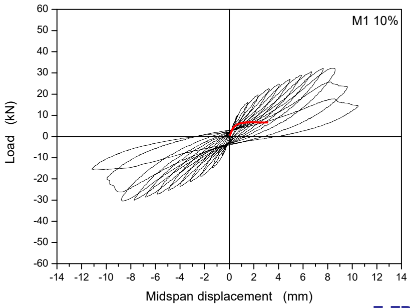

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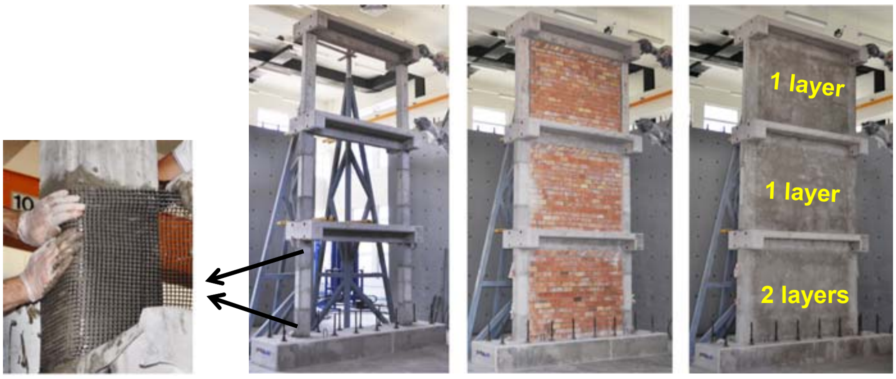
Seismic Retrofitting of Masonry-Infilled RC Frames with Textile-Reinforced Mortar (TRM) (Koutas et al. 2015)



- Shear strengthening of the columns ends with TRM before the infilling
- Application of the externally bonded layers of G-TRM on the faces of infills and proper connection to the members of the surrounding frame

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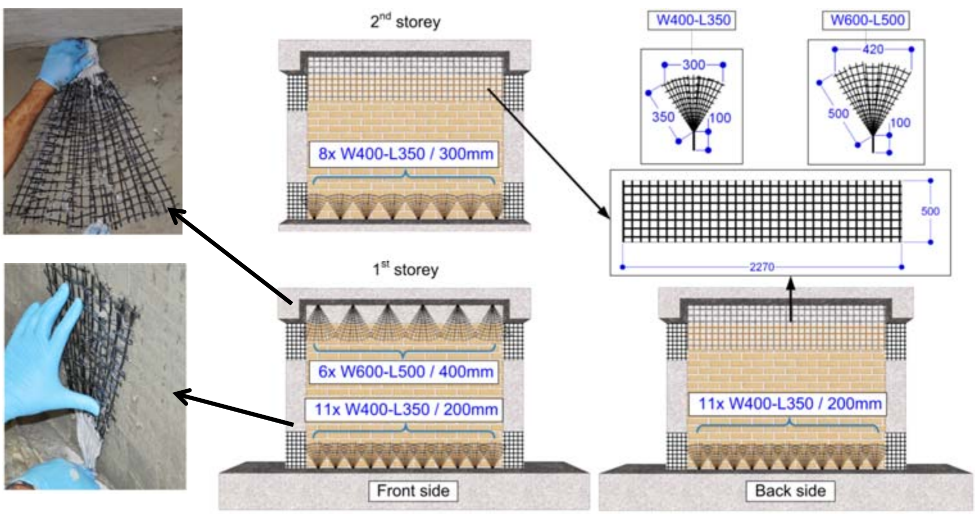

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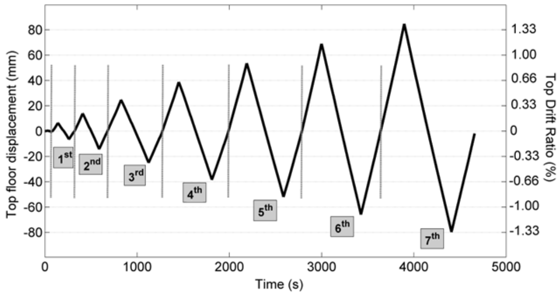
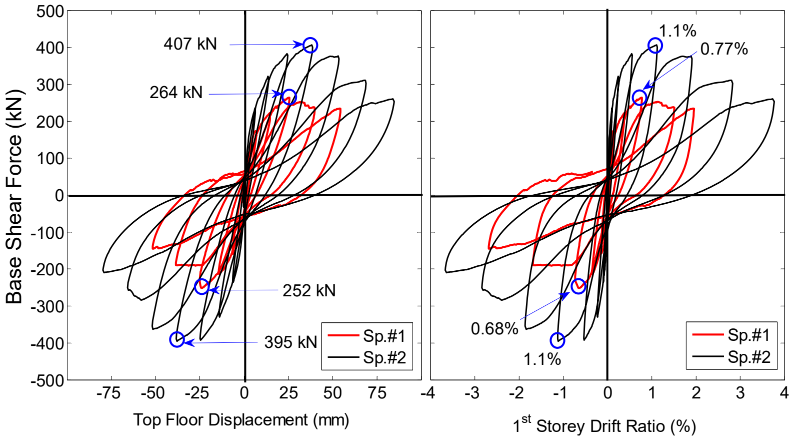
Test set-up

Height-wise triangular distribution of forces

Displacement pattern at the top floor:



T. TRIANTAFILLOU



Enhanced global response of the infilled frame both in terms of lateral strength and deformation capacity.

- **55% increase in the lateral strength**
- **56% higher deformation capacity at the top of the structure at ultimate strength state**

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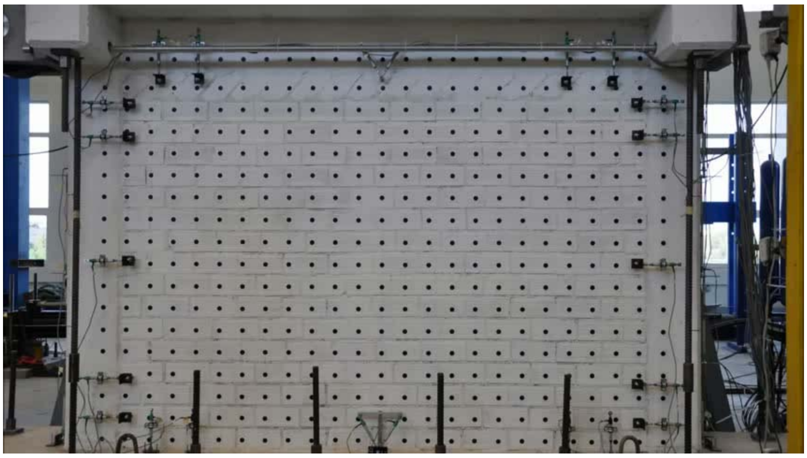
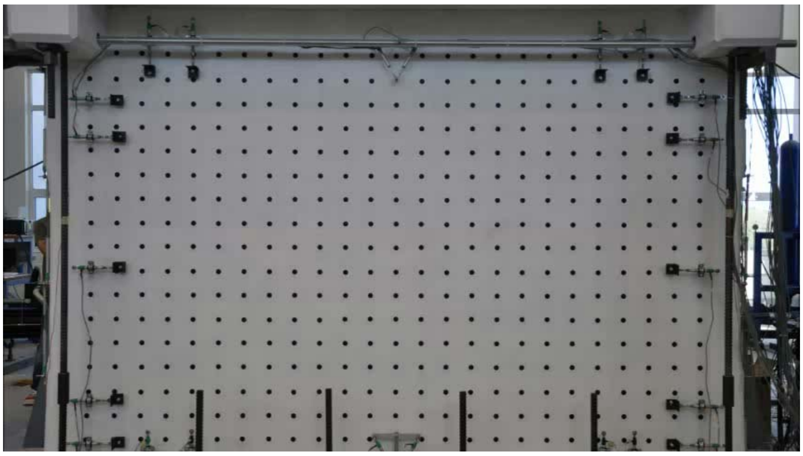
Control Frame – 1st storey : VIDEO 7

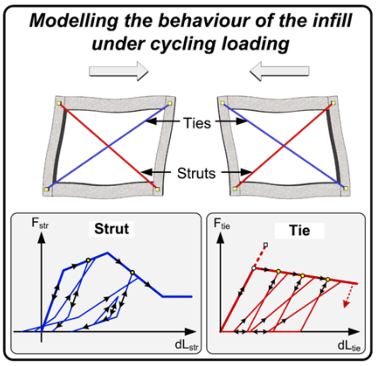


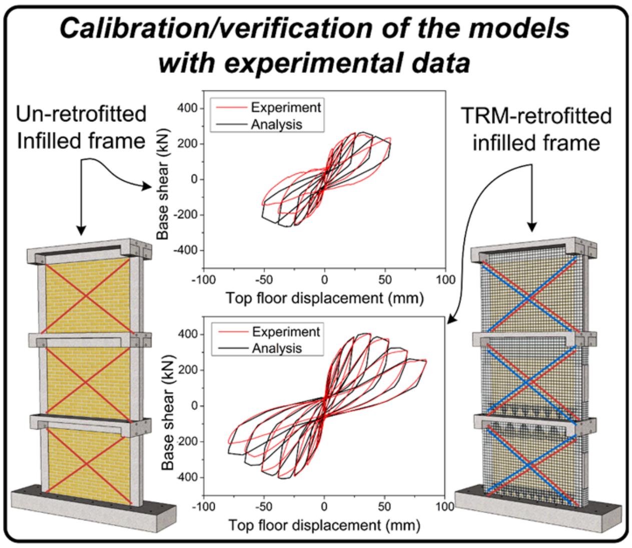
T. TRIANTAFILLOU

Retrofitted frame – 1st storey : VIDEO 8



T. TRIANTAFILLOU





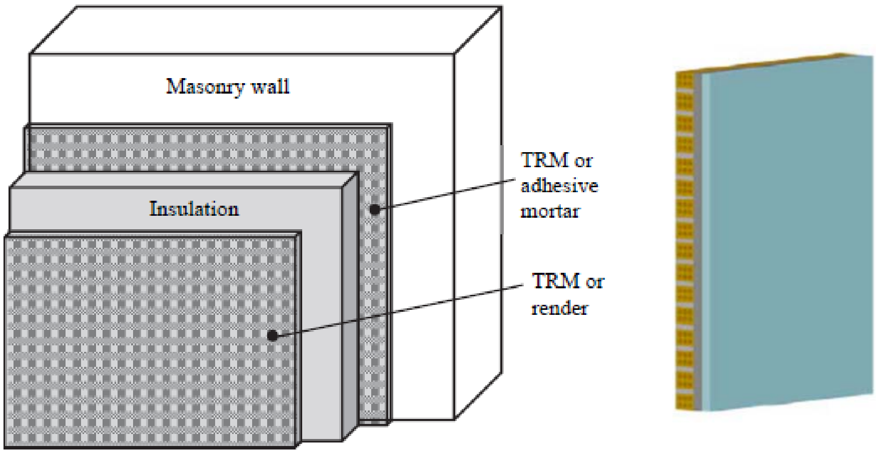
After proper calibration of the model: very good agreement with the experimental results

Combined Seismic + Energy Retrofitting !

