AASHTO GFRP-Reinforced Concrete Design Training Course

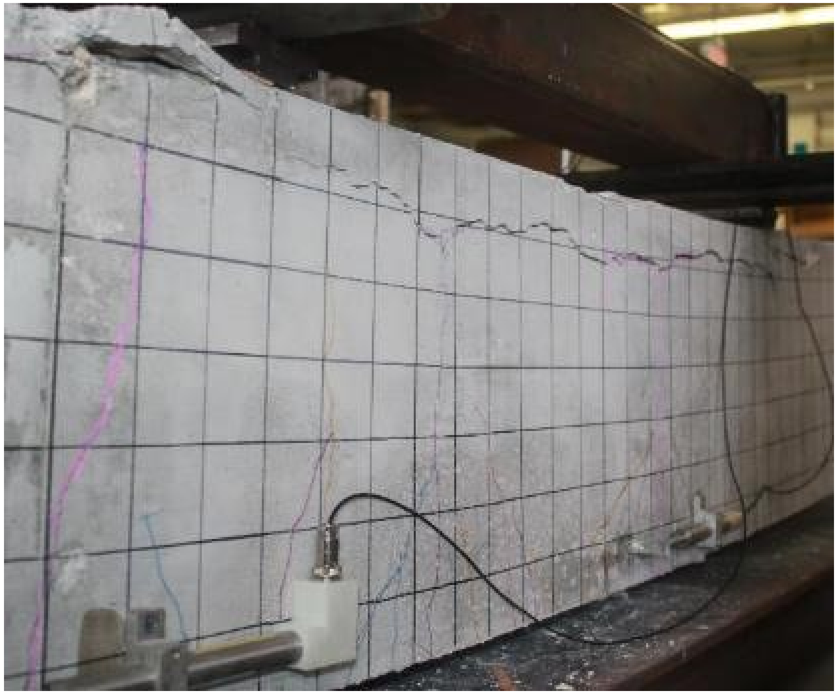
FDOT

UNIVERSITY OF MIAMI

Center for Integration of Composites into Infrastructure

Course Outline

1. Introduction & Materials
2. **Flexure Response**
3. Shear Response
4. Axial Response
5. Case Studies & Field Operations



2. FLEXURE RESPONSE OF GFRP REINFORCED CONCRETE

FDOT

UNIVERSITY OF MIAMI

Center for Integration of Composites into Infrastructure

Table of Contents

FDOT | UNIVERSITY OF MIAMI | Center for Integration of Composites into Infrastructure

Flexural Theory

Assumptions:

1. Plane sections remain plane after deformation
2. Ultimate concrete strain is 0.003
3. Tensile strength of concrete and FRP compressive strength are neglected
4. FRP is perfectly bonded to concrete
5. Stress-strain of FRP is linear until failure

Ultimate Flexural Strength and Demand:

M_n = nominal capacity

M_u = factored demand

$$\boxed{\phi M_n \geq M_u}$$

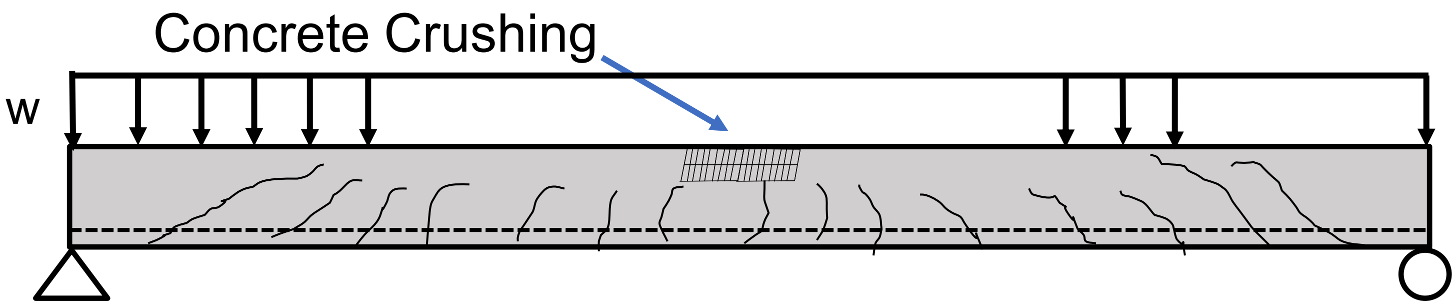
ϕ = strength reduction factor (depends on the mode of failure)

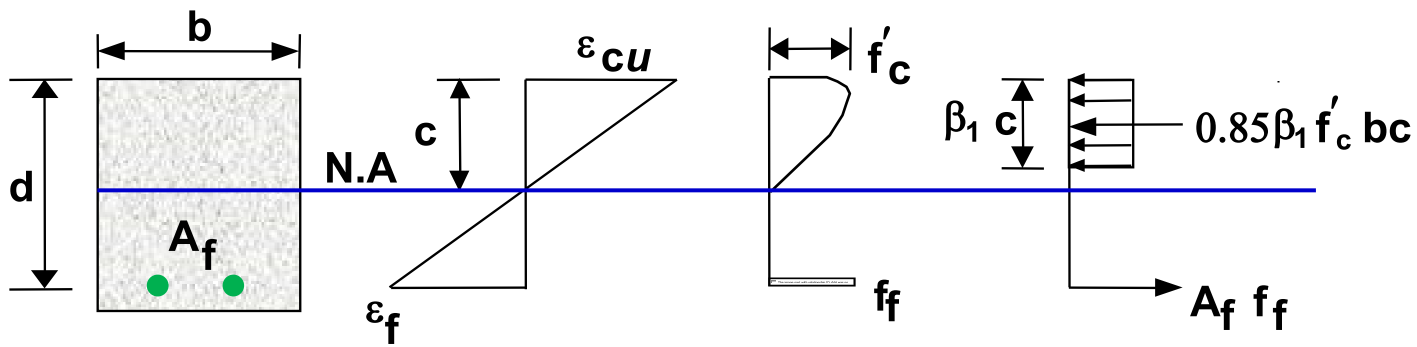
$$M_u = \underbrace{1.25 M_{DC} + 1.75 M_{LL}}_{\text{For } \textbf{\textit{AASHTO,}} \text{ example}}$$

$$M_u = \underbrace{1.2 M_D + 1.6 M_L}_{\text{For ACI, example}}$$

Failure Modes

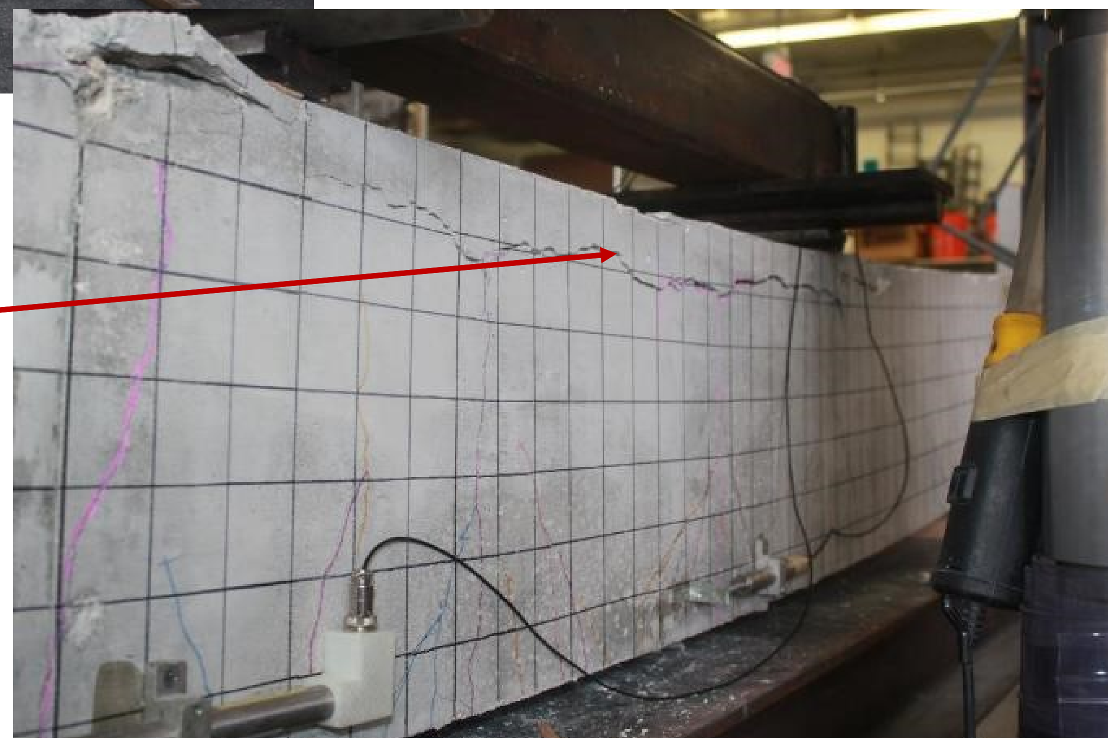
Compression-controlled: concrete crushing