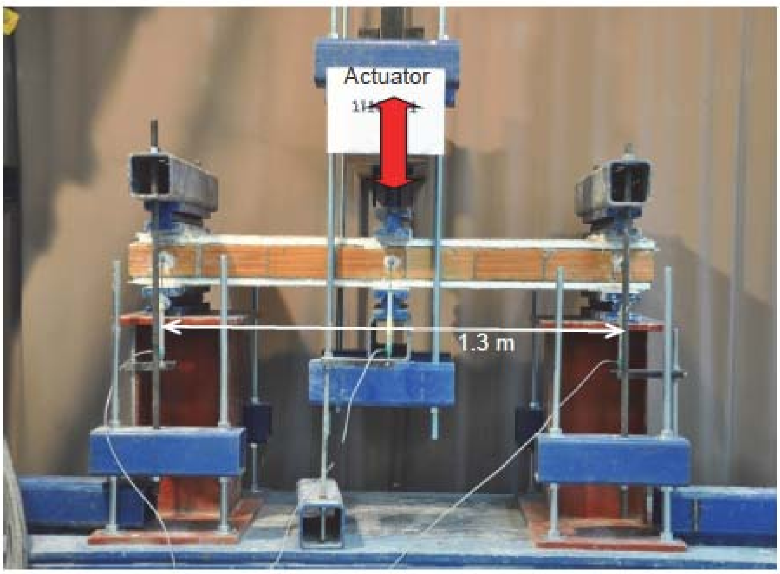
+

External thermal insulation

TRM retrofitting



T. TRIANTAFILLOU



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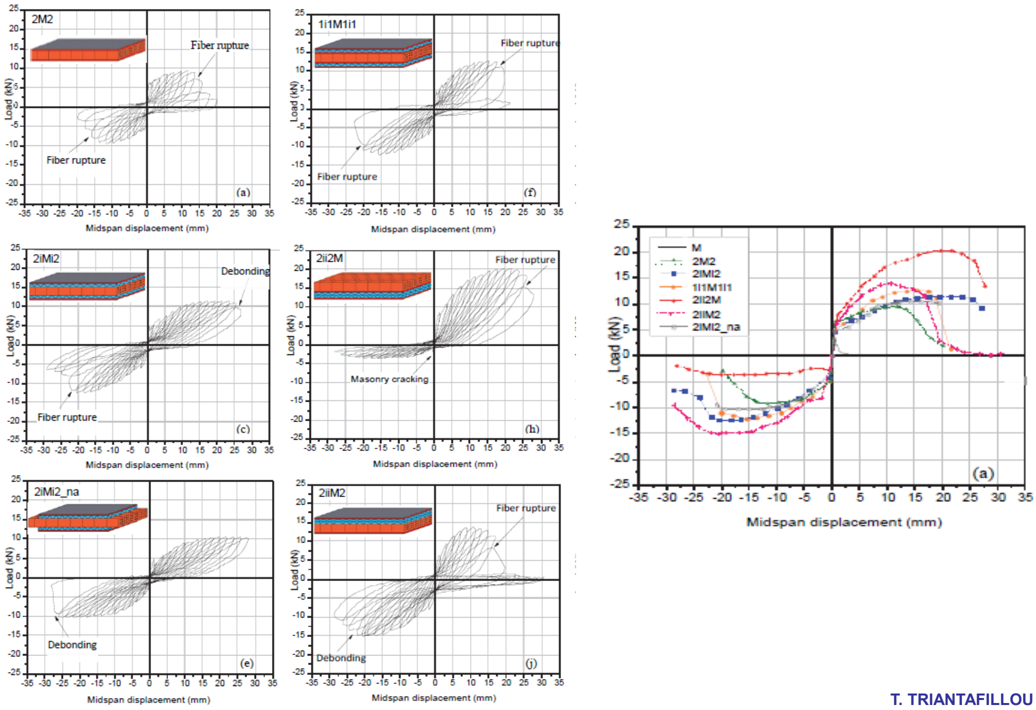


T. TRIANTAFILLOU

FRP HAVE BEEN AND WILL CONTINUE TO BE QUITE SUCCESSFUL FOR CONFINEMENT/SEISMIC RETROFITTING OF RC STRUCTURES

TRM PAVE THE WAY FOR EVEN MORE APPLICATIONS OF CONTINUOUS FIBER REINFORCEMENT IN STRUCTURAL UPGRADING, ESPECIALLY IN MASONRY STRUCTURES

T. TRIANTAFILLOU

Thank you for your attention !

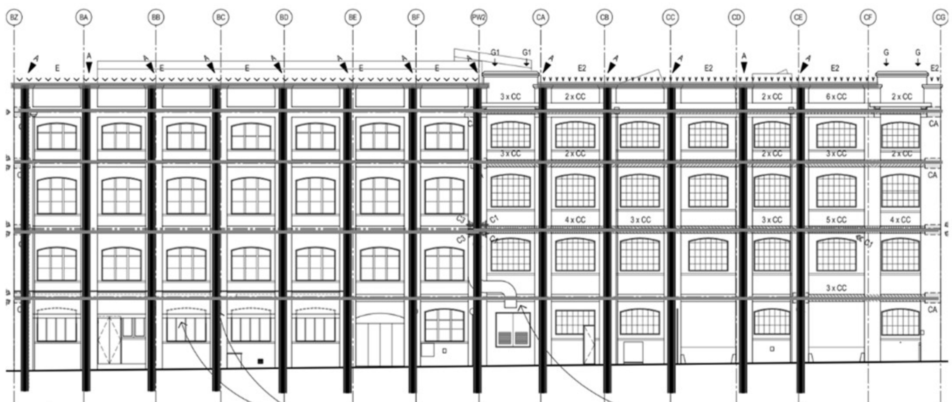
Session 4

NZ Case Studies

BBR Contech
Concrete Solutions
Mapei NZ

Auckland – Rhys Rogers
Wellington – Marc Stewart
Christchurch – Paul Dillon

Textile Centre
(Parnell, Auckland)

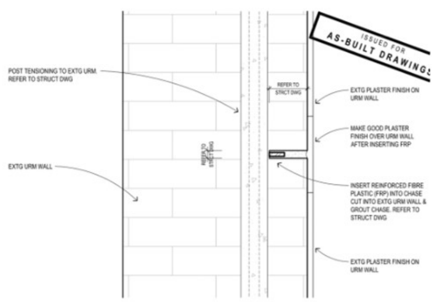


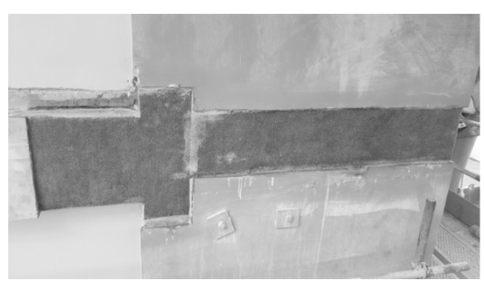
1. Core holes in the masonry piers from parapet down into foundation

2. Install and post-tension macalloy PT bars in the cored holes

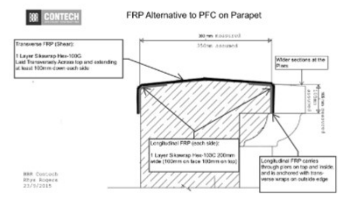
3. Install FRP tensile reinforcement to spandrels

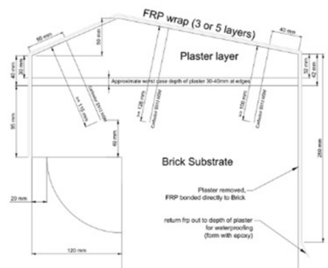
4. FRP wrap parapet cap so it acts as a beam spanning between piers







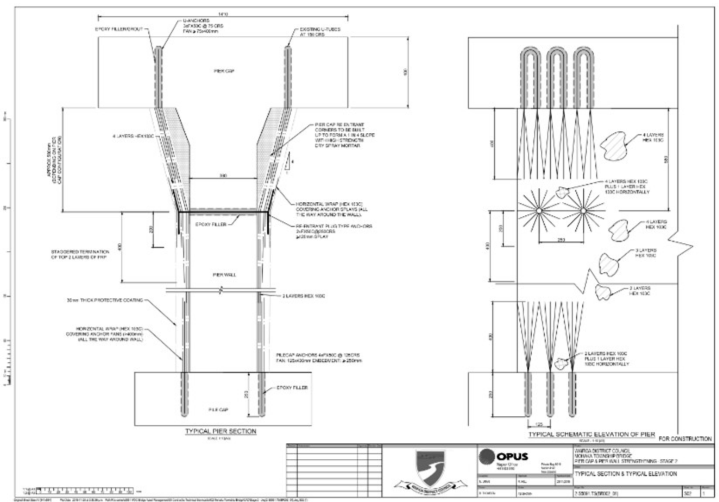




Mohaka Township River Bridge

(Wairoa)





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fib-Short Course: REINFORCING & STRENGTHENING OF STRUCTURES WITH ADVANCED COMPOSITES

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University Of Canterbury
Matariki Building
(Formerly The Registry Building)

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Strengthening of Historic URM Structure

Strengthening Consisted Of:

- New sprayed concrete walls
- New concrete raft slabs
- Structural steel
- TYFO advanced composite materials

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After Strengthening

After Completion

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After Completion

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Fully Functional, High Rise Hotel and Adjacent Carparking Structure

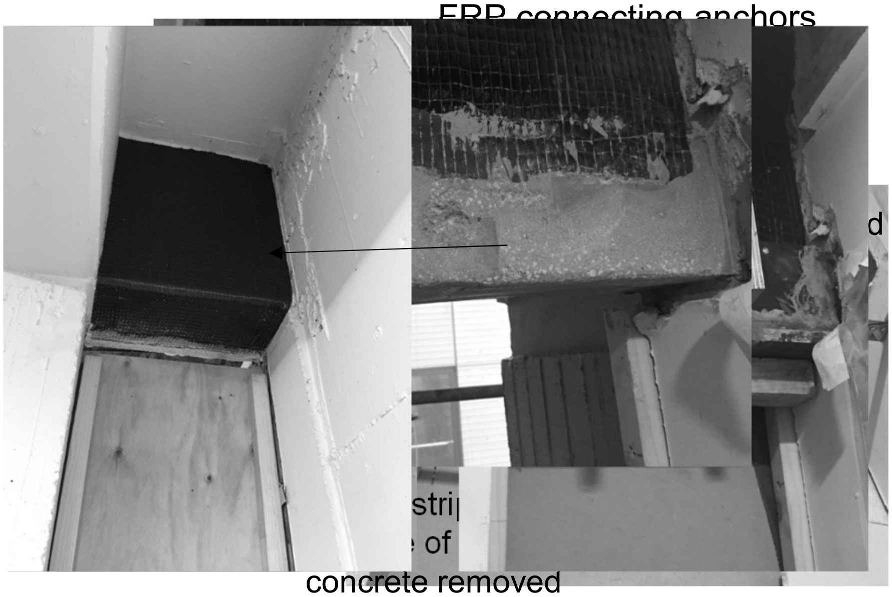
- Remained operational throughout the works

FRP composite strengthening works were multifaceted, including strengthening of:

- Shear walls
- Column confinement
- Beam confinement
- Irregular structural members
- “Dog bone” column/wall confinement/strengthening

Column confinement in operational hotel

Minimal clearance required to achieve wrapping



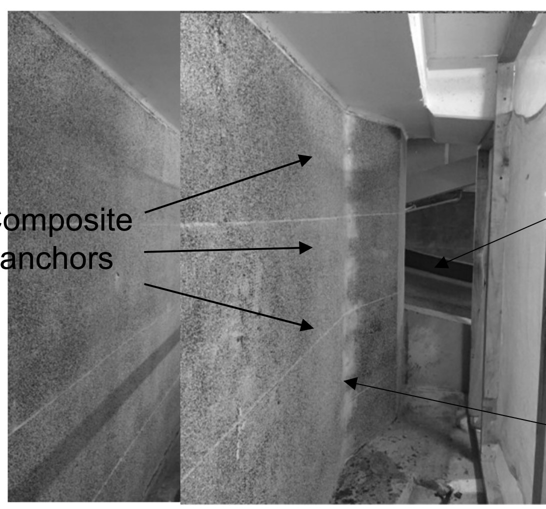
FRP connecting anchors

d

strip

e of

concrete removed



Composite anchors

operational

rking structure

vehicle ramp a

ent to retail

area screened off

he public

Anchors through "Dog Bone" section joining columns and shear wall

"Dog Bone" section of wall

Additional example of column confinement utilising composite anchors

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Strengthening of Plastic Hinge Zone of Beams

- Within multi-storied reinforced concrete structure

- Good example of full confinement of beams utilizing composite anchors to pass through obstructions.
- This logic/method can be utilised in many other situations where obstructions are present (walls adjoining columns, mullions etc).

Rebate filled with proprietary concrete repair material

Installation of composite anchors – topside

Composite anchors cured

ence

ting

holes

surface

hors

Prepa

Comp

remov

and o

holes

- Anchor holes filled with thickened epoxy

Second layer of Tyfo SEH51A installed, sandwiching Tyfo composite anchors between first and second layers

Installation and skimming complete

- Warning notification sign attached to installation to alert other contractors of installation

- Warning labels to remain

- Craig Harris. NCCT (NZ). Dip NZIM.
- MAPEI National Product Manager
- Structural Strengthening ,Building & Construction
- New Zealand

Christchurch Town Hall


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Shale Peak Bridge SH7

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15 Murphy St Wellington

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Session 5

FRP as internal reinforcement for RC/PC structures

Maurizio Guadagnini

FRP as internal reinforcement for RC structures

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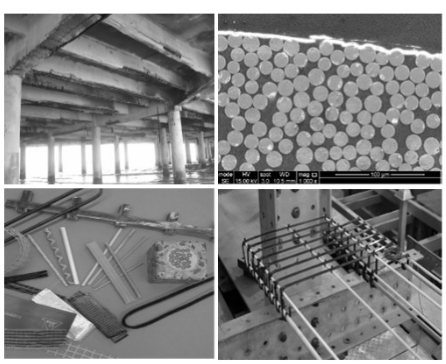
REINFORCING & STRENGTHENING OF STRUCTURES WITH ADVANCED COMPOSITES

Presented by Concrete NZ – Learned Society and The International Federation for Structural Concrete (*fib*)

Dr Maurizio Guadagnini
The University of Sheffield, UK

FRP as internal reinforcement for RC structures

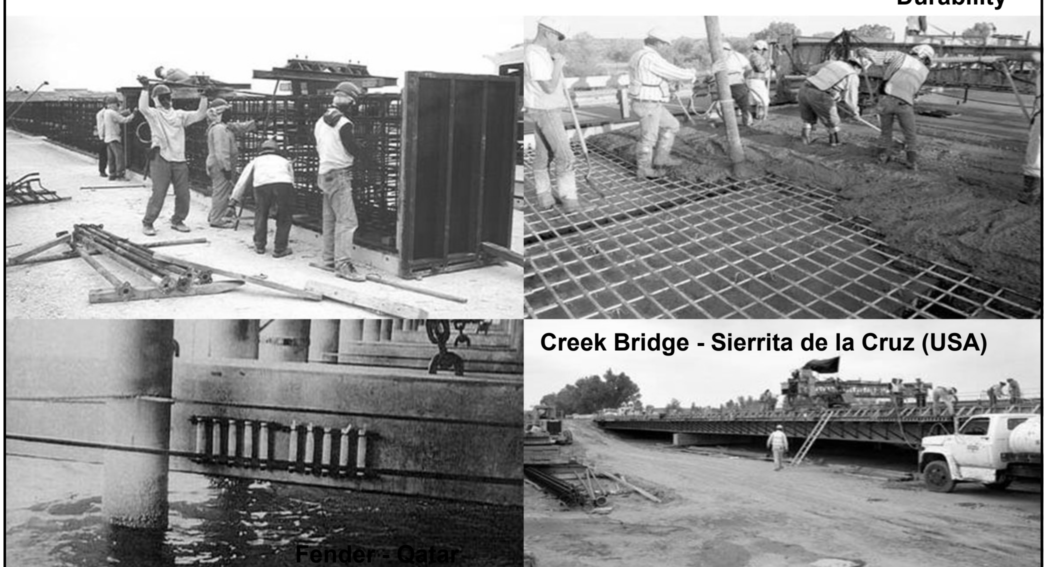


- Why FRP RC?
- Structural behaviour
- Bond
- Flexure
- Shear
- Detailing

fib CEB-FIP

Why FRP RC?

Durability



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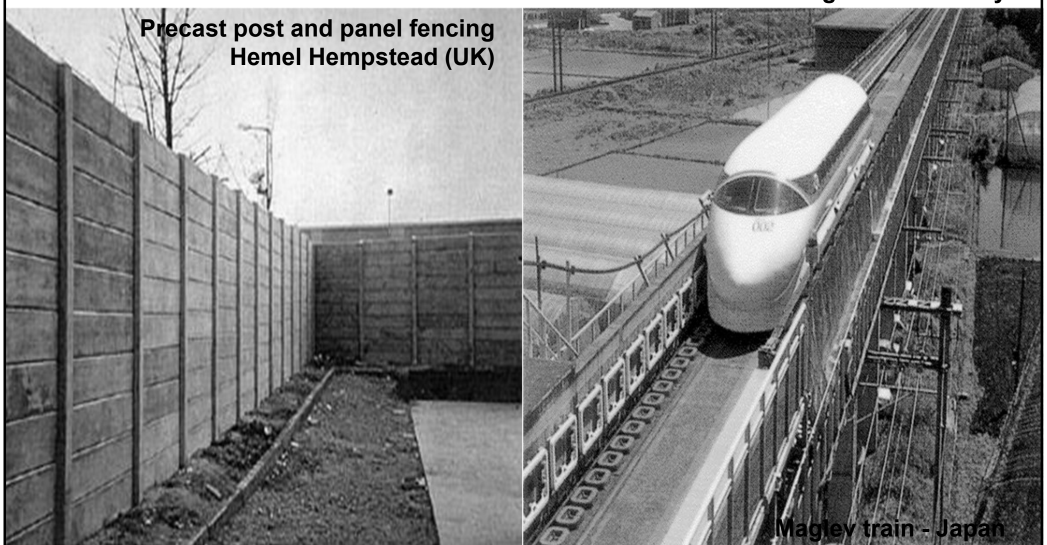
Why FRP RC?

Electromagnetic neutrality



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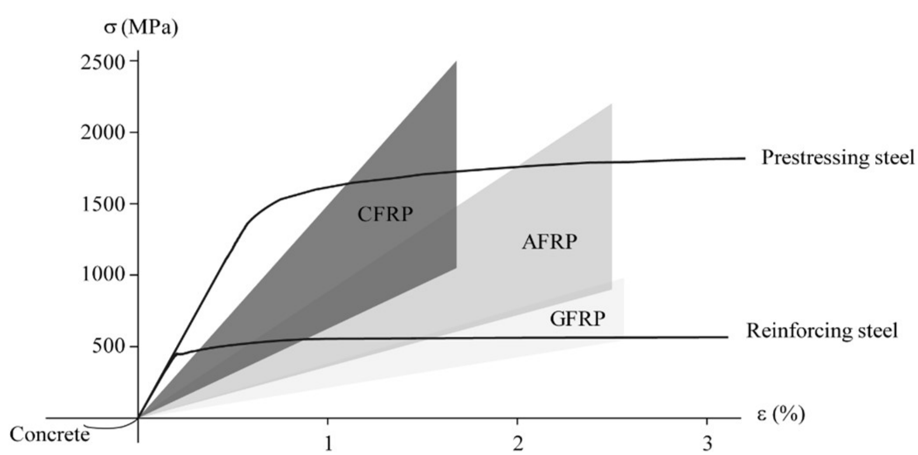
FRP vs Steel - Mechanical Properties



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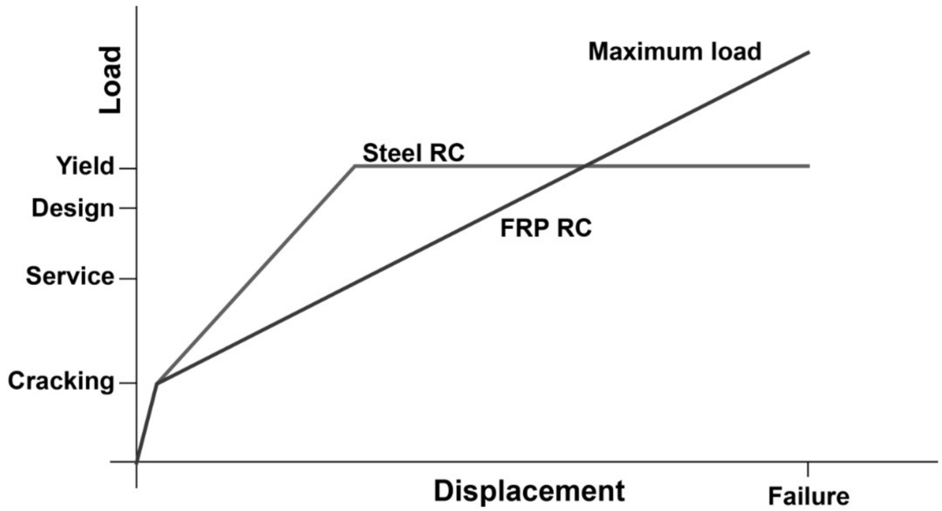
Ductility or Deformability?