$\beta_1 c = a$

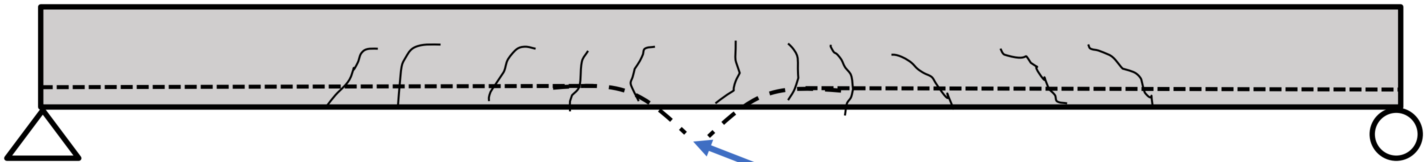
Failure Modes

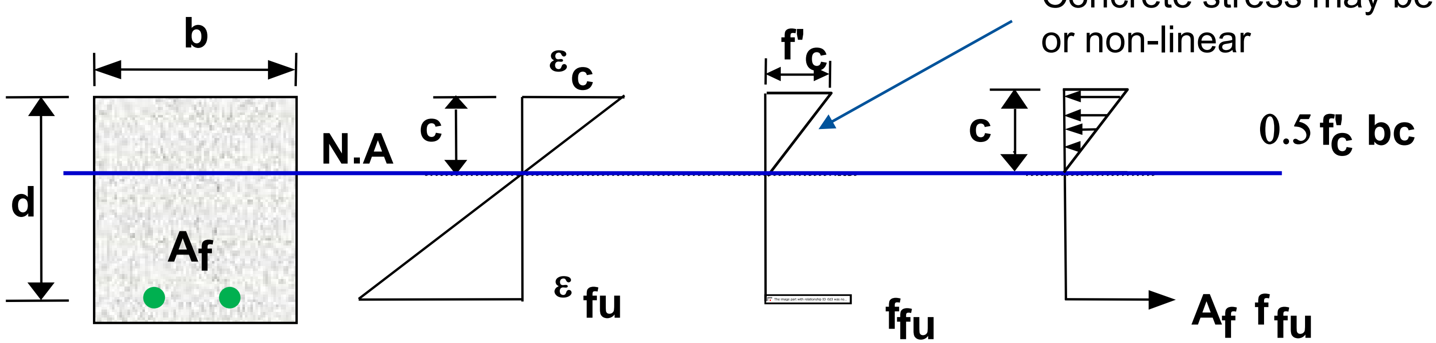


Concrete Crushing Failure in GFRP-RC Beam

Failure Modes

Tension-controlled: FRP rupture



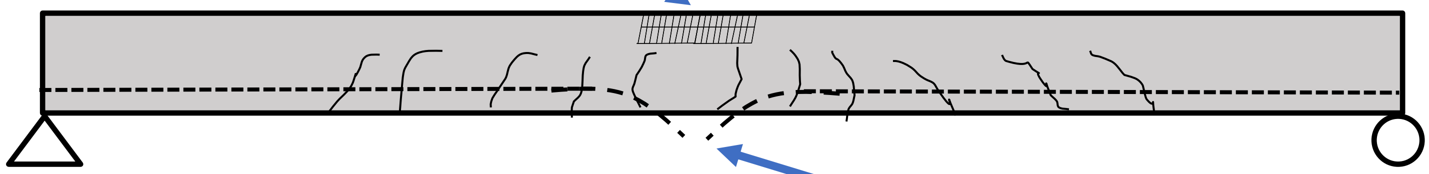


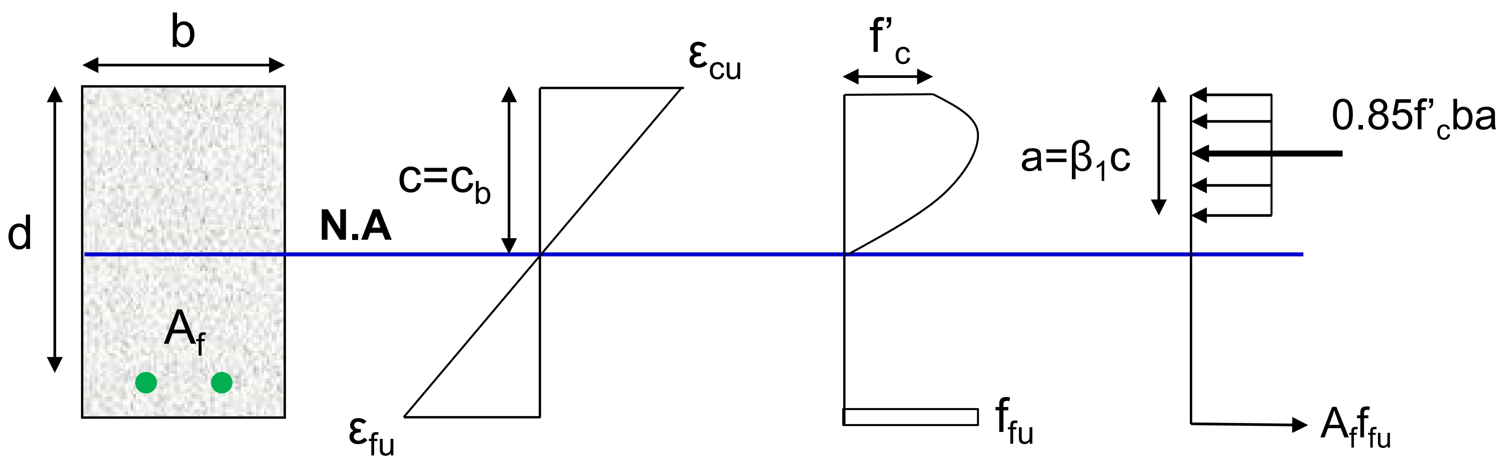
Failure Modes

FRP Rupture Failure in GFRP Reinforced Beam

Failure Modes

Balanced failure: simultaneous concrete crushing & FRP rupture

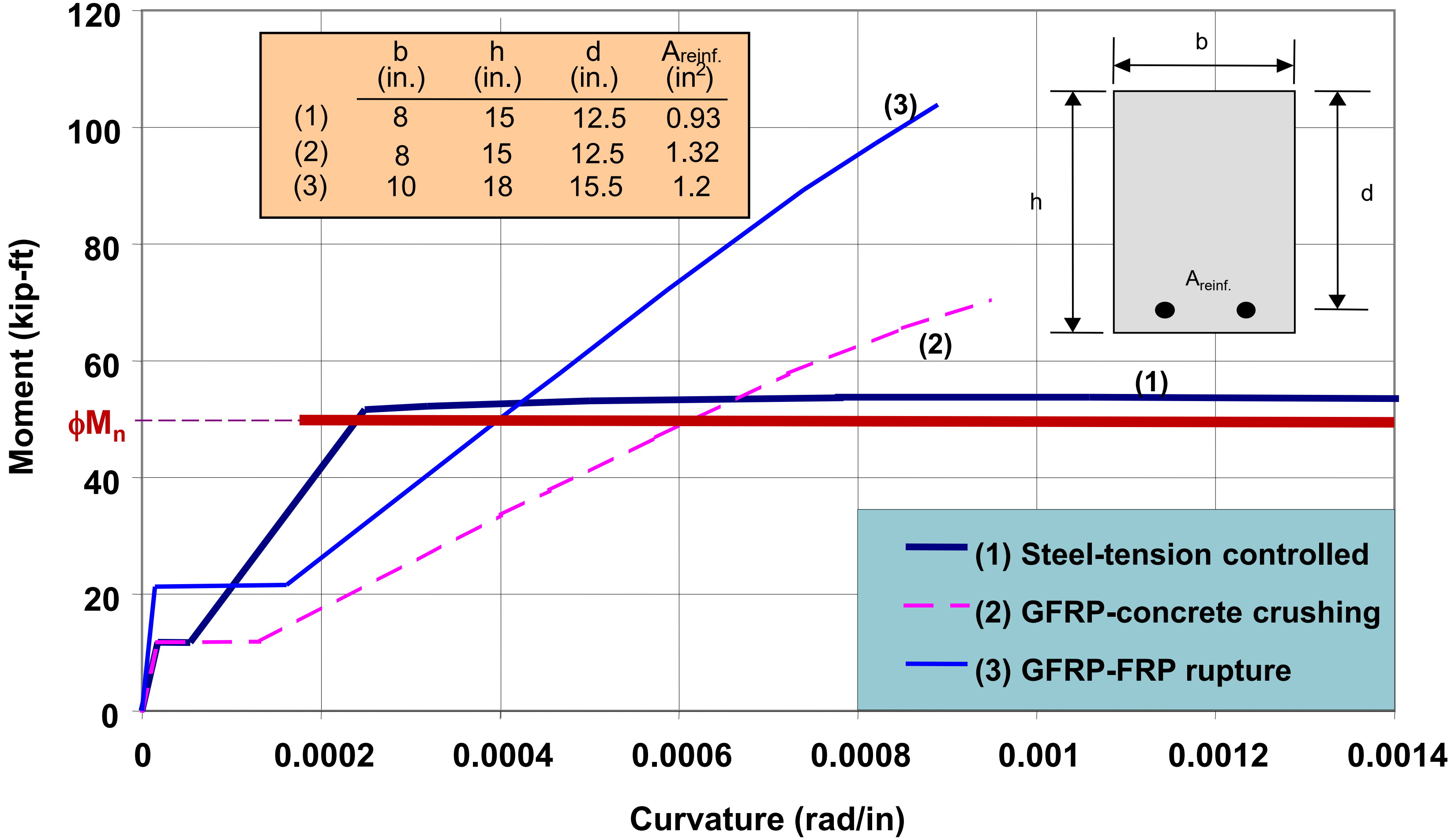




General Considerations

- Flexural capacity of an FRP-reinforced flexural member dependent on **tension or compression failure modes**
- Over or under-reinforced sections are acceptable provided that the strength and serviceability criteria are satisfied
- FRP reinforcement is brittle, but provides warning in terms of large member deflection
- Flexural behavior is not ductile; **therefore safety factors are increased** (i.e., smaller Φ factors)

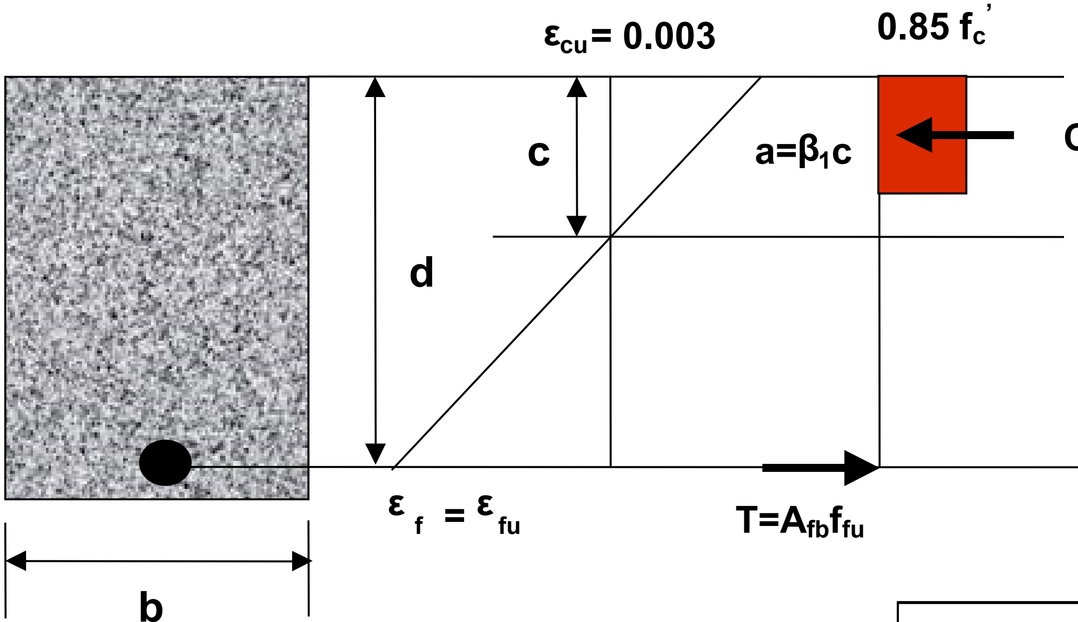
Moment-Curvature Diagrams



	b (in.)	h (in.)	d (in.)	$A_{reinf.}$ (in^2)
(1)	8	15	12.5	0.93
(2)	8	15	12.5	1.32
(3)	10	18	15.5	1.2

FDOT UNIVERSITY OF MIAMI Center for Integration of Composites into Infrastructure

Balanced Failure



$$\rho_f = \frac{A_{fb}}{bd}$$

(AASHTO 2.5.3)

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

Where it can be shown that:

$$C = 0.85 f_c' \beta_1 \frac{\varepsilon_{cu}}{\varepsilon_{cu} + \varepsilon_{fu}} bd$$

$$T = A_{fb} f_{fu} \quad = \rho_{fb} f_{fu} bd$$

Balanced Reinforcement Ratio

IF $\rho_f < \rho_{fb}$

Rupture of FRP will control failure

IF $\rho_f > \rho_{fb}$

Concrete crushing will control failure

TYPICAL VALUES FOR ρ_b , ρ_{fb}

STEEL GRADE 60	$\rho_b = 0.0335$
GFRP F80-E6	$\rho_{fb} = 0.0078$

Table of Contents

- General Considerations
- **Bending Moment Capacity**
- Serviceability
- Static & Cyclic Fatigue
- Anchorage and Development
- Special Considerations
- Concluding Remarks

Nominal Flexural Strength: Compression

Case of **concrete crushing controlling** failure and stress distribution approximated by rectangular stress block

IF $\varepsilon_{ft} < \varepsilon_{fu}$

$$M_n = A_f f_f \left(d - \frac{a}{2}\right)$$ **(AASHTO 2.6.3.2.2-1)**

WHERE

$$a = \frac{A_f f_f}{0.85 f_c' b}$$ **(AASHTO 2.6.3.2.2-2)**

$$f_f = \sqrt{\frac{\left(E_f \varepsilon_{cu}\right)^2}{4} + \frac{0.85 \beta_1 f_c'}{\rho_f} E_f \varepsilon_{cu}} - 0.5 E_f \varepsilon_{cu} \leq f_{fu}$$ **(AASHTO 2.6.3.1-1)**

Nominal Flexural Strength: Tension

FRP rupture controlling failure. Whitney's Stress block not applicable because $\varepsilon_c < \varepsilon_{cu} = 0.003$. Simplified and conservative procedure is proposed (i.e., $c = c_b$):

IF $\boldsymbol{\varepsilon_{ft} = \varepsilon_{fu}}$

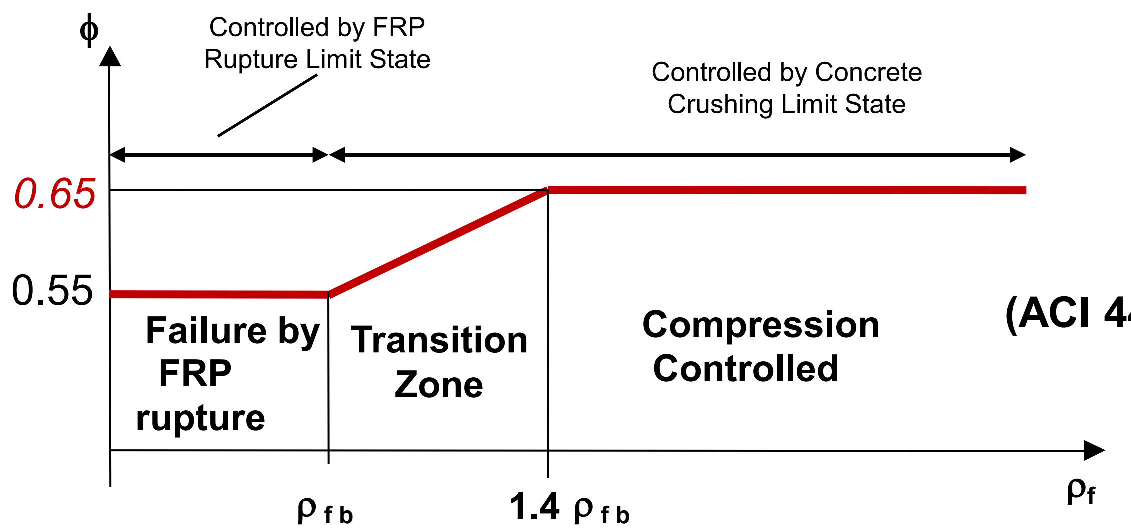
$$M_n = A_f f_{fu}\left(d - \frac{\beta_1 c_b}{2}\right) \qquad \textbf{(AASHTO 2.6.3.2.2.-3)}$$

WHERE

$$c_b = \left(\frac{\varepsilon_{cu}}{\varepsilon_{cu} + \varepsilon_{fu}}\right) d \qquad \textbf{(AASHTO 2.6.3.2.2.-4)}$$

Strength Reduction Factor (ACI)

Φ as a function of GFRP reinforcement ratio

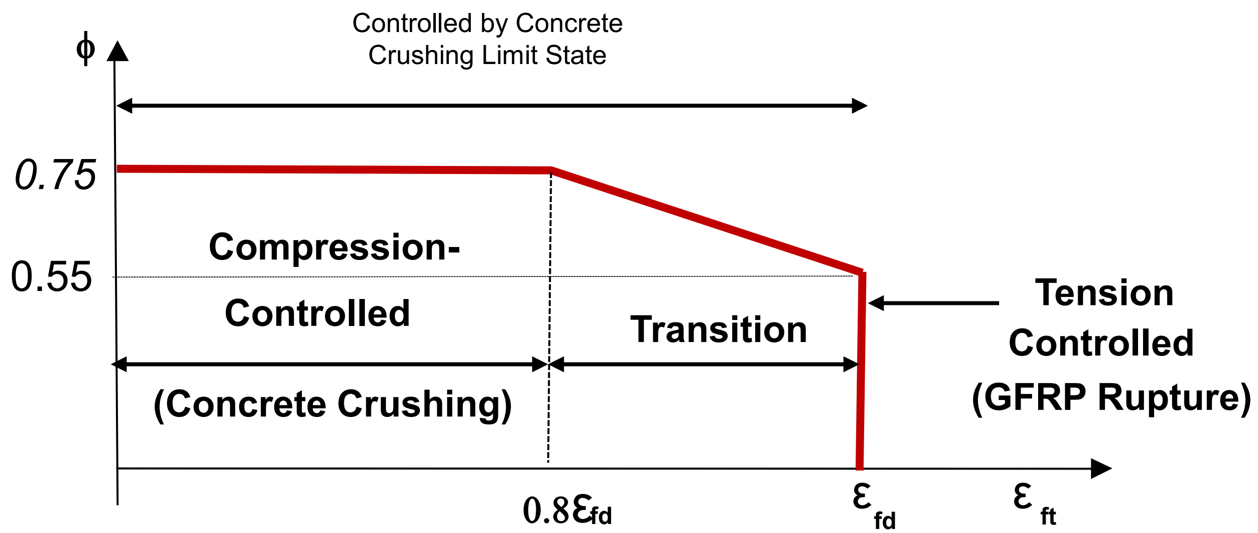


(ACI 440 7.2.3)

$$\phi = \begin{cases} 0.55 \ for \ \rho_f \leq \rho_{fb} \\ 0.3 + 0.25 \dfrac{\rho_f}{\rho_{fb}} for \ \rho_{fb} < \rho_f < 1.4\rho_{fb} \\ 0.65 \ for \ \rho_f \geq 1.4\rho_{fb} \end{cases}$$

(ACI 440 7.2.3)

Strength Reduction Factors (AASHTO)



Φ as a function of strain in GFRP

$$\phi = \begin{cases} 0.55 & for\ \varepsilon_{ft} \le \varepsilon_{fd} \\ 1.55 - \dfrac{\varepsilon_{ft}}{\varepsilon_{fd}} & for \quad 0.80\varepsilon_{fd} < \varepsilon_{ft} < \varepsilon_{fd} \\ 0.75 & for\ \ \varepsilon_{ft} \ge 0.80\varepsilon_{fd} \end{cases}$$

(AASHTO 2.5.5.2)

Minimum Flexural Reinforcement

Minimum longitudinal reinforcement to provide adequate level of protection against **sudden** failure at formation of first flexural crack

$$M_r \geq \begin{cases} 1.33 M_u \\ 1.6 M_{cr} - M_{dnc}\left(\dfrac{S_c}{S_{nc}} - 1\right) \end{cases}$$