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Bond of FRP internal reinforcement

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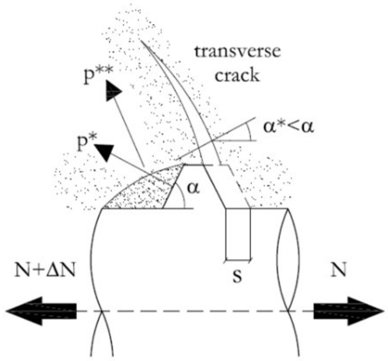
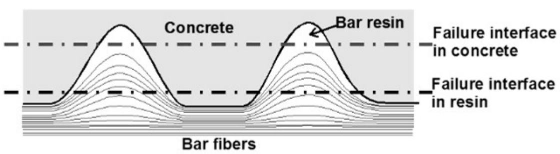
Bond of embedded reinforcement



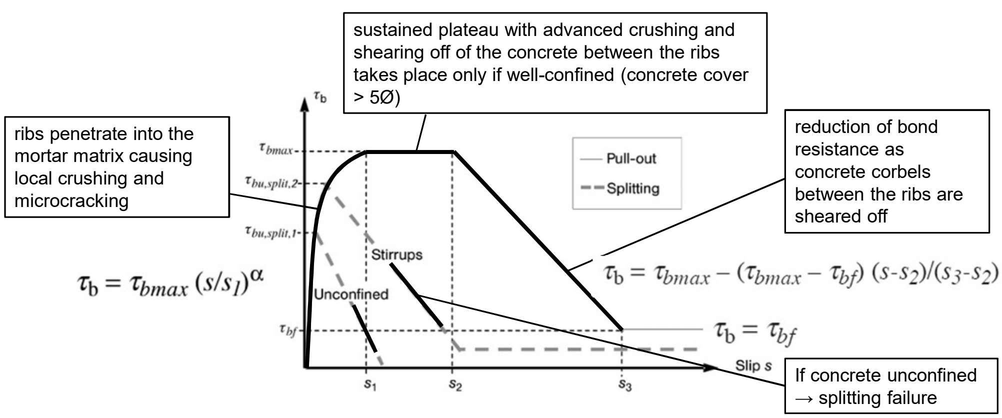
Steel Reinforcement



FRP Reinforcement

fib Model Code 2010 – steel ribbed bars

Bond properties depend on rib geometry (relative rib area), concrete strength, position and orientation of the bar during casting, state of stress, boundary conditions and concrete cover.



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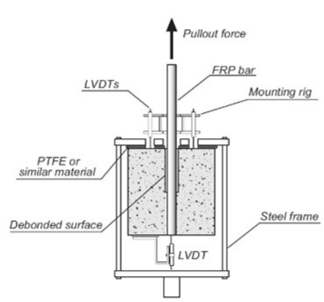
Bond of embedded FRP reinforcement

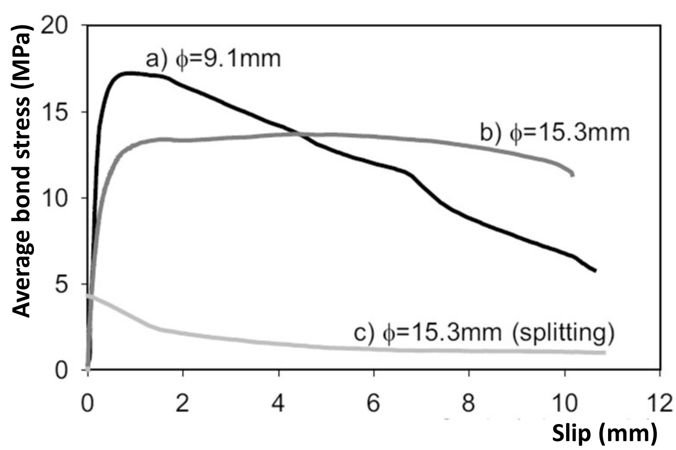
Bond behaviour of FRP reinforcement to concrete depends mainly on:
- reinforcement geometry
- surface characteristics

It **varies from that of conventional steel** reinforcement, given that:
- the **modulus of elasticity** of FRP is generally lower than that of steel, especially in the transverse direction;
- the **shear stiffness** of FRP is significantly lower than that of steel;
- the **surface deformations** relate to the resin matrix, which has a **lower shear strength** than steel.

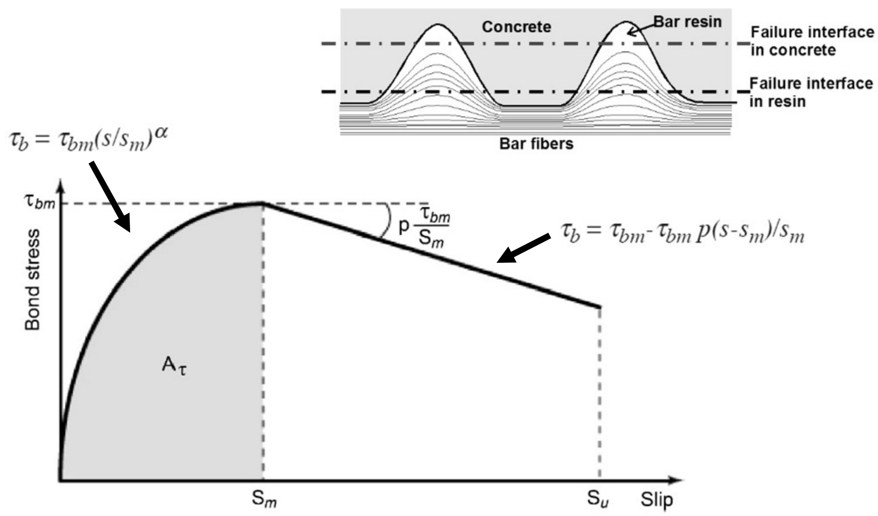
NOTE: It is generally possible to obtain bond strengths for non-metallic reinforcement of similar or greater magnitude than for steel reinforcement.

Bond-slip behaviour of embedded FRP reinforcement





Bond of embedded FRP reinforcement



The parameters have to be calibrated on the basis of experimental results

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Flexural design of FRP RC elements

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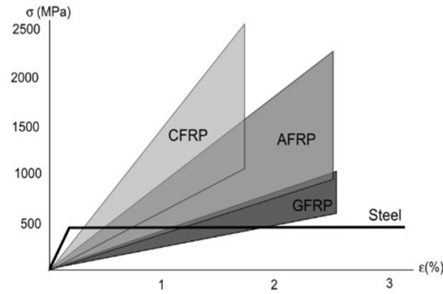
FRP properties

Compared to steel:

- High tensile strength
- Lower modulus of elasticity than steel
- Linear behaviour / no yielding



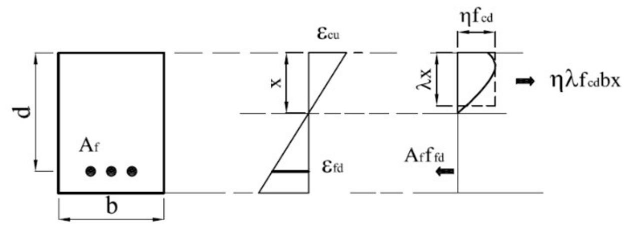
- Creep rupture (static fatigue)
- Compressive strength is neglected

Ultimate Limit State

- Perfect bond between FRP and concrete
- Concrete in compression is nonlinear; concrete in tension is ignored
- FRP is linear elastic
- Types of failure:
  - **Concrete crushing**: $\rho_f > \rho_{fb}$ ($\varepsilon_c = \varepsilon_{cu}$); more desirable: more progressive; higher deformability
  - **FRP failure**: $\rho_f < \rho_{fb}$ ($\varepsilon_f = \varepsilon_{fd}$); more sudden

Type of failure

- **Balanced condition**: ε_{cu} and ε_{fd} reached simultaneously



$$\rho_f = \frac{A_f}{bd}$$

- From compatibility, equilibrium and constitutive laws:

$$\boxed{\frac{x}{d} = \frac{\varepsilon_{cu}}{\varepsilon_{fd} + \varepsilon_{cu}}}$$

$$\boxed{\eta\lambda\, f_{cd}\, bx = A_f f_{fd}} \Rightarrow$$

$$\boxed{f_{fd} = E_f \varepsilon_{fd}}$$

Balanced reinforcement ratio

$$\boxed{\rho_{fb} = \eta\lambda \frac{f_{cd}}{f_{fd}} \frac{E_f \varepsilon_{cu}}{E_f \varepsilon_{cu} + f_{fd}}}$$

$\rho_f < \rho_{fb}$: FRP failure
$\rho_f > \rho_{fb}$: Concrete failure

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EC2-based approach: overview

- **Aim:** Factored flexural capacity > Factored design moment $M_{Rd} \geq M_{Ed}$
- **Material partial factors:** γ_c as for steel RC, γ_f defined in ETA document (1.15÷1.6, 1.5)
- **Long-term properties** in ETA document
- **Environmental factors** may be defined (CNR 2007)
- **Flexural strength**: equilibrium, compatibility, mode of failure

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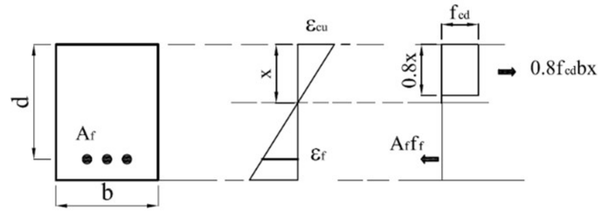
Design for concrete crushing

Assumptions:
EC2 rectangular block
f_{ck}<50 MPa
ε_{cu} = 0.0035

$\rho_f > \rho_{fb}$ ($\varepsilon_c = \varepsilon_{cu}$)

$$\boxed{\rho_{fb} = 0.8\frac{f_{cd}}{f_{fd}}\frac{E_f\varepsilon_{cu}}{E_f\varepsilon_{cu} + f_{fd}}}$$



- Based on equilibrium and compatibility, stress in FRP can be obtained:

$$\boxed{f_f = \left(\sqrt{\frac{(E_f\varepsilon_{cu})^2}{4} + \frac{0.8 f_{cd}}{\rho_f}E_f\varepsilon_{cu}} - 0.5E_f\varepsilon_{cu}\right) \le f_{fd}} \Rightarrow x = \frac{A_f f_f}{0.8 f_{cd} b}$$

- The moment capacity is:

$$\boxed{M_u = A_f f_f (d - 0.4x)}$$
$$\boxed{M_u = 0.8 f_{cd} bx (d - 0.4x)}$$

or alternatively

$$\boxed{M_u = \rho_f f_f \left(1 - 0.5\frac{\rho_f f_f}{f_{cd}}\right) bd^2}$$

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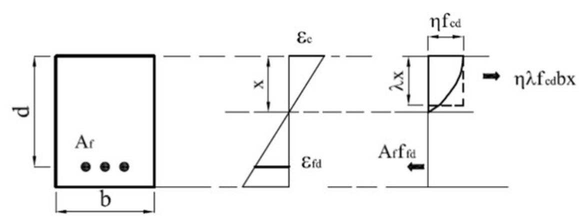
Design for FRP rupture

ε_{cu} = 0.0035

$\rho_f < \rho_{fb}$ ($\varepsilon_c < \varepsilon_{cu}$)



- Two unknowns: ε_c and neutral axis depth (x)
- Theoretically, rectangular stress block is not applicable
- Factors η, λ defining rectangular stress block are unknown: iterative process required for a precise analysis
- Approximate equation is obtained using rectangular stress block (η=1, λ=0.8)

$$\boxed{M_u = \rho_f f_{fd} \left(1 - 0.5\frac{\rho_f f_{fd}}{f_{cd}}\right) bd^2}$$

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ACI 440 design: overview

- Consistent with ACI 318
- Aim: Member capacity > Flexural demand $\phi \cdot M_n \geq M_u$
- Concrete crushing and FRP rupture are accepted
- Flexural strength: equilibrium, compatibility, control of mode of failure

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FRP properties

- Environmental reduction factor for design strength and strain: $\boxed{f_{fu} = C_E \cdot f_{fu}^*}$ $\boxed{\varepsilon_{fu} = C_E \cdot \varepsilon_{fu}^*}$

Table 2.1 Environmental reduction factors

Exposure condition	Fibre type	C_E
Concrete not exposed to earth and weather	Carbon	1.0
	Glass	0.8
	Aramid	0.9
Concrete exposed to earth and weather	Carbon	0.9
	Glass	0.7
	Aramid	0.8

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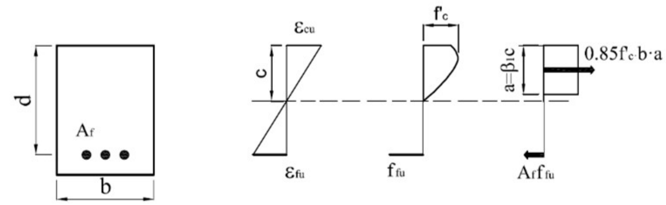
Balanced reinforcement ratio

Assumptions:

ACI rectangular block
ε_{cu} = 0.0030



Eq. stress block (Whitney): α_1 = 0.85
β_1 = 0.85 for $f'_c \le$ 27.6 MPa; $\beta_1 = 0.85 - 0.05/7 \cdot (f'_c - 27.6)$ for f'_c > 27.6 MPa

- Balanced condition: ε_{cu} and ε_{fu} simultaneously

$$\boxed{\rho_{fb} = 0.85\beta_1 \frac{f_c'}{f_{fu}} \frac{E_f \varepsilon_{cu}}{E_f \varepsilon_{cu} + f_{fu}}}$$

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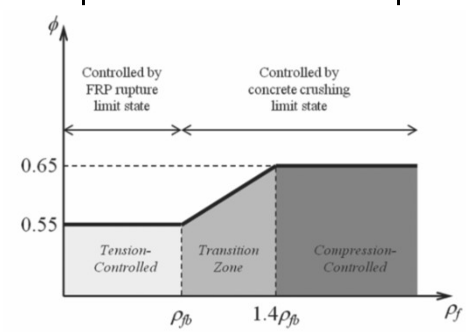
Strength reduction factor: ϕ

- Adopts different values depending on the mode of failure



$$\phi = \begin{cases} 0.55 & \text{if} \quad \rho_f < \rho_{fb} \\ 0.65 & \text{if} \quad \rho_f > 1.4\rho_{fb} \\ 0.3 + 0.25\dfrac{\rho_f}{\rho_{fb}} & \rho_{fb} < \rho < 1.4\rho_{fb} \end{cases}$$

- $\rho_f < \rho_{fb}$: failure controlled by FRP rupture
- $\rho_{fb} < \rho_f < 1.4\,\rho_{fb}$: transition zone (takes into account that if f'_c is higher than specified the member can fail due to FRP rupture)
- $\rho_f > 1.4\,\rho_{fb}$: failure controlled by concrete crushing

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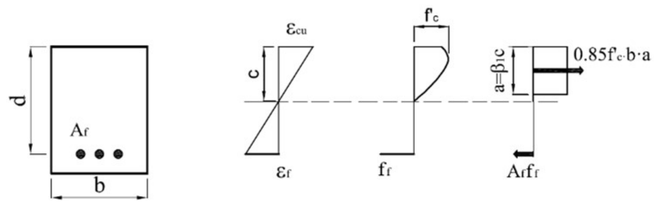
Design for concrete crushing

ε_{cu} = 0.003
$\rho_f > \rho_{fb}$ ($\varepsilon_c = \varepsilon_{cu}$)



- Based on equilibrium and compatibility the stress in FRP can be obtained

$$f_f = \left(\sqrt{\frac{(E_f \varepsilon_{cu})^2}{4} + \frac{0.85\beta_1 f_c'}{\rho_f} E_f \varepsilon_{cu}} - 0.5 E_f \varepsilon_{cu} \right) \le f_{fu} \quad \Rightarrow \quad a = \frac{A_f f_f}{0.85 f_c' b}$$

- The nominal moment capacity is

$$M_n = A_f f_f \left(d - \frac{a}{2} \right)$$

or alternatively

$$M_n = \rho_f f_f \left(1 - 0.59 \frac{\rho_f f_f}{f_c'} \right) b d^2$$

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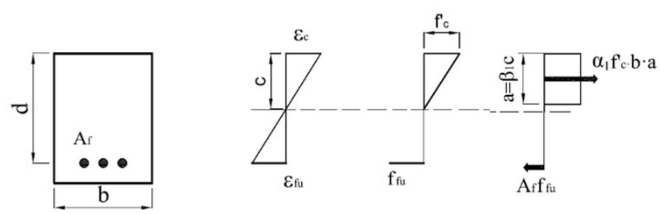
Design for FRP rupture

ε_{cu} = 0.0030
$\rho_f < \rho_{fb}$ ($\varepsilon_c < \varepsilon_{cu}$)



- Two unknowns: ε_c and neutral axis depth (c)
- ACI stress block is not applicable
- Factors β_1, α_1 defining rectangular stress block are unknown: iterative process required
- Approximate and conservative equation

$$M_n = A_f f_{fu} \left(d - \frac{\beta_1 c_b}{2} \right)$$

$$c_b = \left(\frac{\varepsilon_{cu}}{\varepsilon_{cu} + \varepsilon_{fu}} \right) d$$

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Minimum reinforcement

- A minimum flexural reinforcement must be provided to prevent failure upon concrete cracking ($\phi \cdot M_n > M_{cr}$)

$$A_{f,\min,} = \frac{4.9\sqrt{f_c'}}{f_{fu}} b_w d \geq \frac{330}{f_{fu}} b_w d \quad \text{(psi)}$$

$$A_{f,\min} = 0.41 \cdot \frac{\sqrt{f'_c}}{f_{fu}} b_w d \geq \frac{2.3}{f_{fu}} b_w d \quad \text{(MPa)}$$

- If failure is not controlled by FRP rupture the minimum flexural reinforcement is achieved automatically

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Serviceability Limit State

- Often governs the design
- Materials linear behaviour in the service range
- Cracking and tension-stiffening
- Creep and shrinkage of concrete
- Equations similar to steel RC (changes in coefficients)

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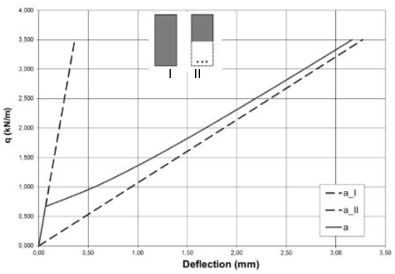
Deflections (EC2 approach)

- EC2 equations for both short and long-term apply
- Deformations obtained interpolating between uncracked and fully cracked states

$$\boxed{a = (1-\zeta)a_I + \zeta a_{II}} \qquad \zeta = 1 - \beta\left(\frac{M_{cr}}{M_s}\right)^2$$



- a = curvature for rigorous calculation; deflection as simplification
- β = accounts for bond, duration/repeated loading (std. 0,5-1)

- Alternatively, an effective moment of inertia may be defined (fib 2013, Balazs et al. 2013)

$$\boxed{I_{eff} = \frac{I_I \cdot I_{II}}{\zeta \cdot I_I + (1-\zeta) \cdot I_{II}} \le I_I} \qquad \boxed{I_{eff} = \frac{I_{II}}{1-\beta\left(1-\frac{I_{II}}{I_I}\right)\left(\frac{M_{cr}}{M}\right)^2} \le I_I}$$

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Long-term deflections (EC2 approach)

- Creep of concrete: Immediate plus creep deflection is obtained using an effective modulus of elasticity

$$\boxed{E_{c,ef} = \frac{E_c}{1+\varphi}}$$

φ = creep coefficient

- Shrinkage of concrete: Deflection is obtained from integration of shrinkage curvatures

$$\boxed{C_{cs} = \varepsilon_{cs} \cdot \alpha_e \cdot \frac{S}{I}}$$

ε_{cs} = free shrinkage strain
α_e = effective modular ratio
S = first moment of reinforcement area about the centroid of the section

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Deflections (ACI approach)

Short-term

- Calculations based on elastic behaviour
- Use of an effective moment of inertia I_e (constant along the member)

$$\boxed{I_e = \frac{I_{cr}}{1-\gamma\left(\frac{M_{cr}}{M_a}\right)^2\left(1-\frac{I_{cr}}{I_g}\right)} \leq I_g}$$

- γ depends on load and boundaries and accounts for change in stiffness along the beam. For a simply supported beam, uniformly distributed load (Bischoff and Gross 2011)

$$\gamma = 1.72 - 0.72\left(M_{cr}/M_a\right)$$

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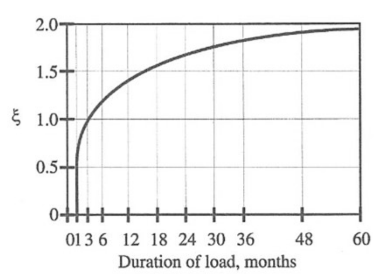
Long-term deflections (ACI approach)

- Time-dependent effects due to creep and shrinkage

$$\boxed{\Delta_{LT,sus} = \lambda\left(\Delta_i\right)_{sus}} \qquad \lambda = 0.6\xi$$

Duration of load	ξ
3 months	1.0
6 months	1.2
12 months	1.4
5 years or more	2.0



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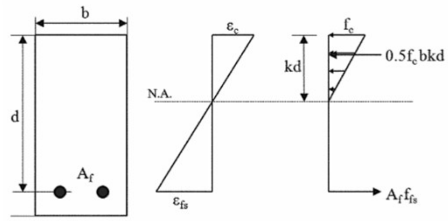
Creep rupture and fatigue (ACI 440.1R)

- FRP bars subjected to constant load over time can fail by creep rupture
- To avoid creep rupture, sustained stresses in FRP bars should be limited ($M_{s,sus}$: unfactored moment due to sustained loads)

$$\boxed{f_{fs,sus} = M_{s,sus} \frac{n_f d(1-k)}{I_{cr}}}$$



Fibre type	GFRP	AFRP	CFRP
Creep rupture stress limit $f_{f,s}$	$0.20\ f_{fu}$	$0.30\ f_{fu}$	$0.55\ f_{fu}$