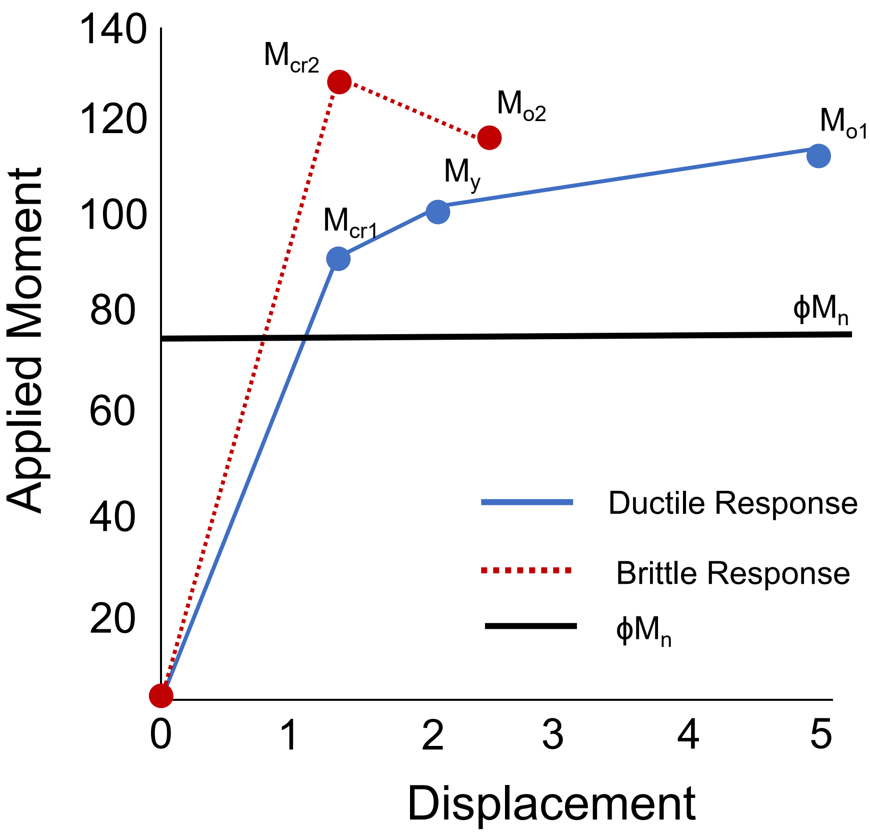
f_r = modulus of rupture of concrete

M_{dnc} = total unfactored dead load moment

S_c = section modulus of the extreme fiber of the composite section

S_{nc} = section modulus for the extreme fiber of the monolithic or non-composite section

(AASHTO 2.6.3.3-1)



Illustrative example on ductile and brittle responses

Table of Contents

FDOT UNIVERSITY OF MIAMI Center for Integration of Composites into Infrastructure

Serviceability

- Stresses under sustained and cyclic loading must be checked to avoid **static (creep-rupture)** and **cyclic fatigue** rupture

- The substitution of FRP for steel on an **equal area basis** would typically result in **larger deflections** and **wider crack widths**

- Deflections under service loads and crack control often **govern design**
  - **Cracking** – Excessive crack width is undesirable for aesthetic and other reasons (e.g., prevent leakage that can damage structural concrete)
  - **Deflection** – Deflections should be within acceptable limits imposed by the use of the structure (e.g., supporting attached nonstructural elements without damage)

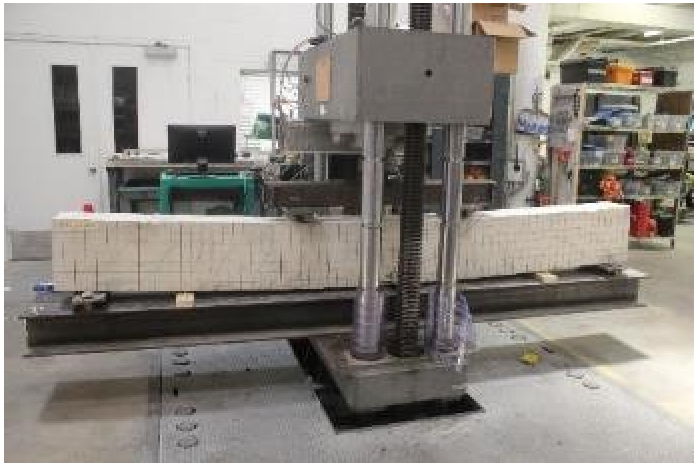
- Designing FRP-RC beams for **concrete crushing** typically satisfies serviceability criteria

Crack Control: Bond Coefficient

Bond coefficient (k_b) accounts for the degree of bond between FRP bar and surrounding concrete in ACI 440. **Bond reduction factor** (C_b) defined as the inverse of k_b in AASHTO. Function of **surface configuration** and materials varies from 70 to 120% of steel bars. **83%** assumed in AASHTO 2.6.7.

$$\boldsymbol{k_b} = \begin{cases} > 1 & \textit{Worse than steel} \\ < 1 & \textit{Better than steel} \end{cases}$$

$$\boldsymbol{C_b = \frac{1}{k_b}} = \begin{cases} > 1 & \textit{Better than steel} \\ < 1 & \textit{Worse than steel} \end{cases}$$



Testing of GFRP-RC beam using four-point setup

Control of Crack Width

FRP bars are corrosion-resistant; therefore, **larger crack widths** as compared to steel-RC concrete can be tolerated

The maximum crack width (*w*) is: **GFRP: 0.028 in**

Steel: 0.017 in

Indirect approach controls flexural cracking in terms of maximum bar spacing adopted in AASHTO GFRP 2nd Ed.:

$$s_{max} \leq \min\left(1.15\frac{C_b E_f w}{f_{fs}} - 2.5c_c, 0.92\frac{C_b E_f w}{f_{fs}}\right)$$

(AASHTO 2.6.7-1)

E_f = tensile modulus of elasticity of GFRP reinforcement (ksi)

f_{fs} = calculated tensile stress in GFRP reinforcement at the service limit state (ksi)

c_c = clear cover, not greater than 2 in. plus half the bar diameter (in.)

Control of Crack Width

For calculated stress level and crack width limit, d_c (concrete cover) shall satisfy:

$$\boxed{d_c \leq \frac{C_b E_f w}{2 f_{fs} \xi}}$$

(AASHTO 2.6.7-2)

d_c = thickness of concrete cover measured from extreme tension fiber to center of flexural GFRP reinforcement located closest thereto (in.)

E_f = tensile modulus of elasticity of GFRP reinforcement (ksi)

f_{fs} = calculated tensile stress in GFRP reinforcement at the service limit state (ksi)

C_b = reduction factor that accounts for the degree of bond between GFRP reinforcing bars and surrounding concrete

w = maximum crack width in a concrete component (in.)

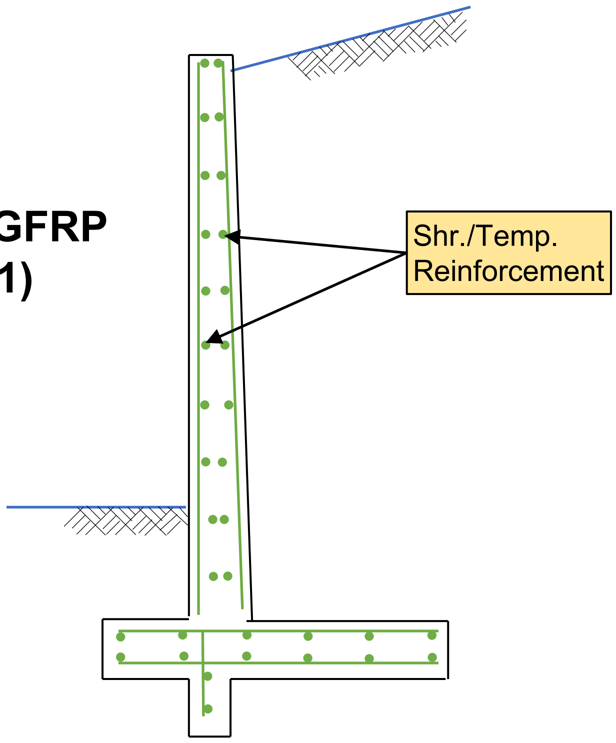
ξ = ratio of distance from neutral axis to extreme tension fiber, $(h - kd)$, to distance from neutral axis to center of tensile reinforcement, $(d - kd)$.

Shrinkage & Temperature Reinforcement

Area of shrinkage and temperature reinforcement may be divided between each face and shall be:

$$\rho_{f,st} = \max\left(\frac{3{,}132}{E_f f_{fd}}; 0.0014\right) \leq 0.0036$$

AASHTO GFRP (2.9.6-1)



Adapted from FDOT Index 400-010

Spacing:

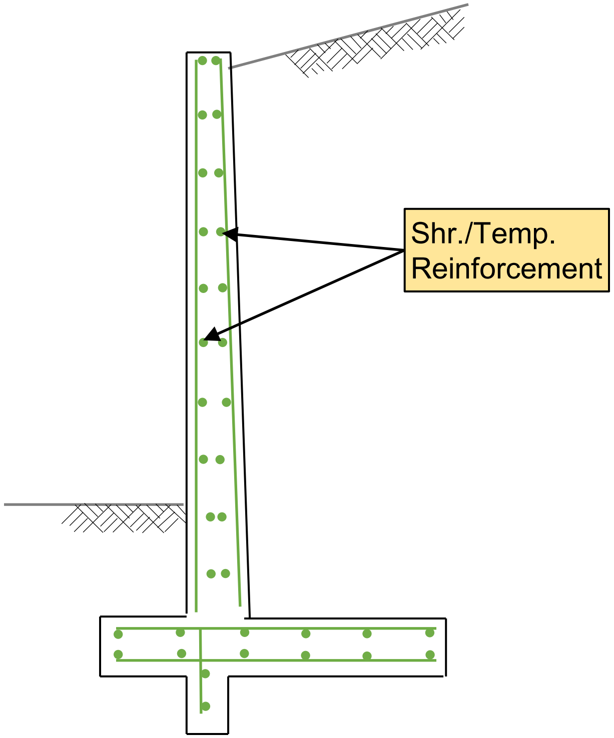
- $\leq 3t_{slab}$ or 12 in.
- Evenly distributed on both surfaces if member is greater than 6 in.

Shrinkage & Temperature Reinforcement

GFRP-RC retaining wall example:

Properties

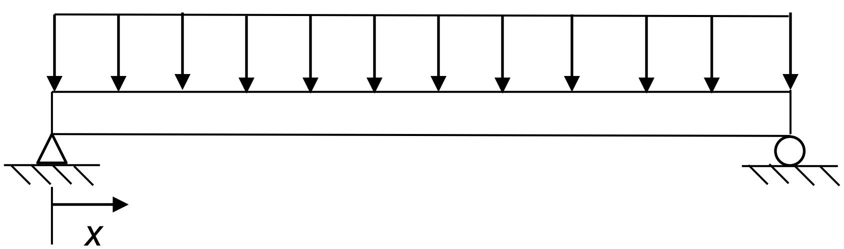
Width of Wall Average:	13.4 in.
Height of Wall:	18 ft.
Bar Size of Temp. Reinf.	#4
Elastic Modulus of GFRP, E_f	6,500 ksi, 8,700 ksi
Design Tensile Strength, f_{fd}	75.6 ksi, 105 ksi

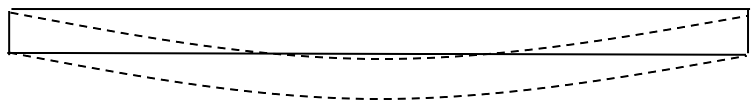


Minimum Area of S/T Reinf.		
A_s	=	0.29 in^2/ft
$A_{f,6500\ ksi}$	=	0.58 in^2/ft
$A_{f,8700\ ksi}$	=	0.55 in^2/ft

Deflections

- AASHTO does not allow deflection control by **indirect method** (e.g., specifying minimum thickness of a member)
- **Direct method** of limiting computed deflections:
  - ✓ Simplified: Effective moment of inertia, I_e
  - ✓ Direct integration of moment curvature relationship





Curvature

$$\phi = \frac{\delta^2 y}{\delta x^2} = \frac{M}{EI}$$

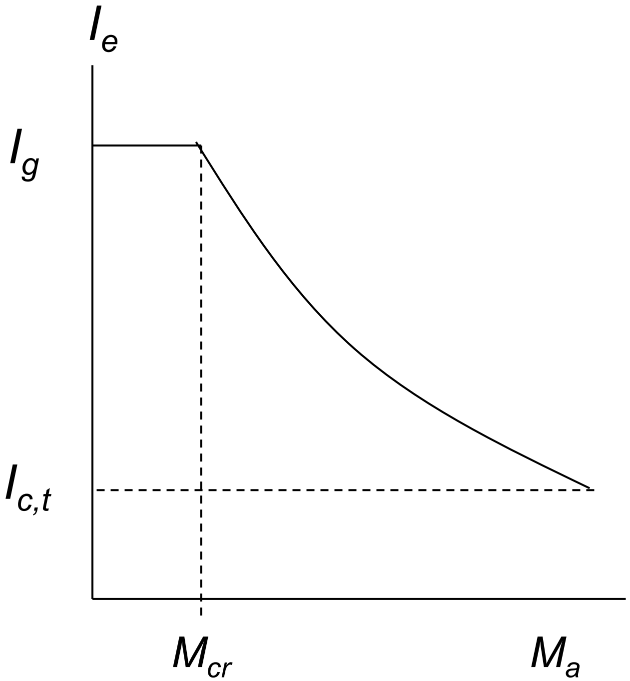
Deflection

$$y = \iint \frac{M}{EI} dx = \frac{kwl^3}{EI}$$

Deflections: Effective Inertia

Short-Term Loading

The overall (equivalent) flexural stiffness, $\boldsymbol{E_c I_e}$, of a member that has experienced cracking at service varies between $\boldsymbol{E_c I_g}$ an $\boldsymbol{E_c I_{cr}}$



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

AASHTO (2.6.3.2-1)

$$M_{cr} = f_r \frac{I_g}{y_t}$$

AASHTO (2.6.3.4.2-2 & 2.6.3.4.2-3)

$$\gamma_d = 1.72 - 0.72 \frac{M_{cr}}{M_a}$$

M_a = maximum moment in a member at the stage deflection is computed, *kip-in*

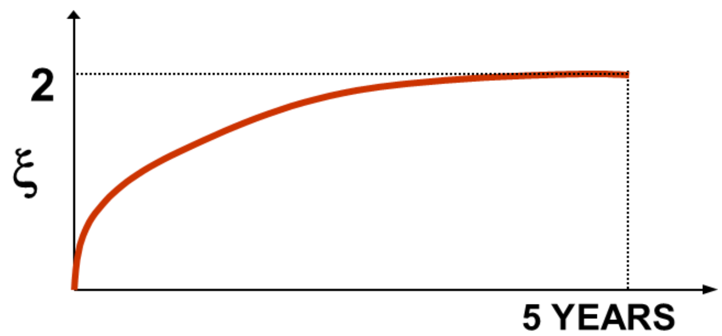
Long-Term Deflection

The long-term deflection can be calculated from:

$$\Delta_{(cp+sh)} = \xi(\Delta_i)_{sus}$$

$(\Delta_i)_{sus}$ = short term deflection due to sustained load (*DL + 0.2LL*)

ξ = time dependent factor for sustained load, $2n_f$



Unless a more exact determination is made, long-term deflection may be:

- If instantaneous deflection is based on I_g: $4.0(\Delta_i)_{sus}$
- If instantaneous deflection is based on I_e: $3.0(\Delta_i)_{sus}$

AASHTO GFRP (2.6.3.4.2)

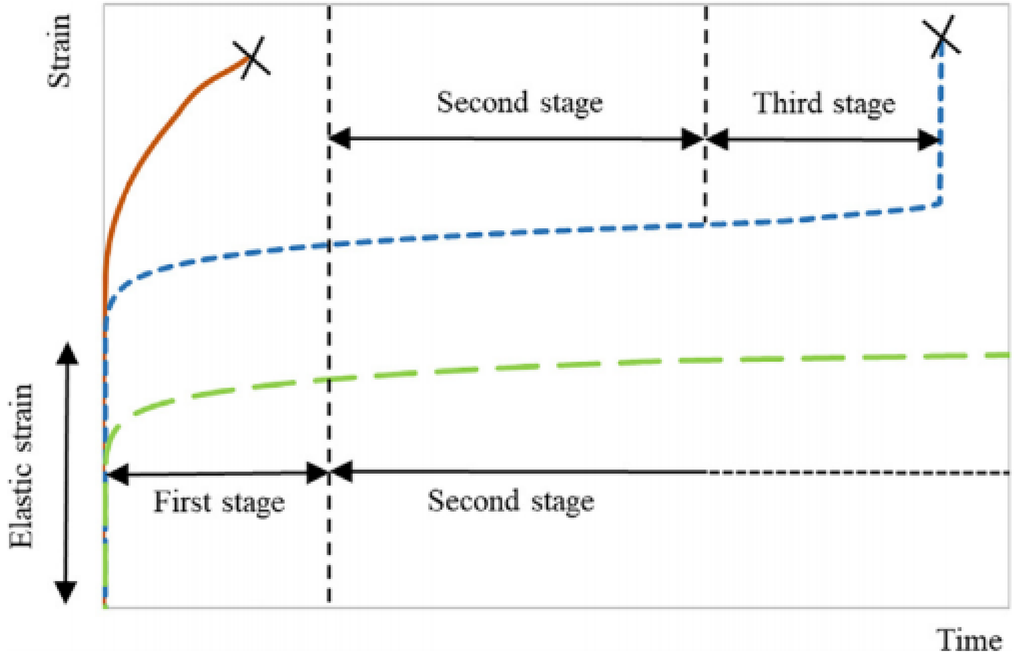
Table of Contents

Static Fatigue (Creep Rupture)

FRP reinforcing bars subjected to sustained load can suddenly fail after a time period called endurance time. This phenomenon is known as creep rupture (or static fatigue)

Sustained Load

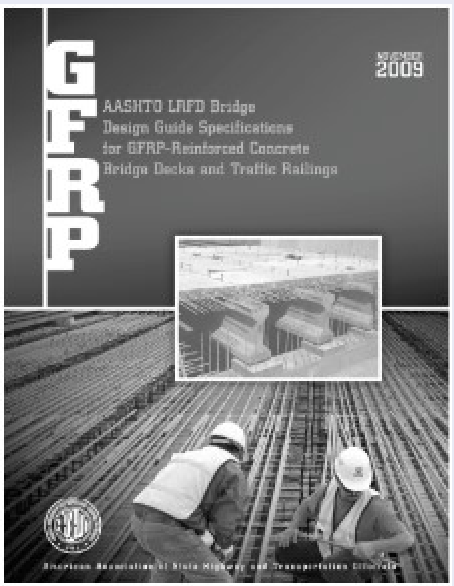
Sustained Load

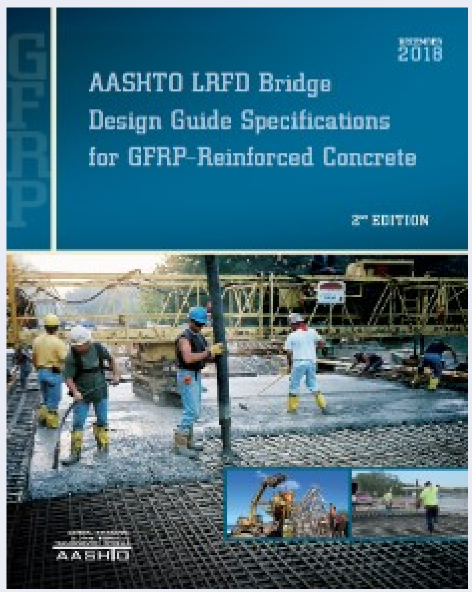


Creep stages in an FRP unidirectional composite

Creep Rupture Reduction Factor

AASHTO GFRP-1	AASHTO GFRP-2
2009	**2018**
Design of bridges	**Design of bridges**
C_C = 0.20	***C_C = 0.30***





$$f_{f,s} = C_E C_C f^*_{fu}$$

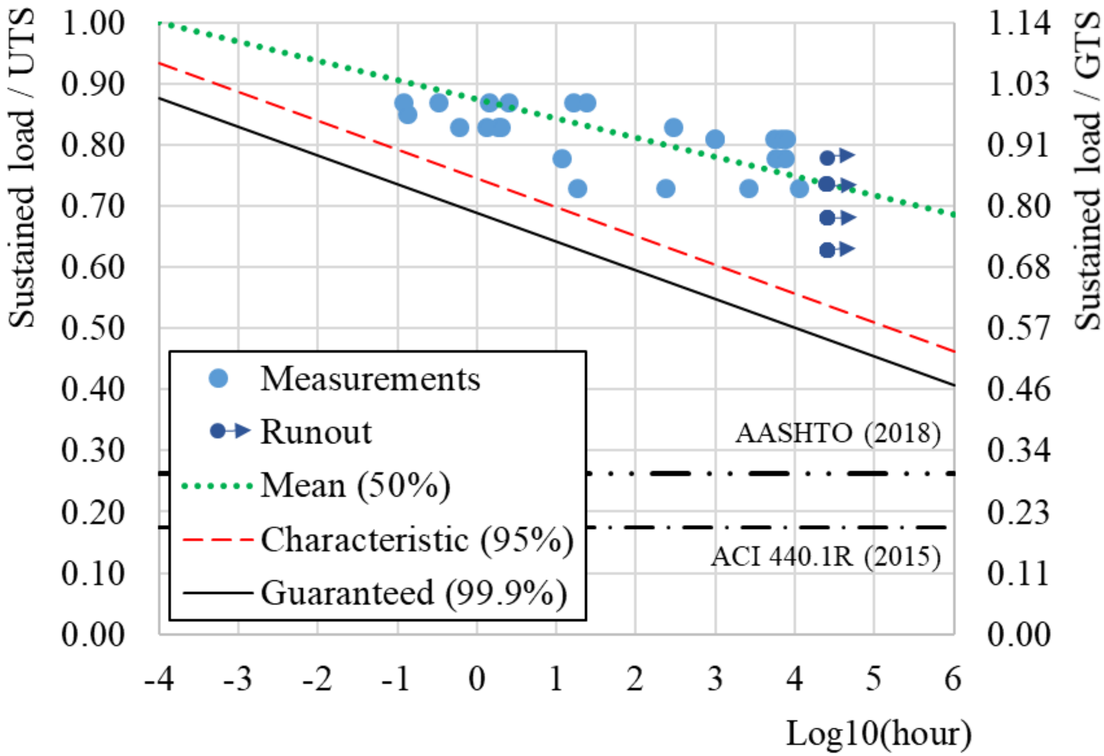
Creep rupture may govern the design of bridge

Deck of the Halls River Bridge in Homosassa (FL)

State-of-the-art in Creep Rupture

Research shows how current limits are conservative and may be raised as technology advances

Sustained Load



Sustained load versus logarithmic time-to-failure

→ +50%

From 0.20 to 0.46 using guaranteed unconditioned line

Creep Rupture Provision

Maximum sustained tensile stress in GFRP reinforcement, f_{fs}, calculated using **dead loads and live loads** included in **Service I** load combination with live load reduced from **1.0** to **0.2**

$$f_{fs} \leq C_c f_{fd}$$ maximum sustained tensile stress in GFRP reinforcement, ksi **(AASHTO 2.5.3-1)**

where

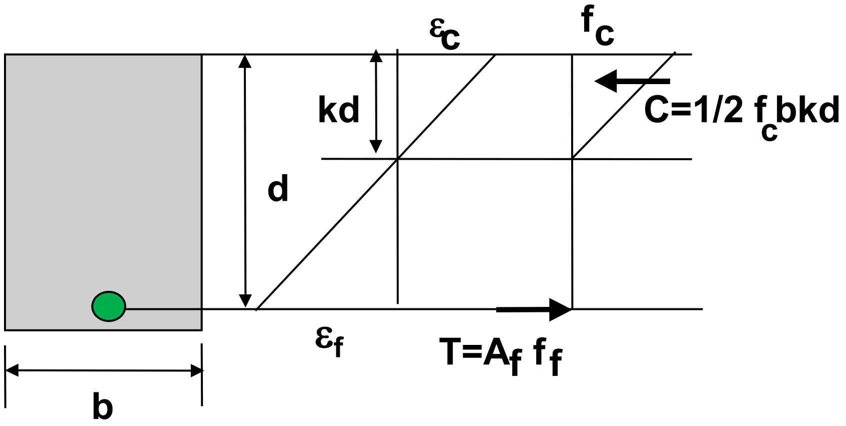
$$f_{fs} = \frac{n_f d(1-k)}{I_{cr}} M_{s,s}$$ **(AASHTO 2.5.3-2)**

Creep rupture reduction factor, **C_c, shall be equal to 0.3** unless manufacturer can provide a research report following ASTM D7337

Creep Rupture Stress

Based on elastic analysis and the sustained moment, $M_{s,s}$,

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



Where:

$f_{f,s}$ = stress level induced in the FRP by sustained loads, psi **(AASHTO 2.5.3-1)**

$M_{s,s}$ = the moment due to all sustained loads

$n_f = \frac{E_f}{E_c}$ modular ratio

$$k = \sqrt{2\rho_f n_f + \left(\rho_f n_f\right)^2} - \rho_f n_f \qquad I_{cr} = \frac{bd^3}{3}k^3 + n_f A_f d^2 (1-k)^2$$

Cyclic Fatigue

The maximum tensile stress in the GFRP reinforcement, $f_{f,f}$, shall satisfy:

$$f_{f,f} \leq C_f C_E f^*_{fu} = C_f f_{fd}$$

where:

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

(**AASHTO 2.5.4**)

C_f = fatigue rupture reduction factor **(set at 0.25 pending future research)**

f_{fd} = design tensile strength of GFRP reinforcing bars (Eq. 2.4.2.1-1) (ksi)

n_f = modular ratio (E_f/E_c)

d = distance from the extreme compression fiber to centroid of tensile bar (in.)

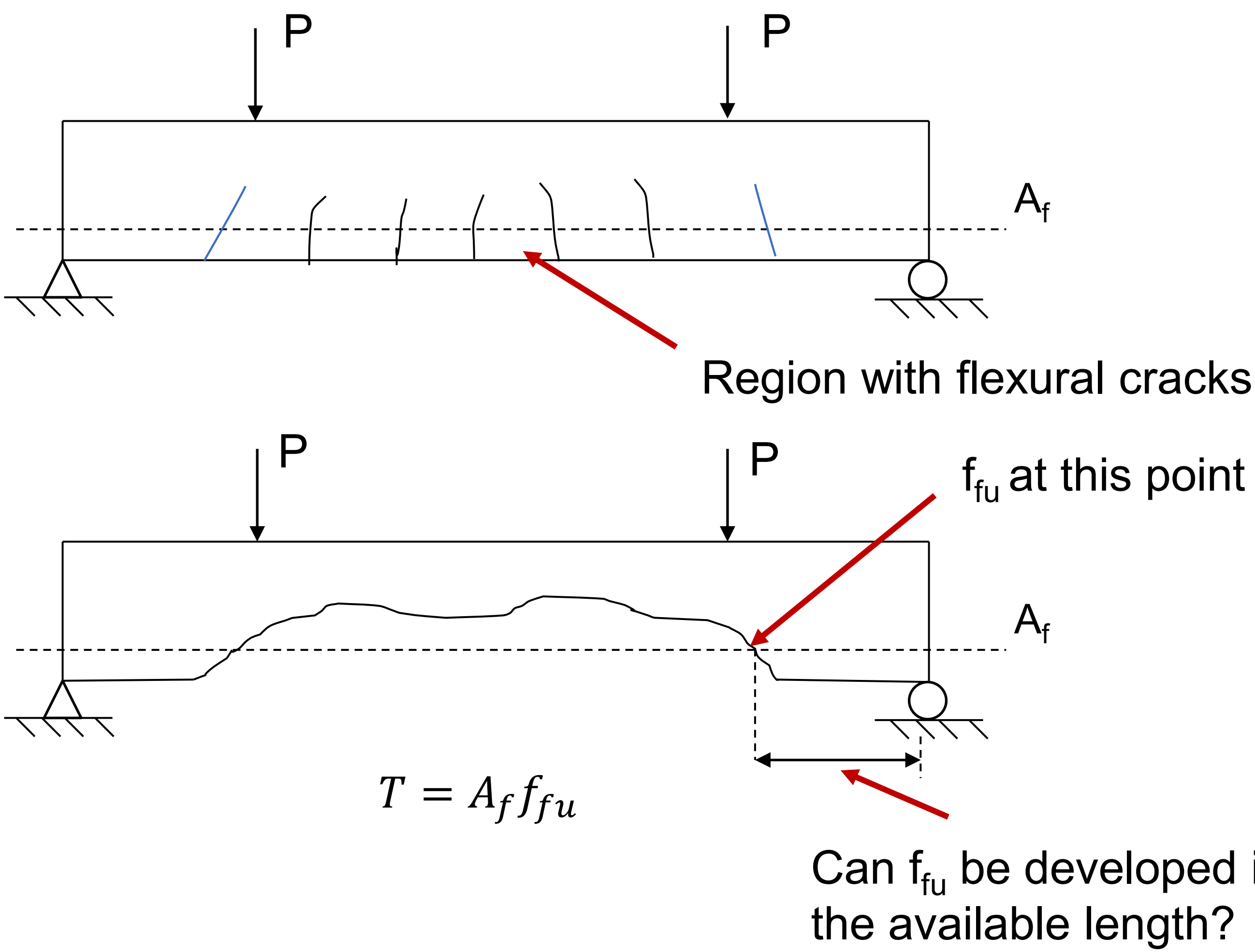
k = ratio of depth of neutral axis to reinforcement depth

I_{cr} = moment of inertia of transformed cracked section (in^4)

$M_{s,f}$ = moment due to dead loads + fatigue load

Table of Contents

Anchorage Introduction



$$T = A_f f_{fu}$$

Development Length of Straight Bars

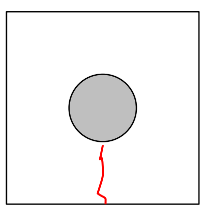
GFRP development length is **typically longer compared to steel** and is a function of the tensile stress in the bar

$$l_d \geq \max\left(\frac{\alpha\dfrac{f_{fr}}{\sqrt{f_c'}} - 340}{13.6 + \dfrac{C}{d_b}}d_b; 20d_b\right)$$

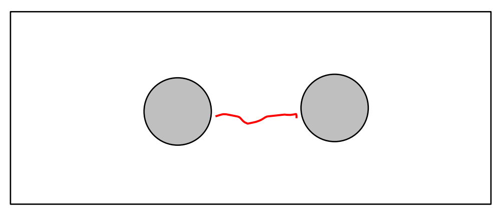
AASHTO (2.9.7.4.1-1)

where

α = 1.5 for top bars and 1.0 for bottom bars
f_{fr} = the required reinforcing stress
C = the lesser of the clear cover or ½ the center to center bar spacing
Note: the value of C/d_b is limited to a max of 3.5