- Same limitations are applied to FRP stress for fatigue regimes, using the moment of sustained loads plus fatigue loading cycle

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Shear design of FRP RC elements

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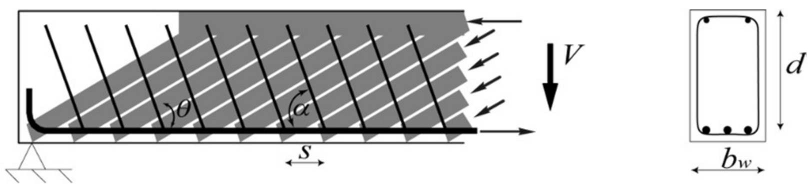
Truss Analogy

Fixed angle truss analogy with strut inclination θ=45°



Variable angle truss analogy with strut inclination 21.8°<θ<45°

Truss Analogy

Fixed angle

θ=45°

$$V = V_c + V_s$$

Variable angle

21.8°<θ<45°

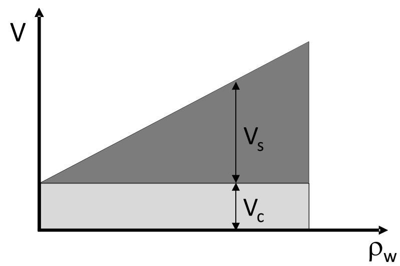
$$V = V_c$$

or

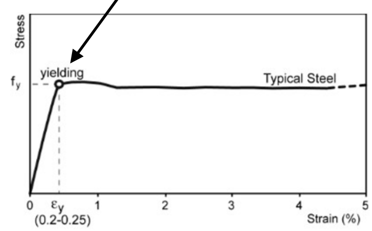
$$V = V_s$$

Design approach for steel RC

The design approach for steel RC relies on:

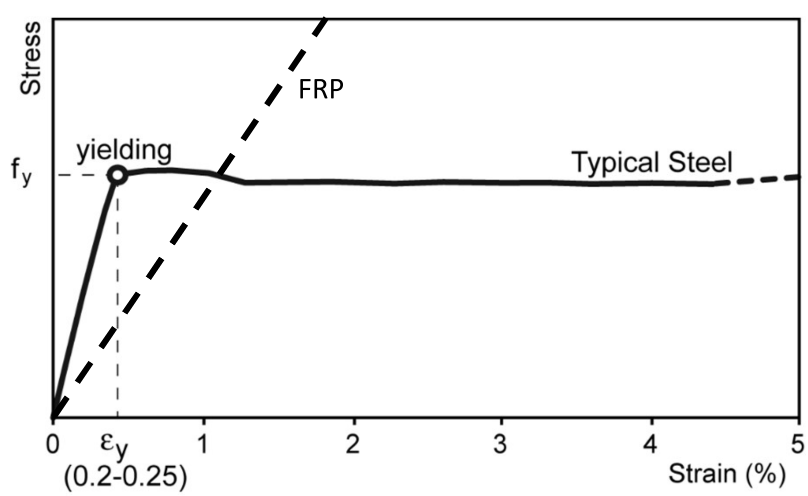


$$V_s = \frac{f_y \cdot A_v}{s} d$$

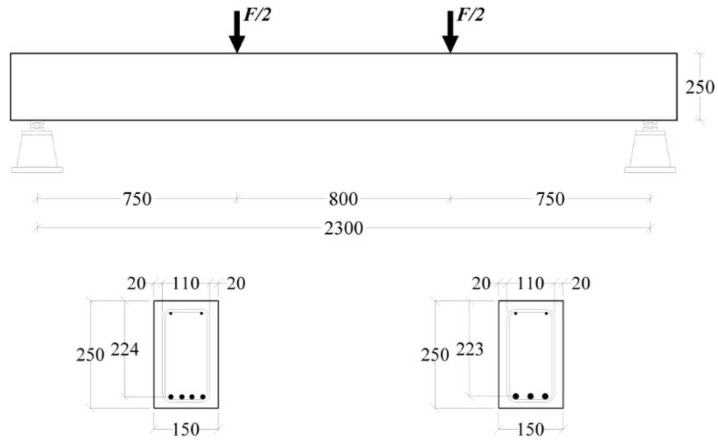


$$V = V_c + V_s$$

Reinforcement properties



Steel RC vs. FRP RC



Main reinforcement:
Steel bars, 4 Ø 12, 452 mm^2

Main reinforcement:
GFRP bars, 3 Ø 13.5, 429 mm^2

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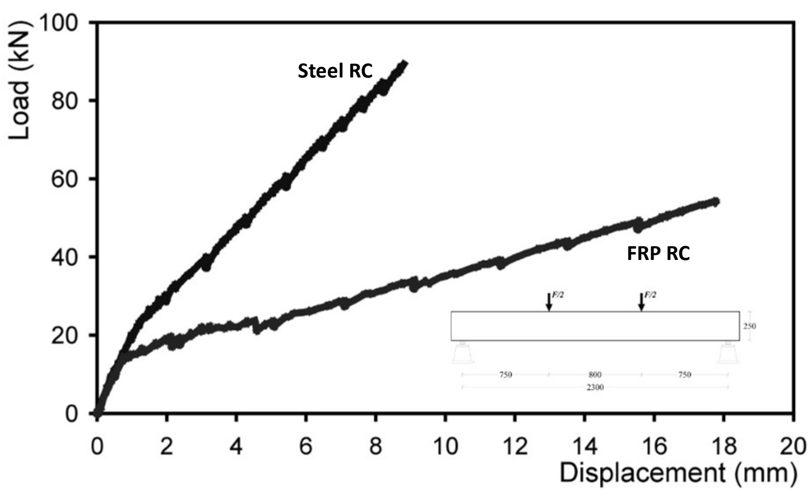
Steel RC vs. FRP RC (shear performance)



Guadagnini, Pilakoutas & Waldron (2006)

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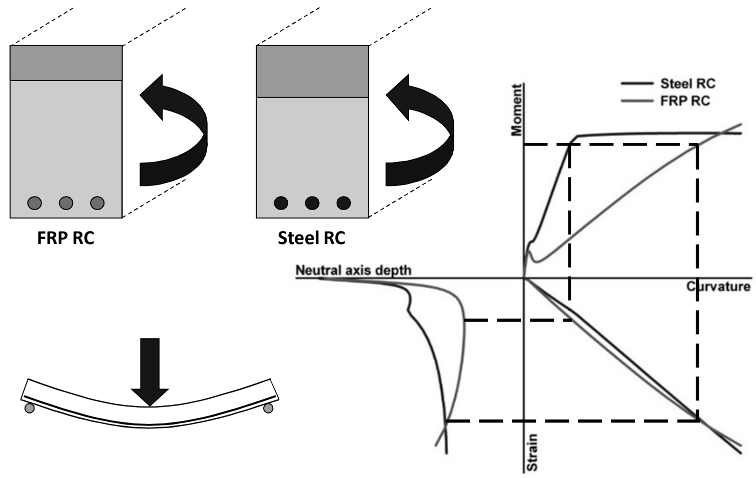
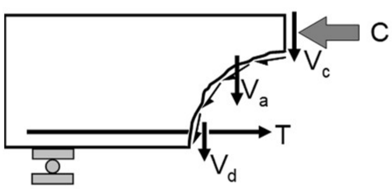
Shear resisting mechanisms

Beams without shear reinforcement

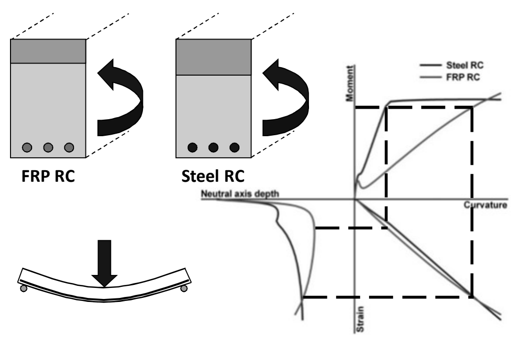


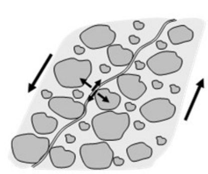
Shear resisting mechanisms

Beams without shear reinforcement

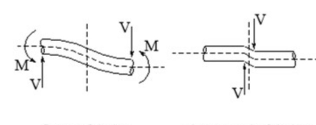


V_c = Concrete in compression





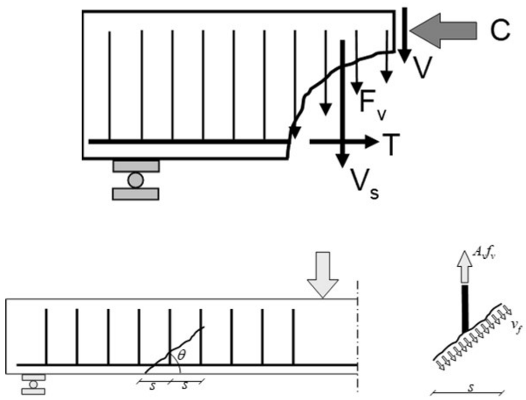
V_a = Aggregate interlock

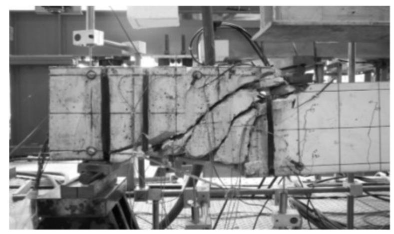


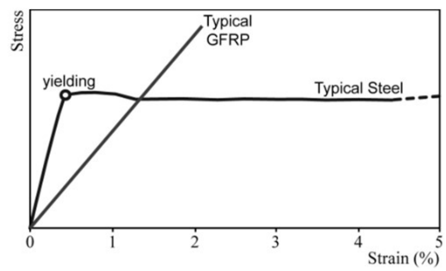
V_d = Dowel action

Shear resisting mechanisms

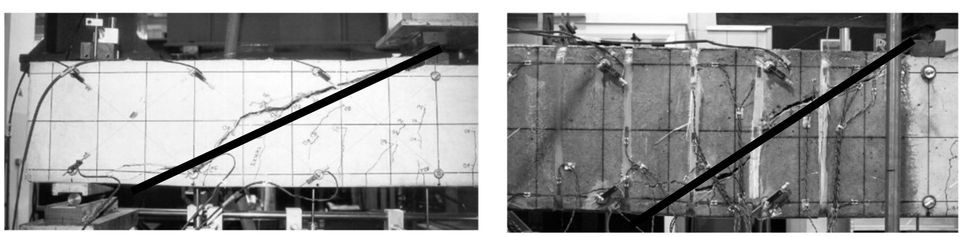
Contribution of vertical reinforcement



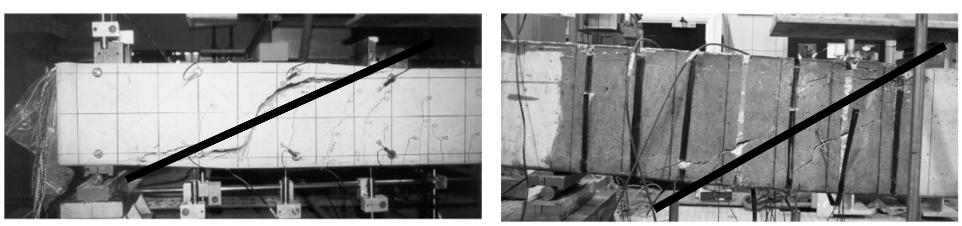




Shear failures



Steel flexural reinforcement

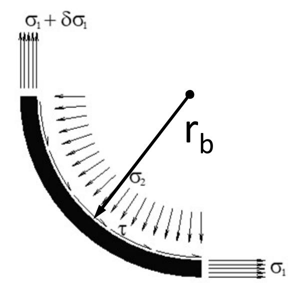


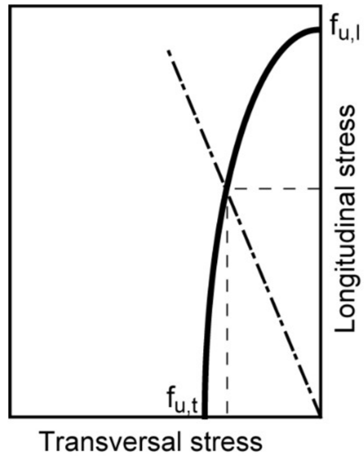
GFRP flexural reinforcement

Guadagnini, Pilakoutas & Waldron (2006)

Possible failure of shear links

The tensile strength of FRP bars is largely reduced when subjected to a biaxial state of stress





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Shear resistance of RC beams

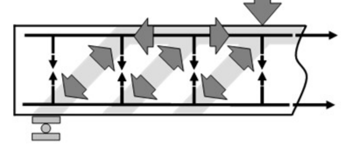
V_c

Concrete contribution

Empirical equation
- concrete in compression
- aggregate interlock
- dowel action

V_s

Contribution of shear reinforcement



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Steel RC code equations

Concrete contribution

EC2

$$V_{Rd,c} = \left[C_{Rd,c} \left(1 + \sqrt{\frac{200}{d}} \right) \left(100 \cdot \frac{A_{sl}}{b_w d} \cdot f_{ck} \right)^{1/3} \right] b_w d$$

ACI 318

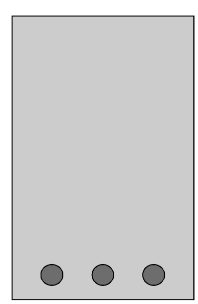
$$V_c = \left(0.16\sqrt{f_c'} + 17\rho \frac{V_u d}{M_u} \right) b_w d \qquad V_c = \frac{1}{6}\sqrt{f_c'} b_w d \ \textbf{?}$$

Design approach for FRP RC

Concrete contribution

Strain Approach:

if a design using FRP reinforcement maintains the same strain as when conventional steel is used and the same design forces are developed, then that design will lead to the same safe result



Design approach for FRP RC

Concrete contribution

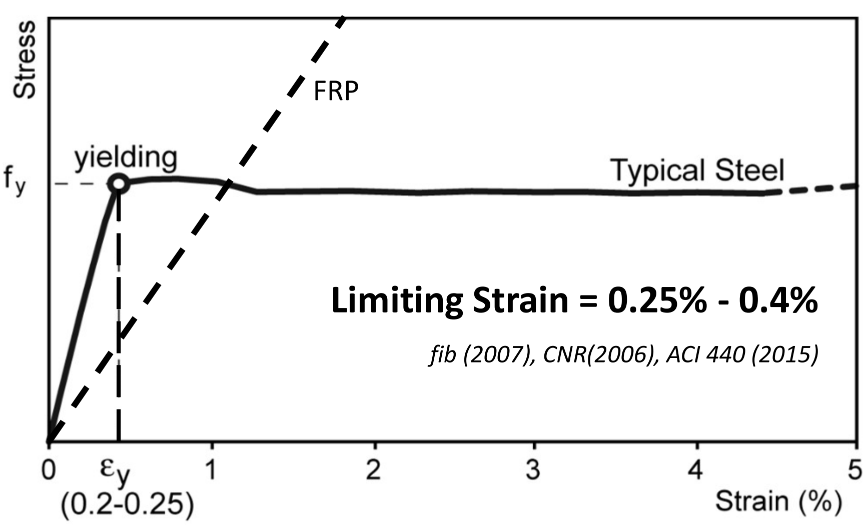
$$\varepsilon_f = \varepsilon_s \qquad F_f = F_s$$

$$F_f = \varepsilon_f \cdot E_f \cdot A_f = \varepsilon_s \cdot E_s \cdot A_s = F_s$$

$$A_{s(eq)} = A_f \cdot \frac{E_f}{E_s}$$

Design approach for FRP RC

Contribution of shear reinforcement



Limiting Strain = 0.25% - 0.4%

fib (2007), CNR(2006), ACI 440 (2015)

FRP RC modified equations (EC2)

Modifications for FRP RC

$$A_{s(eq)} = A_f \cdot \frac{E_f}{E_S}$$

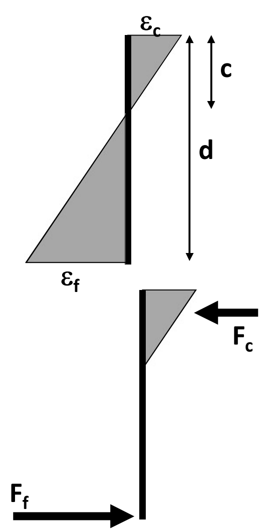
$$V_{Rd,c} = \left[C_{Rd,c} \left(1 + \sqrt{\frac{200}{d}} \right) \left(100 \cdot \frac{A_f}{b_w d} \cdot \frac{E_f}{E_s} \cdot f_{ck} \right)^{1/3} \right] b_w d$$

ACI 440-1R modifications (2015)

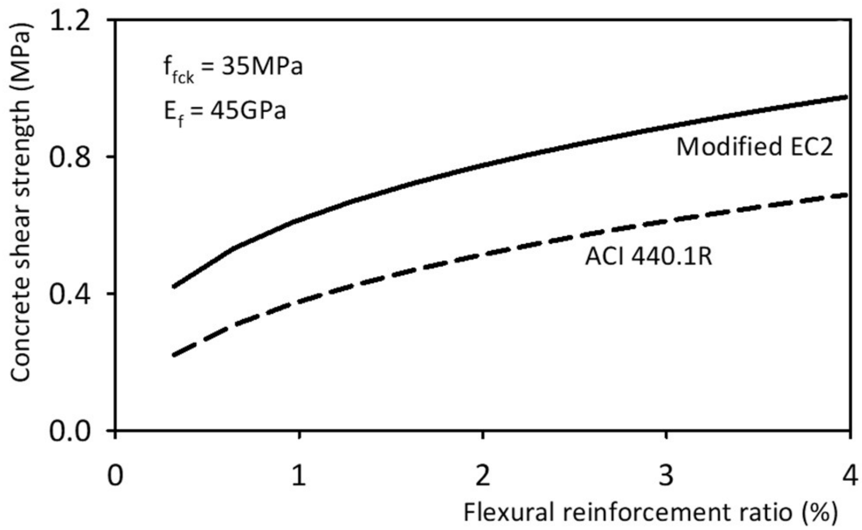
$$V_c = \left(\frac{12}{5} k \right) \frac{1}{6} \sqrt{f_c'} b_w d$$

$$k = \frac{c}{d} = \sqrt{2\rho_f n_f + (\rho_f n_f)^2} - \rho_f n_f$$

$$\rho_f = \frac{A_f}{b_w d} \qquad n_f = \frac{E_f}{E_c}$$



EC2 vs ACI440



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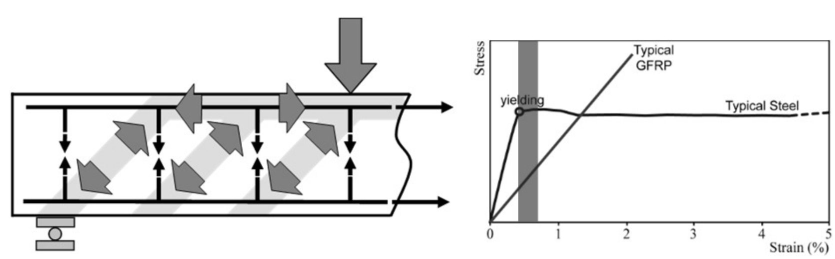
Shear Steel RC code equations



$$V_{Rd,s} = \frac{f_{yw} \cdot A_{sw}}{s} d$$

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FRP shear reinforcement

Possible modification for EC2

$$V_f = \frac{f_{fw} \cdot A_{fw} \cdot z \cdot \cot\theta}{s}$$

$$f_{fw} = 0.004 E_f \le f_{fb}$$

Fixed or Variable angle ?

21.8°<θ<45° ?

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FRP shear reinforcement

ACI 440-1R (2015)

$$V_f = \frac{f_{fw} \cdot A_{fw}}{s} d$$

$$f_{fw} = 0.004 E_f \le f_{fb}$$

$$f_{fb} = \left(0.05 \frac{r_b}{d_b} + 0.3\right) f_{fu} \le f_{fu}$$

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fib CEB-FIP

Shear design - Summary

- The lower stiffness of the FRP reinforcement affects shear resistance
- Similar design approach can be used if strains are controlled
- Strength of shear reinforcement may be affected by its geometry

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Detailing of FRP reinforcement

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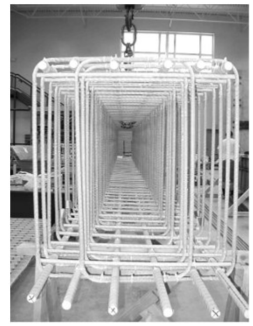
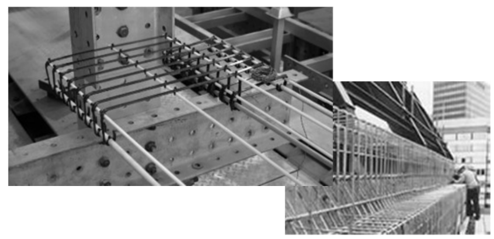
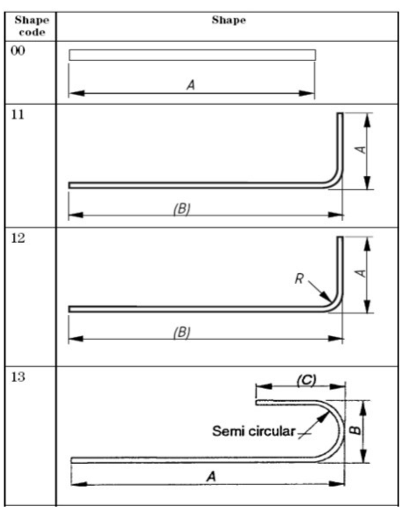
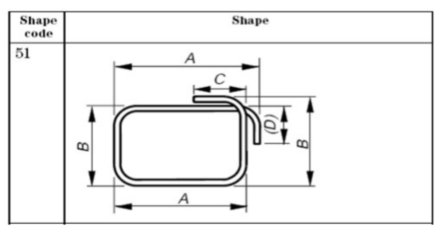
Bending reinforcement