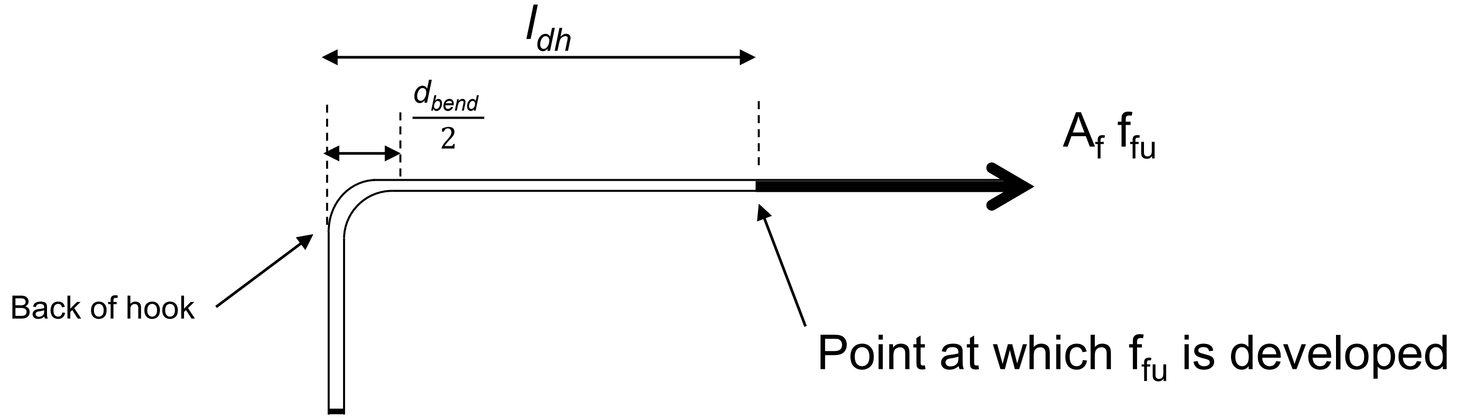
bar-cover



bar-bar

Development Length of Bent Bars

"Standard hook" consists of the hook itself plus a straight length. Development length for a hooked bar (l_{dh}) is measured as shown:



The following expression is recommended:

$$l_{dh} = \begin{cases} 63.2 \dfrac{d_b}{\sqrt{f_c'}} \; for & f_{fd} \le 75\ ksi \\ \dfrac{f_{fd}}{1.2} \dfrac{d_b}{\sqrt{f_c'}} \; for & 75\ ksi < f_{fd} \le 150\ ksi \\ 126.4 \dfrac{d_b}{\sqrt{f_c'}} \; for & f_{fd} \ge 150\ ksi \end{cases}$$

AASHTO (2.9.7.4.3.1)

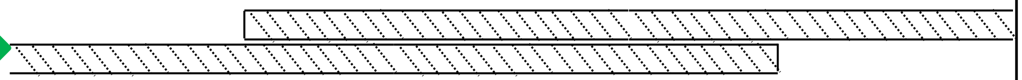
$$l_{dh} \ge 12 d_b$$

$$l_{dh} \ge 9\ in$$

Splices

Types of splices currently possible with GFRP bars

Lap Splice



Mechanical Splice (Coupler)

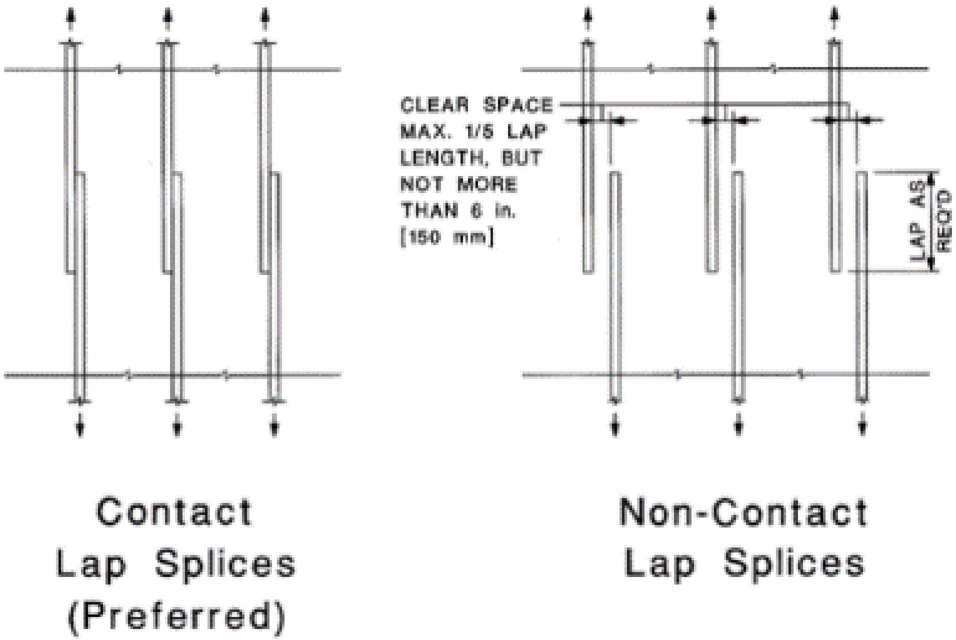
Splicing GFRP bars by mechanical connections is **not permitted** unless the full tensile capacity of the GFRP bar is achieved as substantiated by tensile test data per ASTM D7205

FDOT

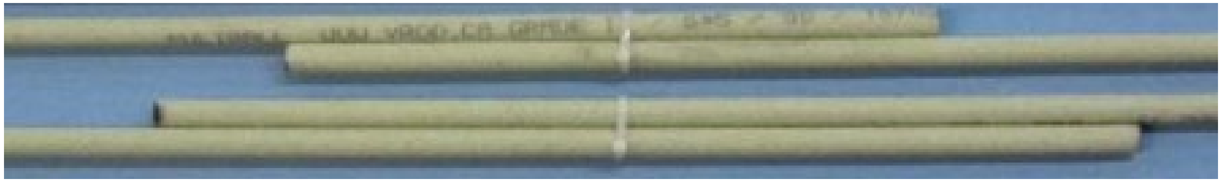
UNIVERSITY OF MIAMI

Center for Integration of Composites into Infrastructure

Lap Splices

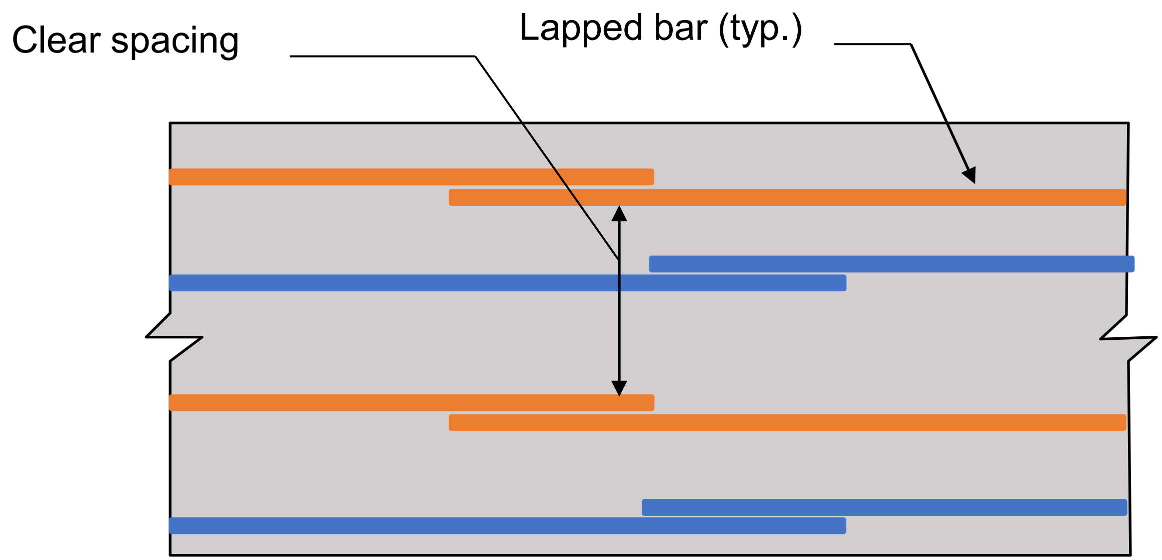


Lap Splices: Two overlapping bars, possibly tied together; staggered to reduce congestion; must overlap by required lap length



Tension Lap Splices

AASHTO spec requires staggered splices to provide redundancy



Clear spacing of lap-spliced bars for determination of l_{st} for staggered splices

Minimum splice length:

No splice class distinction

For GFRP splices: $l_{st} = \begin{cases} 1.3 l_d \\ 12\ in \end{cases}$

l_d calculated to provide 25% tensile force

AASHTO (2.9.7.6)

Table of Contents

- General Considerations
- Bending Moment Capacity
- Serviceability
- Static & Cyclic Fatigue
- Anchorage and Development
- **Special Considerations**
- Concluding Remarks

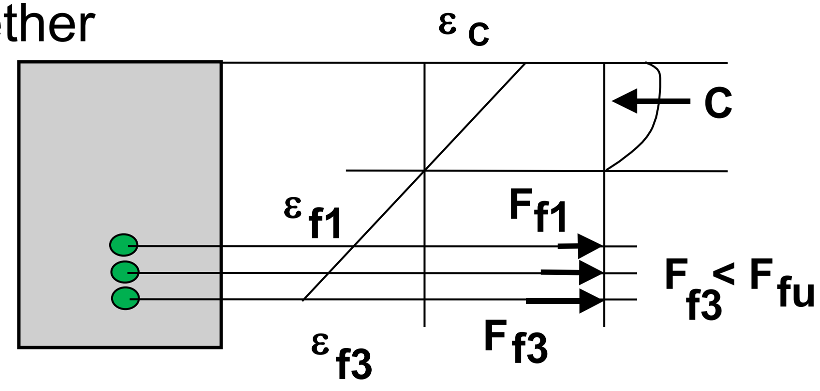
Center for Integration of Composites into Infrastructure

Special Considerations

Multiple layers and/or differing types of bars

AASHTO (2.6.3.2.4)

- Due to linear-elastic behavior of FRP, multiple layers cannot be lumped together
- Stresses need to be computed at each individual layer



Moment Redistribution

AASHTO (1.3)

- Plastic hinges shall not be assumed
- Moment redistribution should <u>not</u> be considered

Compression Reinforcement

- FRP has a lower compression strength and stiffness than tensile equivalent properties. Difficult to measure, but higher than concrete

- Any FRP bar in compression should be ignored in design calculations and substituted with an equivalent area of concrete (n_f = 1 in compression)

AASHTO Article 1.3

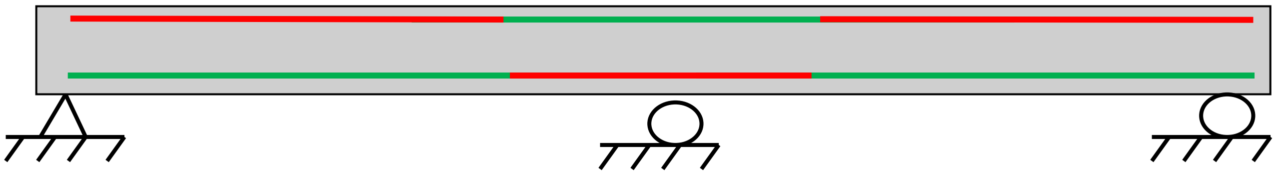


Table of Contents

Concluding Remarks

- Flexural capacity of FRP-reinforced flexural member dependent on **tension or compression failure**
- FRP reinforcement is brittle, but provides failure warning in terms of member deflection
- **Serviceability requirements** may govern design. **Allowable stresses** under sustained or cyclic loading must be checked
- FRP can be placed in compression zones but not be considered in calculations
- Reduced **bond properties** affect development length and crack control

Questions?

Thank U

FLEXURE RESPONSE OF GFRP REINFORCED CONCRETE

2.1 Review Questions: Fundamentals

UNIVERSITY OF MIAMI

Center for Integration of Composites into Infrastructure

Review Questions

2.1.1) The substitution of GFRP for steel on an equal area basis would typically result in: ______________________________.

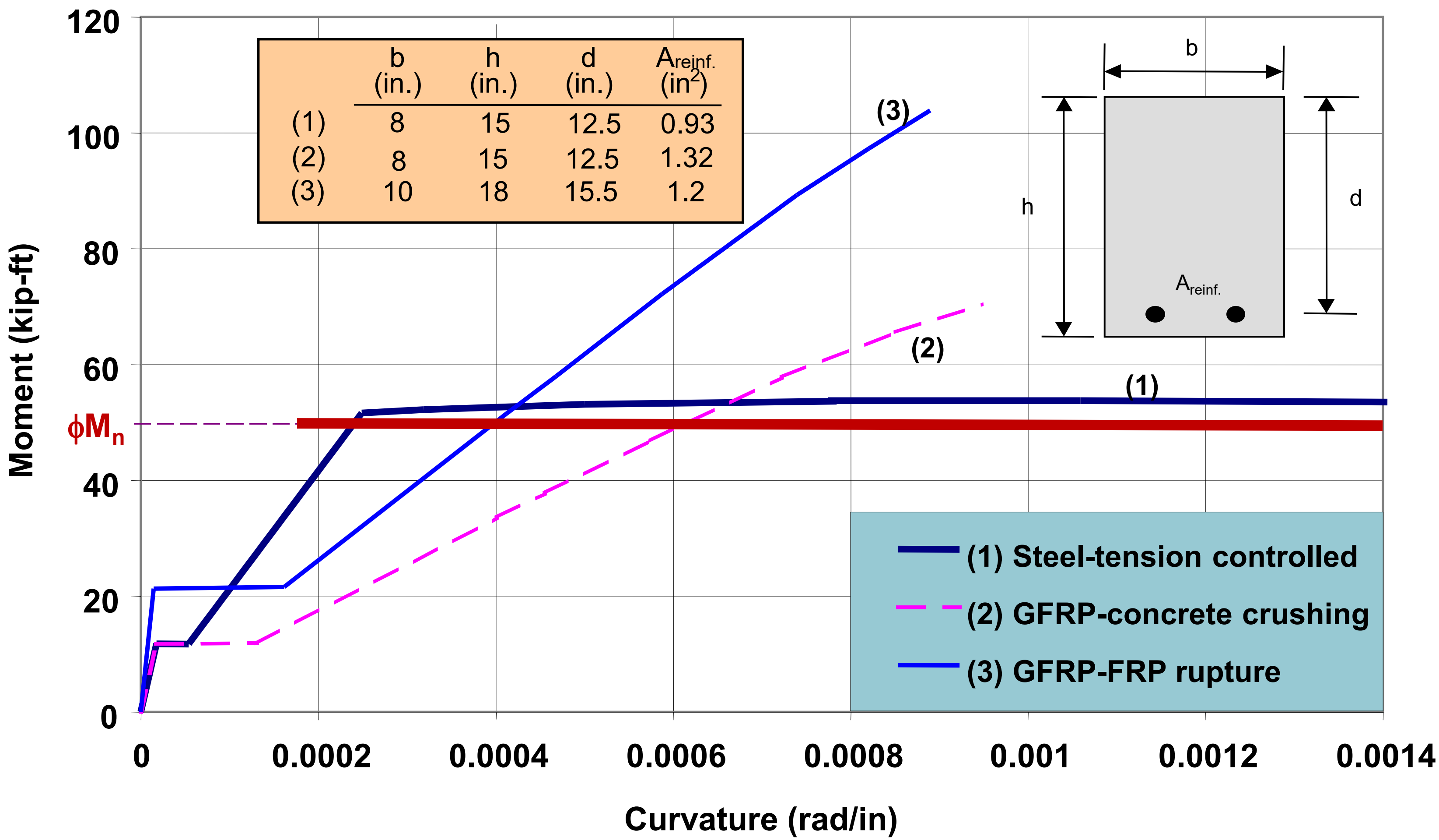
a. No difference

b. Larger deflections and wider crack widths

c. Wider crack widths

d. Larger deflections

Moment-Curvature Diagrams



Review Questions

2.1.1) The substitution of GFRP for steel on an equal area basis would typically result in: ____________________________.

a. No difference

b. Larger deflections and wider crack widths

c. Wider crack widths

d. Larger deflections

Review Questions

2.1.2) When designing structures with FRP the preferred failure mode in flexure is: ______________________________.

a. FRP rupture

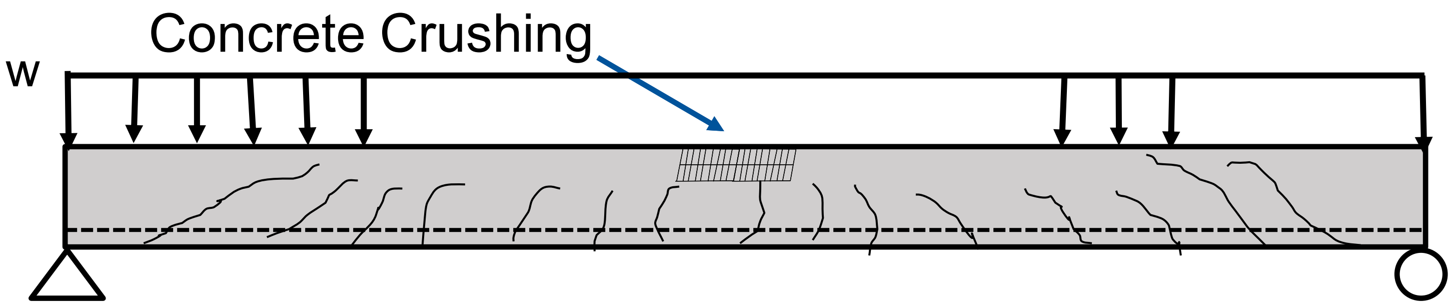
b. Concrete crushing

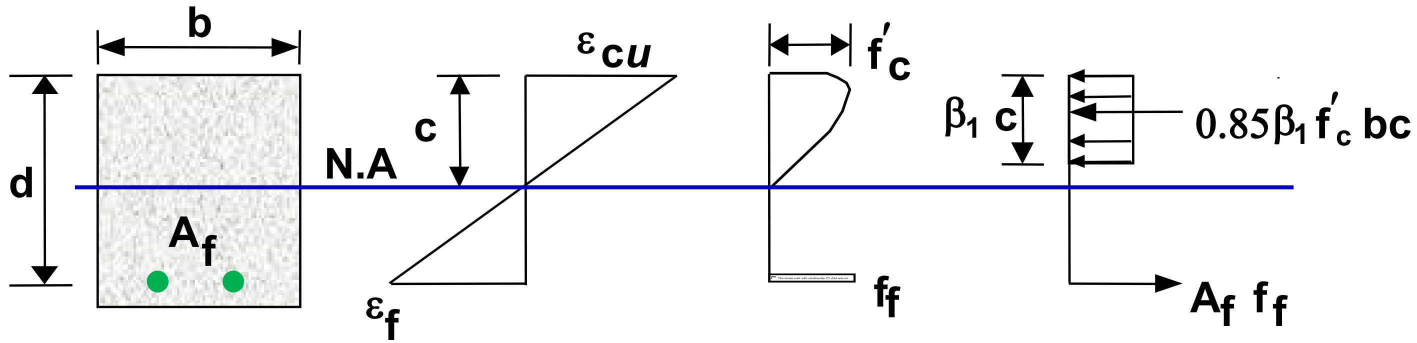
c. None – it is not safe to design with FRP

d. Debonding between reinforcement and concrete

Failure Modes

Compression-controlled: concrete crushing





β_1 c = a

Review Questions

2.1.2) When designing structures with GFRP the preferred failure mode in flexure is: ______________________________.

a. GFRP rupture

b. Concrete crushing

c. None – it is not safe to design with GFRP

d. Debonding between reinforcement and concrete

Review Questions

2.1.3) In GFRP-RC flexural design the safety factor is increased (Φ is reduced): ______________________________.

a. To account for the design of over-reinforced members

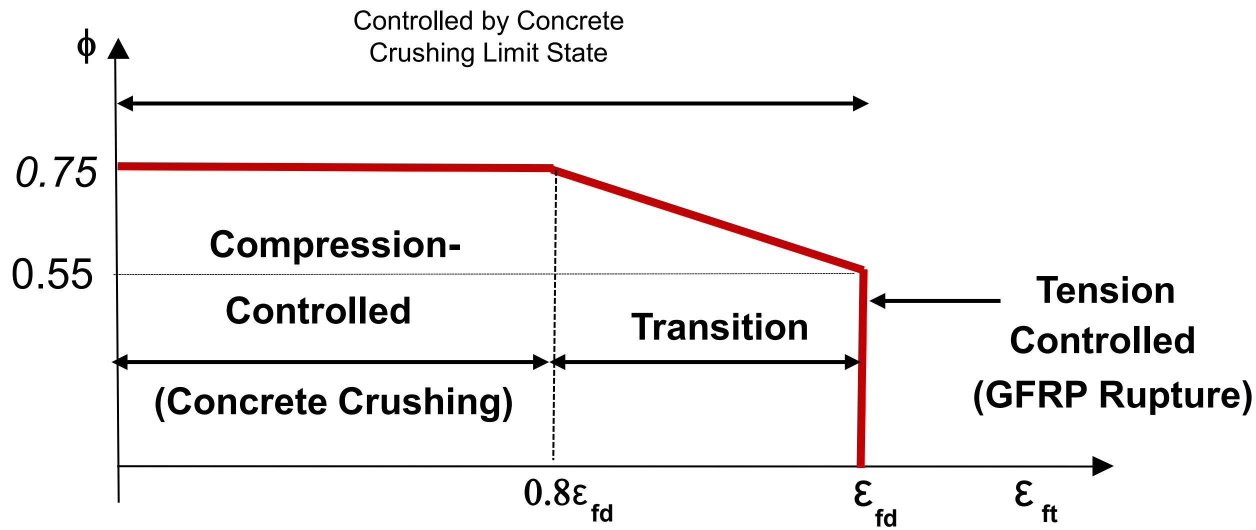
b. To consider the long-term behavior

c. To consider the lack of ductility

d. Because a member governed by GFRP bar rupture will have a brittle failure

FDOT UNIVERSITY OF MIAMI Center for Integration of Composites into Infrastructure

Strength Reduction Factors (AASHTO)



$$\phi = \begin{cases} 0.55 & for\ \varepsilon_{ft} \leq \varepsilon_{fd} \\ 1.55 - \dfrac{\varepsilon_{ft}}{\varepsilon_{fd}} & for \quad 0.80\varepsilon_{fd} < \varepsilon_{ft} < \varepsilon_{fd} \\ 0.75 & for\ \ \varepsilon_{ft} \geq 0.80\varepsilon_{fd} \end{cases}$$

(AASHTO 2.5.5.2)

Review Questions

2.1.3) In GFRP-RC flexural design the safety factor is increased (Φ is reduced): ______________________________.

a. To account for the design of over-reinforced members

b. To consider the long-term behavior

c. To consider the lack of ductility

d. Because a member governed by GFRP bar rupture will have a brittle failure

Review Questions

2.1.4) A member governed by GFRP bar rupture will have a brittle failure: ______________________________.

a. True

b. False

Failure Modes

FRP Rupture Failure in GFRP Reinforced Beam

Review Questions

2.1.4) A member governed by GFRP bar rupture will have a brittle failure: ______________________________.

a. True

b. False

Review Questions

2.1.5) For the flexural design of GFRP-RC members which of the following assumptions **<u>is false</u>**:

a. Plane sections remain plane after deformation

b. Tensile strength of concrete is not neglected

c. Stress-strain of FRP is linear until failure

d. FRP is completely bonded to concrete

Flexural Theory

Assumptions:

1. Plane sections remains plane after deformation
2. Ultimate concrete strain is 0.003
3. Tensile strength of concrete is neglected
4. FRP is perfectly bonded to concrete
5. Stress-strain of FRP is linear until failure

Ultimate Flexural Strength:

M_n = nominal capacity

M_u = factored capacity

$$\boxed{\phi M_n \geq M_u}$$

ϕ = strength reduction factor (depends on the mode of failure)

$$M_u = \underbrace{1.25 M_{DC} + 1.75 M_{LL}}_{\text{For } \textit{AASHTO} \text{ example}}$$

$$M_u = \underbrace{1.2 M_D + 1.6 M_L}_{\text{For } \textit{ACI} \text{ example}}$$

Review Questions

2.1.5) For the flexural design of GFRP-RC members which of the following assumptions **is false**:

a. Plane sections remain plane after deformation

b. Tensile strength of concrete is not neglected

c. Stress-strain of FRP is linear until failure

d. FRP is completely bonded to concrete

Review Questions

2.1.6) Tension lap splice for GFRP bars is: _______________.

a. The same as the development length of the bar

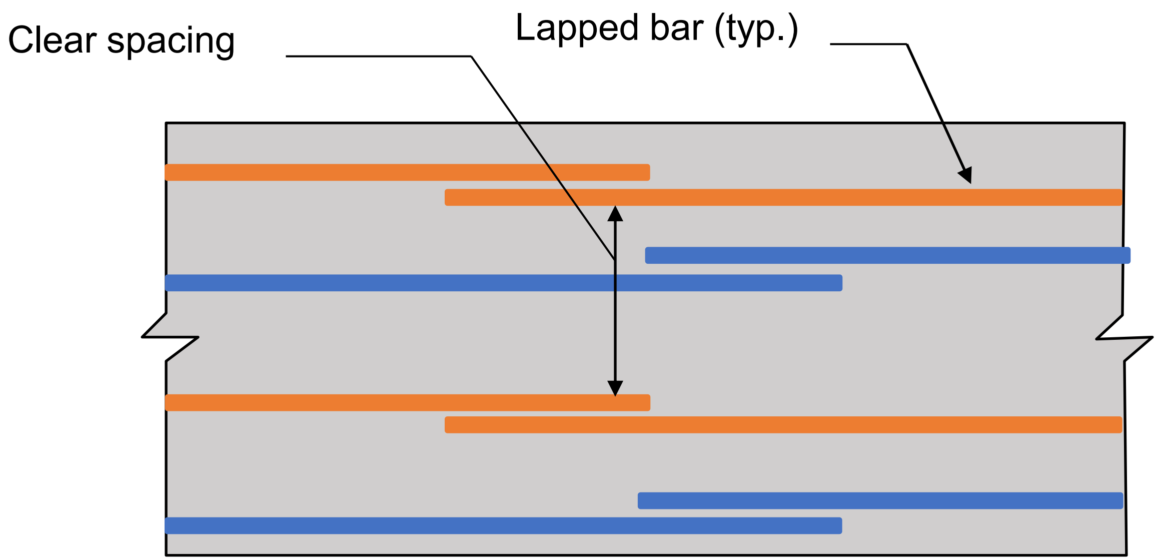
b. 1.25 times the development length of the bar

c. 1.30 times the development length of the bar

d. 1.60 times the development length of the bar

Tension Lap Splices

AASHTO specification requires staggered splices to provide redundancy



Clear spacing of lap-spliced bars for determination of l_d for staggered splices

No splice class distinction

Minimum splice length:

For GFRP splices: $$l_{st} = \begin{cases} 1.3 l_d \\ 12\ in \end{cases}$$

l_d calculated to provide 25% tensile force

AASHTO (2.9.7.6)

Review Questions

2.1.6) Tension lap splice for GFRP bars is: _______________.

a. The same as the development length of the bar

b. 1.25 times the development length of the bar

c. 1.30 times the development length of the bar

d. 1.60 times the development length of the bar

Review Questions

2.1.7) When designing with GFRP the load factors are: __________________________.

a. Higher than the ones used when designing steel RC

b. Lower than the ones used when designing steel RC

c. The same as the ones used when designing steel RC

d. Not defined yet

Load Factors

AASHTO Table 3.4.1-1

Load Combination Limit State	*DC DD DW EH EV ES EL PS CR SH*	*LL IM CE BR PL LS*	*WA*	*WS*	*WL*	*FR*	*TU*	*TG*	*SE*	Use One of These at a Time				
										EQ	*BL*	*IC*	*CT*	*CV*
Strength I (unless noted)	γ_p	1.75	1.00	—	—	1.00	0.50/1.20	γ_{TG}	γ_{SE}	—	—	—	—	—
Strength II	γ_p	1.35	1.00	—	—	1.00	0.50/1.20	γ_{TG}	γ_{SE}	—	—	—	—	—
Strength III	γ_p	—	1.00	1.40	—	1.00	0.50/1.20	γ_{TG}	γ_{SE}	—	—	—	—	—
Strength IV	γ_p	—	1.00	—	—	1.00	0.50/1.20	—	—	—	—	—	—	—
Strength V	γ_p	1.35	1.00	0.40	1.0	1.00	0.50/1.20	γ_{TG}	γ_{SE}	—	—	—	—	—
Extreme Event I	γ_p	γEQ	1.00	—	—	1.00	—	—	—	1.00	—	—	—	—
Extreme Event II	γ_p	0.50	1.00	—	—	1.00	—	—	—	—	1.00	1.00	1.00	1.00
Service I	1.00	1.00	1.00	0.30	1.0	1.00	1.00/1.20	γ_{TG}	γ_{SE}	—	—	—	—	—
Service II	1.00	1.30	1.00	—	—	1.00	1.00/1.20	—	—	—	—	—	—	—
Service III	1.00	0.80	1.00	—	—	1.00	1.00/1.20	γ_{TG}	γ_{SE}	—	—	—	—	—
Service IV	1.00	—	1.00	0.70	—	1.00	1.00/1.20	—	1.0	—	—	—	—	—
Fatigue I—*LL*, *IM* & *CE* only	—	1.50	—	—	—	—	—	—	—	—	—	—	—	—
Fatigue II—*LL*, *IM* & *CE* only	—	0.75	—	—	—	—	—	—	—	—	—	—	—	—

Load factors are applicable with inclusion of new creep rupture limit state and load factors.

$$1.2DL + 0.2LL$$

Review Questions

2.1.7) When designing with FRP the load factors are: ____________________________.

a. Higher than the ones used when designing steel RC

b. Lower than the ones used when designing steel RC

c. The same as the ones used when designing steel RC

d. Not defined yet

Review Questions

2.1.8) The purpose of shrinkage/temperature reinforcement is: ___________________________.

a. Distribute load

b. Improve development capacity of GFRP

c. Control crack width

d. Reduce member thickness

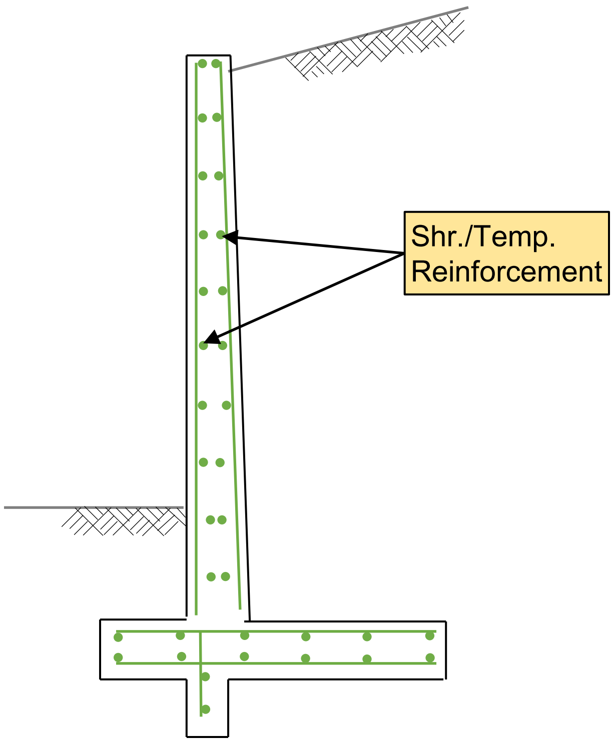
Shrinkage & Temperature Reinforcement

GFRP reinforced retaining wall example:

Properties

Width of Wall Average:	13.38 inches
Height of Wall:	18 feet
Bar Size of Temp. Reinf.	4
Elastic Modulus of GFRP, E_f	6500 ksi, 8700 ksi
Design Tensile Strength, f_{fd}	75.6 ksi, 105 ksi

Minimum Area of S/T Reinf.	
A_s =	0.29 in^2/ft
$A_{f,6500\ ksi}$ =	0.58 in^2/ft
$A_{f,8700\ ksi}$ =	0.55 in^2/ft



Review Questions

2.1.8) The purpose of shrinkage reinforcement is: ______________________.

a. Distribute load

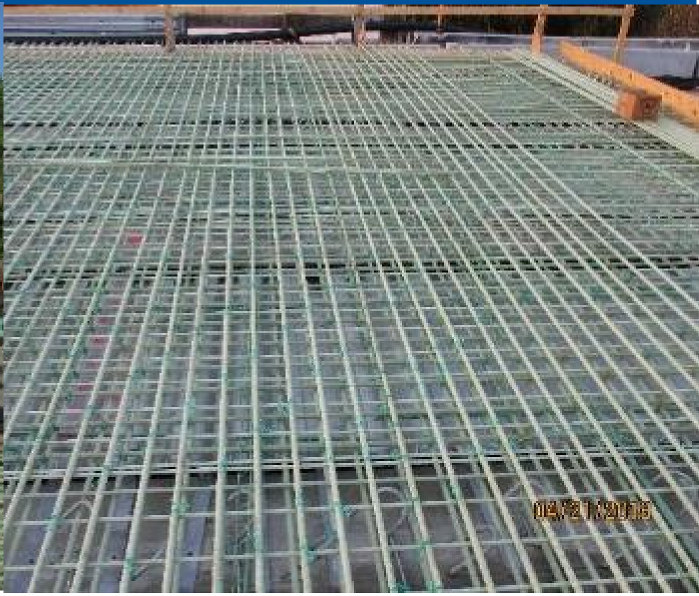
b. Improve development capacity of FRP

c. Control crack width

d. Reduce member thickness

FLEXURE RESPONSE OF GFRP REINFORCED CONCRETE

2.2 Design Example: Flat Slab



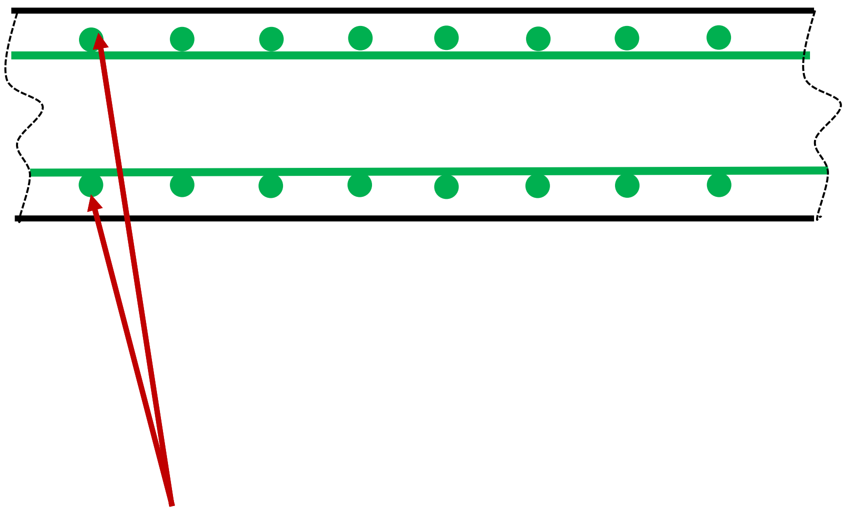
FDOT

UNIVERSITY OF MIAMI

Center for Integration of Composites into Infrastructure

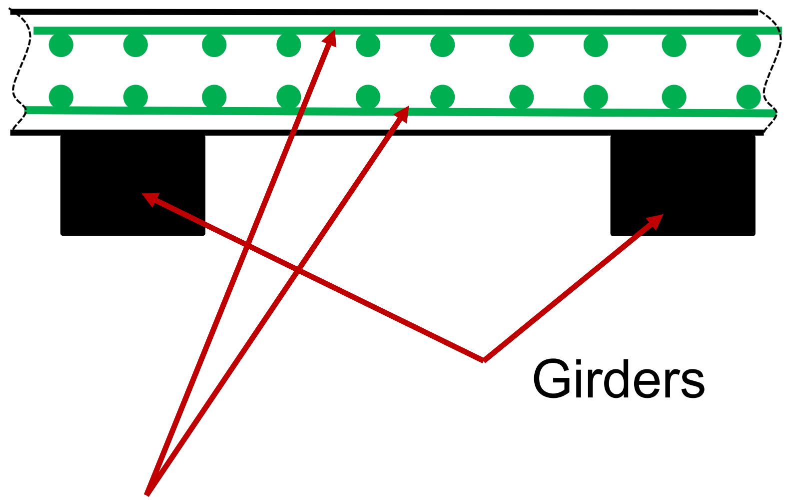
Flat Slab vs. Deck Type

Flat Slab



Longitudinal Bars Provide Flexural Resistance

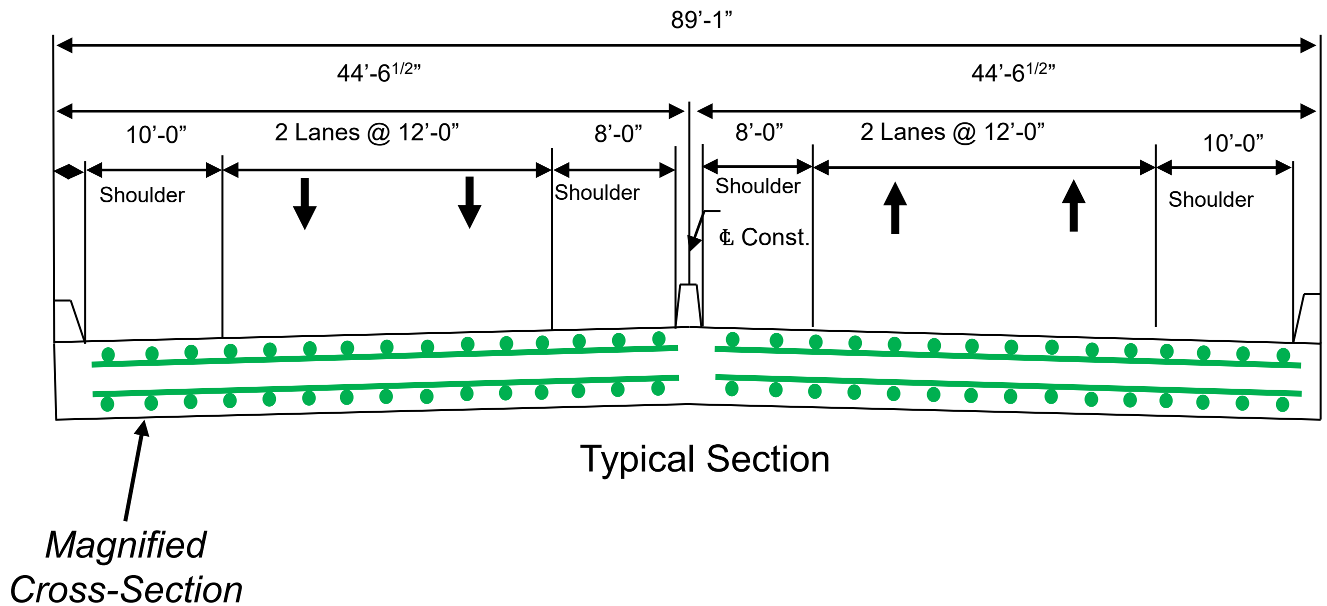
Deck



Transverse Bars Provide Flexural Resistance

Objectives

- Demonstrate the design of a FLAT SLAB bridge superstructure utilizing method prescribed by AASHTO



- Show calculations with emphasis on flexural design for positive moment

Analysis of Flexural Member with GFRP

Given A_f, d, b, f_{fd}

Determine $\rho = \frac{A_f}{bd}$

Calculate stress in GFRP reinforcement at nominal flexural resistance:

$$f_f = \sqrt{\frac{(E_f \varepsilon_{cu})^2}{4} + \frac{0.85\beta_1 f_c'}{\rho_f} E_f \varepsilon_{cu}} - 0.5 E_f \varepsilon_{cu} \le f_{fd}$$