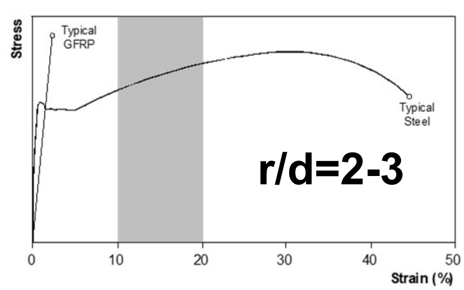
- End anchorages
- Element connection
- Shear reinforcement

Bending reinforcement



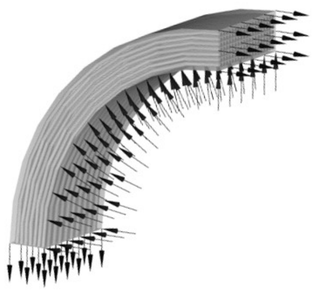


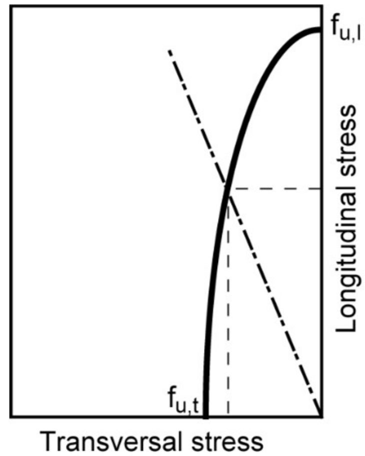


r/d=2-3

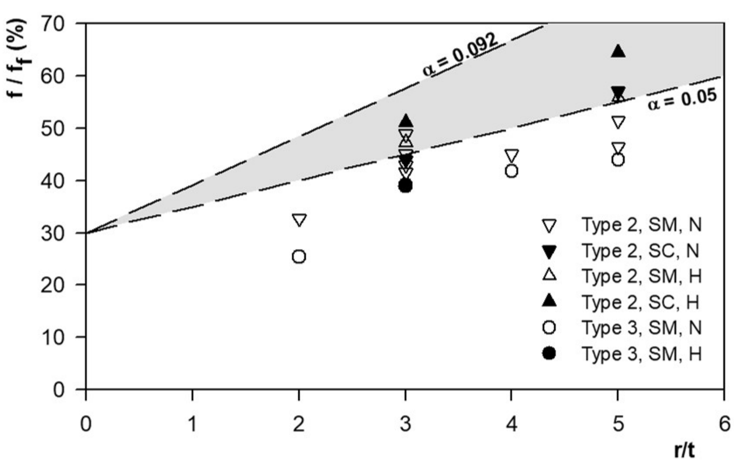
Strength of bent FRP bars

The tensile strength of FRP bars is largely reduced when subjected to a biaxial state of stress

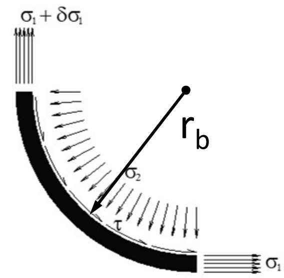




Strength of bent FRP bars



Strength of bent FRP bars



$$f_{fb} = \left(0.05\frac{r_b}{d_b} + 0.3\right) f_{fu} \le f_{fu}$$

$r_b/d_b \geq 3$

r_b is the radius of the bend in the bar
d_b is the nominal diameter of the FRP bar
f_{fu} is the design strength of a straight portion of the bar

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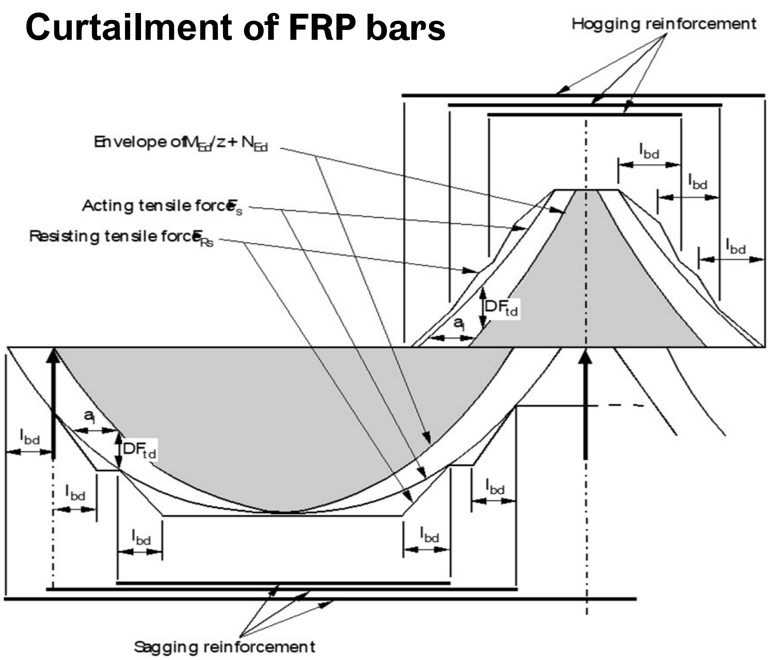
Curtailment of FRP bars



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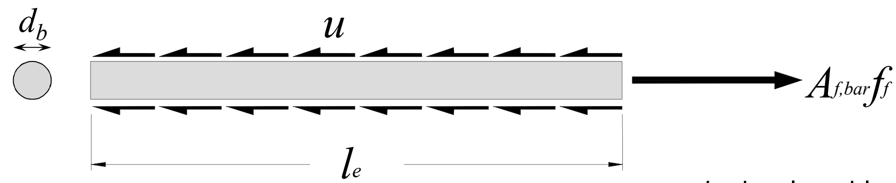
Development length of straight FRP bars (ACI 440.1R-15)



$$l_e \pi d_b u = A_{f,bar} f_f$$

l_e=developed length
u=average bond stress
d_b=bar diameter

$$\frac{u}{0.083\sqrt{f_c'}} = 4.0 + 0.3\frac{C}{d_b} + 100\frac{d_b}{l_e}$$

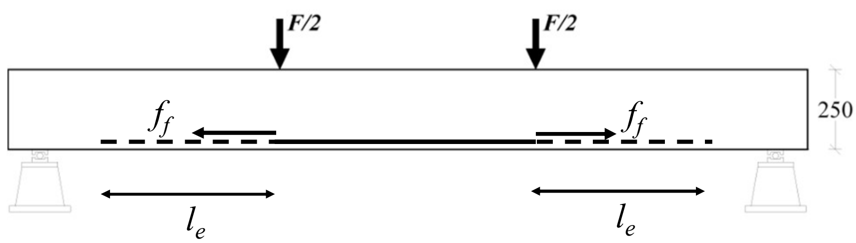
(SI units)

Based on Orangun's (1977) equation

$$f_{fe} = \frac{0.083\sqrt{f_c'}}{\alpha}\left(13.6\frac{l_e}{d_b} + \frac{C}{d_b}\frac{l_e}{d_b} + 340\right) \le f_{fu}$$

α=1.0 for "bottom cast bars", or 1.5 for "top cast bars"

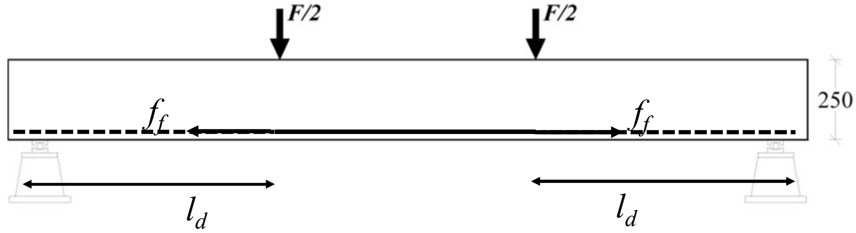
Development length of straight FRP bars (ACI 440.1R-15)



$$f_{fe} = \frac{0.083\sqrt{f_c'}}{\alpha}\left(13.6\frac{l_e}{d_b} + \frac{C}{d_b}\frac{l_e}{d_b} + 340\right) \le f_{fu}$$

C = the lesser of the cover to the center of the bar or one-half of the center-on-center spacing of the bars being developed

Development length of straight FRP bars (ACI 440.1R-15)



$$l_d = \frac{\alpha \dfrac{f_f}{0.083\sqrt{f_c'}} - 340}{13.6 + \dfrac{C}{d_b}} d_b$$

C = the lesser of the cover to the center of the bar or one-half of the center-on-center spacing of the bars being developed

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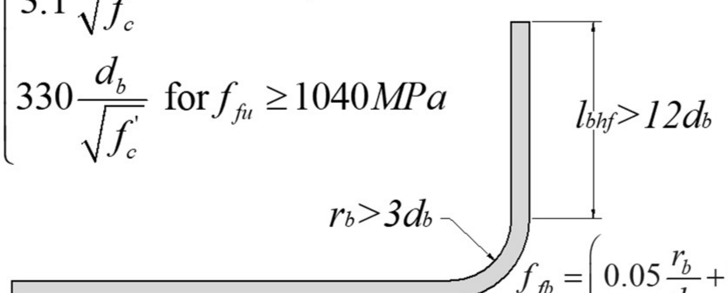
Development length of bent FRP bars (ACI 440.1R-15)

$$l_{bhf} = \begin{cases} 165\dfrac{d_b}{\sqrt{f_c'}} & \text{for } f_{fu} \leq 520MPa \\ \dfrac{f_{fu}}{3.1}\dfrac{d_b}{\sqrt{f_c'}} & \text{for } 520 < f_{fu} < 1040MPa \\ 330\dfrac{d_b}{\sqrt{f_c'}} & \text{for } f_{fu} \geq 1040MPa \end{cases}$$

$$l_{bhf} = K_4 \frac{d_b}{\sqrt{f_c'}}$$



$$f_{fb} = \left(0.05\frac{r_b}{d_b} + 0.3\right) f_{fu} \leq f_{fu}$$

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Concluding Remarks

- **Use of FRP reinforcement is effective when corrosion is an issue or when specific properties are required** (e.g. electromagnetic neutrality)
- **Same fundamental principles can be used for design of steel/FRP RC elements** (with some modifications)
- **Different philosophy should be adopted to address the issue of safety** (brittle failure modes)

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References

***fib* Bulletin 40**
Technical report on ***"FRP reinforcement in RC structures"*** drafted by a working party of Task Group 9.3, FRP (Fibre Reinforced Polymer) reinforcement for concrete structures, in Commission 9, Reinforcing and prestressing materials and systems, 2007, 160 pp, ISBN 978-2-88394-080-2.

***fib* Model Code for Concrete Structures 2010**
October 2013, ISBN 978-3-433-03061-5

ACI 440.1R-15
American Concrete Institute (2015). Guide for the Design and Construction of Structural Concrete Reinforced with Fiber-Reinforced Polymer (FRP) Bars. ACI 440 Committee.

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