Check to see if GFRP has ruptured

$$f_f < f_{fd}$$

No

FRP Rupture

$$c = \frac{\varepsilon_{cu}}{\varepsilon_{cu} + \varepsilon_{fd}} d$$

$$a = \beta_1 c_b$$

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

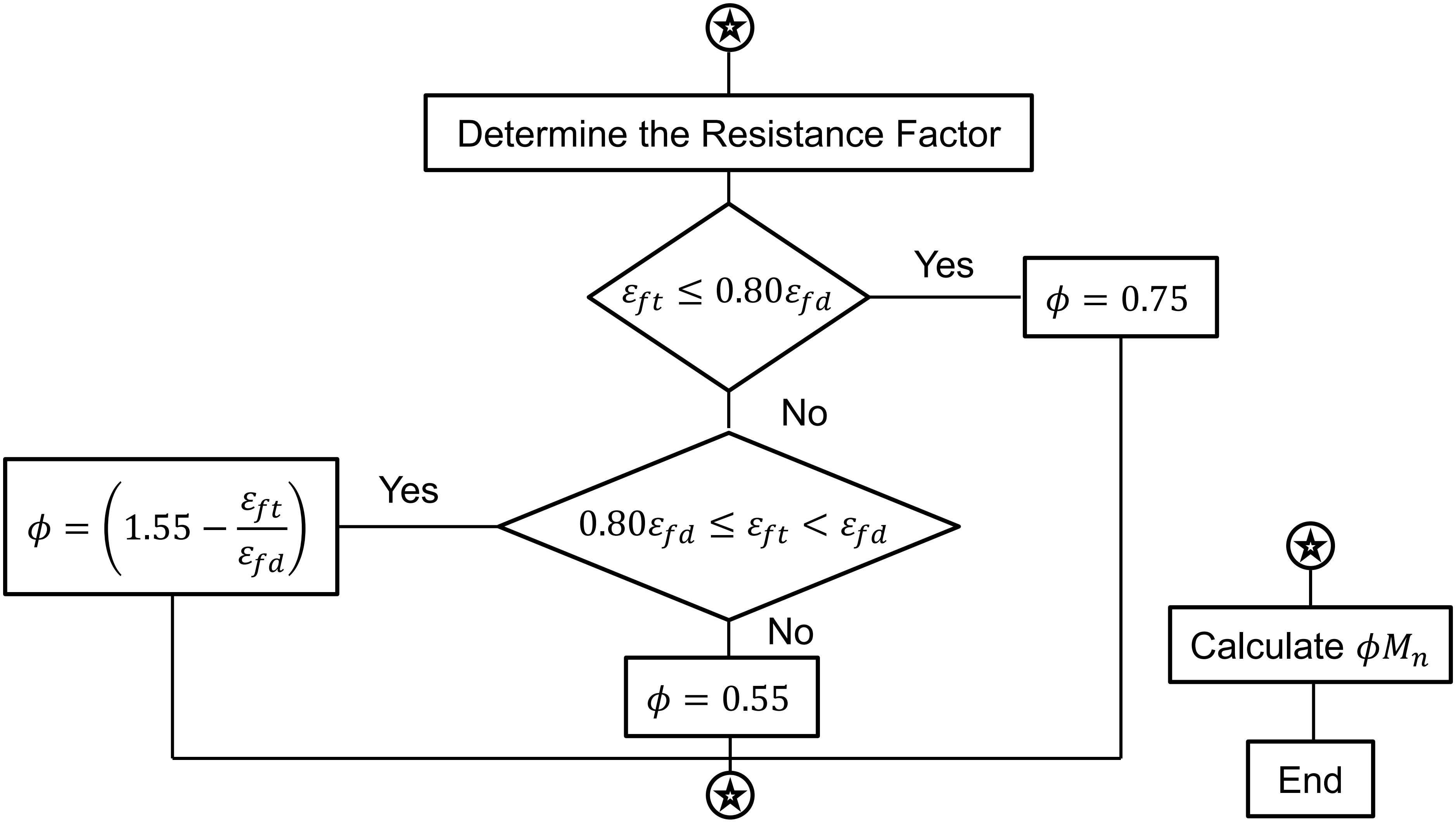
Yes

Concrete Crushing

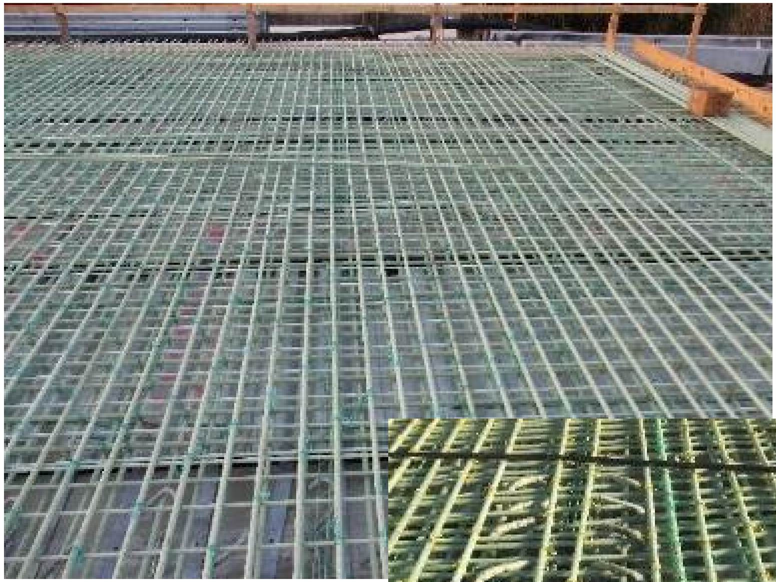
$$a = \frac{A_f f_f}{0.85 f_c' b}$$

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

Analysis of Flexural Member with GFRP



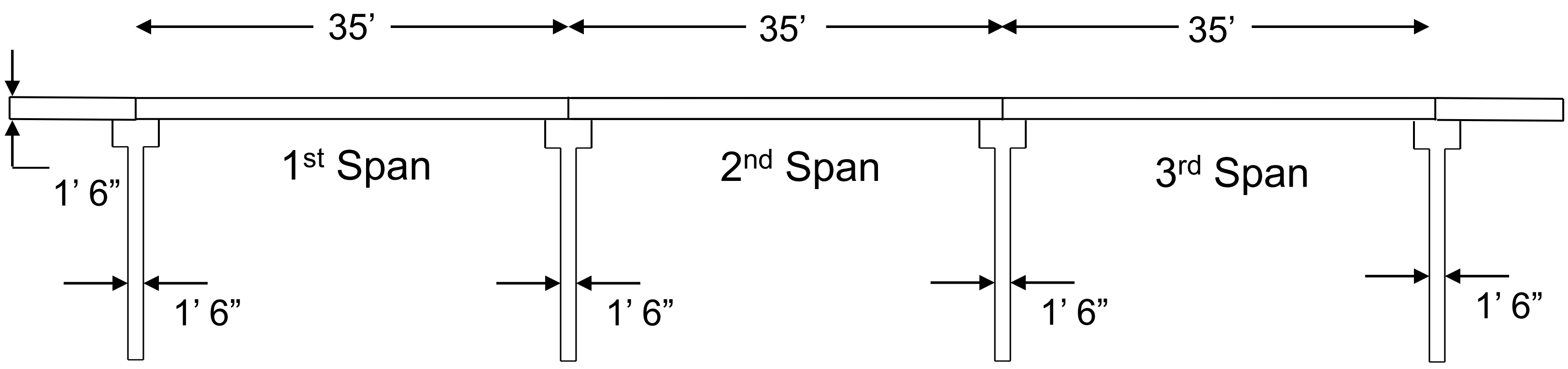
Context



GFRP Slab Design

Bridge Geometry

Dimension of Bridge in Front View



Overall bridge length = 105 ft

Bridge design span length = 35 ft

Ideal span range: up to 40 ft

GFRP Reinforcement Properties

Geometric Properties

$t_{slab} = 18\ inches$ — thickness of slab

$b = 12\ inches$ — design strip width

$Cover = 2\ inches$ — cover for GFRP members

Effective Depth of Reinforcement

$$d_{fl.slab} = 18in - 2in - \frac{1.27in}{2} = 15.9\ inches$$

Primary Reinforcement

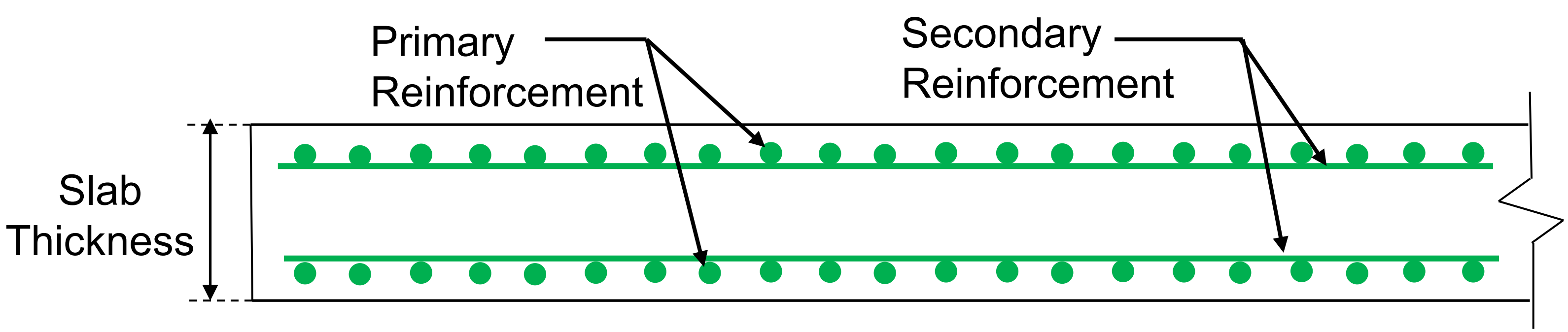
$BarSize_{slab.pr} = 10$

$BarSpace_{slab.pr} = 4\ inches$

Secondary Reinforcement

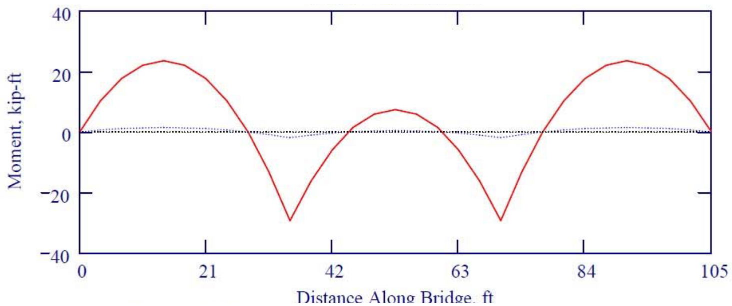
$BarSize_{slab.sec} = 6$

$BarSpace_{slab.sec} = 8\ inches$

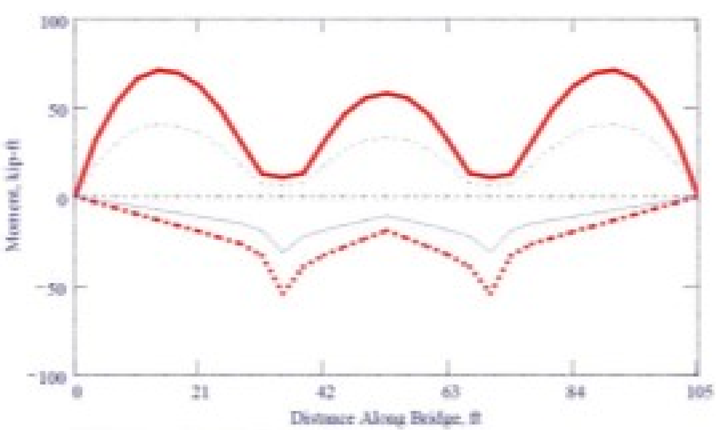


Dead & Live Load Analysis

Dead Load Moments



Strength I & Service I Live Load Moments



Maximum positive moment and corresponding fatigue values

Strength I	$M_{str1.pos} = 100.9\ kip - ft$
Service I	$M_{ser1.pos} = 64.6\ kip - ft$
Live Load Only	$M_{liveLoad.pos} = 39.6\ kip - ft$

Maximum negative moment and corresponding fatigue values

Strength I	$M_{str1.neg} = 93.2\ kip - ft$
Service I	$M_{ser1.neg} = 61.9\ kip - ft$
Live Load Only	$M_{liveLoad.neg} = 39.6\ kip - ft$

Check Primary Reinforcement

Positive Moment Region – Flexural Strength at Support

$f_c' = 4{,}500\ psi$

concrete compressive strength

$\alpha_1 = \max\left(0.75, 0.85 - 0.02(f_c' - 10)\right) = 0.90$

[AASHTO BDS 5.6.2.2]

$\beta_1 = 0.85 - 0.05(4.5ksi - 4ksi) = 0.83$

[AASHTO BDS 5.6.2.2]

<u>Area of primary reinforcement per linear foot</u>

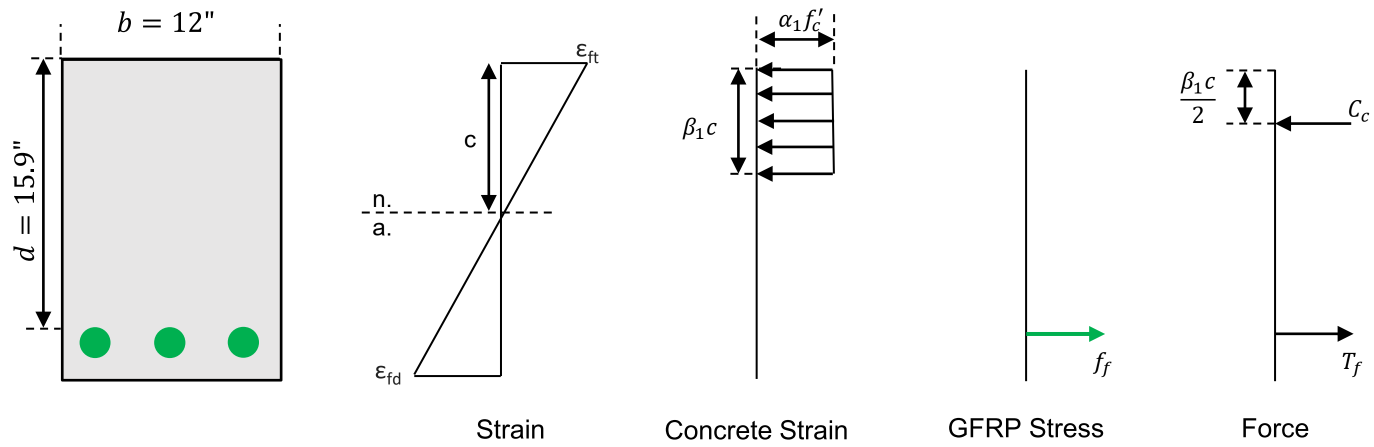
$$A_{fl.slab} = (1.27in^2)\left(\frac{12in}{4in}\right) = 3.8\ in^2$$

area of GFRP reinforcement per foot of negative moment

<u>Reinforcement Ratio</u>

$$\rho_f = \frac{A_f}{b \cdot d} = \frac{3.8in^2}{(1ft)(15.9in)} = 0.02001$$

Check Primary Reinforcement



Effective strength in GFRP reinforcements at strength limit state

$$f_f = \sqrt{\frac{(E_f \varepsilon_{cu})^2}{4} + \frac{0.85\beta_1 f_c'}{\rho_f} E_f \varepsilon_{cu}} - 0.5E_f \varepsilon_{cu}$$

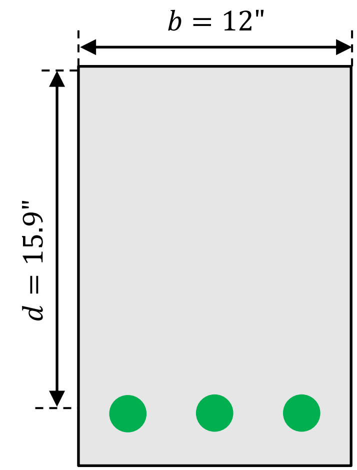
$$= \sqrt{\frac{(6500ksi \cdot 0.003)}{4} + \frac{0.85(0.83)(4.5)}{0.02001}(6500\ ksi)(0.003)} - 0.5(6500ksi)(0.003) = 46.6ksi$$

$$f_f = \min(f_f, f_{fd}) = \min(46.6ksi, 54.1ksi) \quad \boxed{= 52.3ksi} \quad f_f < f_{fd}$$

compression controlled

Calculate Resistance Factor

GFRP strain check



$$\varepsilon_{ft} = \frac{f_f}{E_f} = \frac{52.3ksi}{6500ksi} = 0.00716$$

$$\varepsilon_{fd} = \frac{f_{fd}}{E_f} = \frac{54.1ksi}{6500ksi} = 0.00833$$

Calculate Resistance Factor for Flexural Strength (GFRP)

$$\phi = \begin{cases} 0.75 & if \quad \varepsilon_{ft} \leq 0.80\varepsilon_{fd} \\ \left(1.55 - \dfrac{\varepsilon_{ft}}{\varepsilon_{fd}}\right) & if \quad 0.80\varepsilon_{fd} < \varepsilon_{ft} < \varepsilon_{fd} \\ 0.55 & otherwise \end{cases}$$

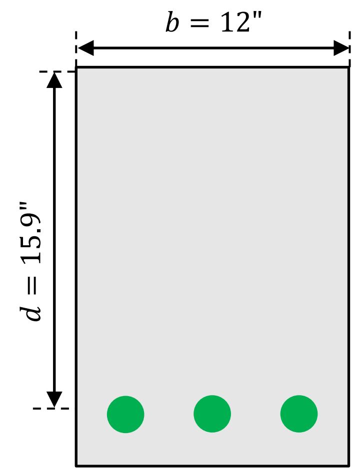
$$\varepsilon_{ft} \leq 0.00667$$

$$0.00667 \leq \varepsilon_{ft} \leq 0.00833$$

$$\boxed{\phi = 0.69}$$

Check Primary Reinforcement

$$a = \frac{A_f f_f}{0.85 f_c' b} = \frac{(3.8in^2)(52.3ksi)}{0.85(4500psi)(12in)} = 3.9in$$



Section

Calculate corresponding moment

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

$$M_n = (3.8in^2)(46.6ksi)\left(15.9in - \frac{3.9in}{2}\right) = 205.9k - ft$$

$$\boxed{M_r = \phi M_n = (0.69)(224.3k - ft) = 142.1k - ft}$$

$$\boxed{Demand/Capacity: Moment_{2.slab} = \frac{M_{str1.pos}}{M_{r.2.slab}} = 0.71}$$

2.3 Design Example
Creep Rupture (Flat Slab)

FDOT

UNIVERSITY OF MIAMI

Center for Integration of Composites into Infrastructure

Creep Rupture Limit State

Creep Rupture Limit State

$M_{1.creep.slab} = 50.7k - ft$ Sustained loads only

$$E_c = 120{,}000\Phi_{limerock} w_c^{2.0} \sqrt[3]{f_c' \cdot ksi^2} = 4165ksi \qquad n = \frac{E_f}{E_c} = \frac{6500ksi}{4165ksi} = 1.6$$

$$k = \sqrt{2pn - (pn)^2} - pn = \sqrt{2(0.02) - (0.02 \cdot 1.6)^2} - (0.02)(1.6) = 0.2$$

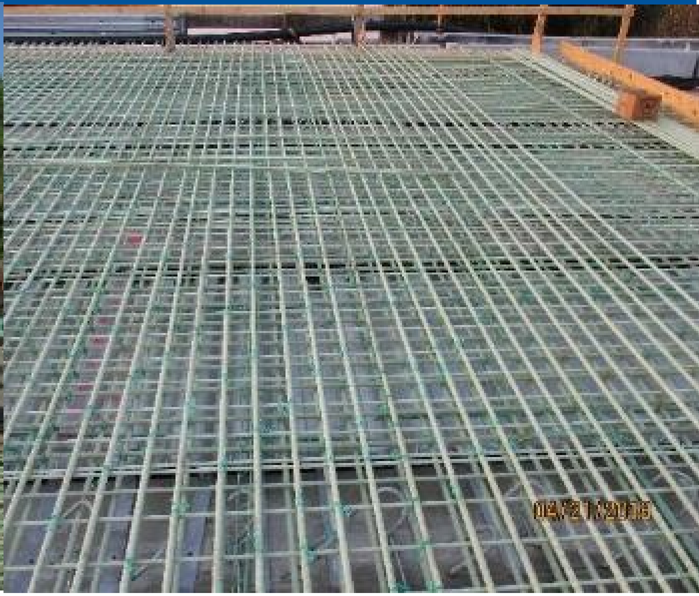
$$I_{cr} = \frac{bd^3}{3}k^3 + nAst(d - kd)^2 =$$

$$= \frac{(12in)(15.9in)^3}{3}(0.2)^3 + (1.6)(3.8in^2)(15.9in - 0.2 \cdot 15.9in^2)^2 = 1108in^4$$

$$f_{f1.creep} = \frac{n \cdot d_{f1.slab}(1 - k_{1.slab})}{I_{cr}} \cdot M_{1.creep.slab} \qquad \boxed{= 11.3ksi}$$

$$\boxed{C_c C_E f_{fu} = C_c f_{fd} = (0.3)(54.1ksi)} \qquad \boxed{C_c f_{fd} = 16.2ksi}$$

2.4 Design Example
Minimum Reinforcement
(Flat Slab)



FDOT

UNIVERSITY OF MIAMI

Center for Integration of Composites into Infrastructure

Minimum Reinforcement

$$f_r = 0.24\sqrt{f'_{c.super} \cdot (ksi)} = 0.51ksi$$

Concrete Modulus of rupture

$$S_r = \frac{t_{slab}^3}{6} b = 648in^3$$

Uncracked concrete section modulus

$$M_{cr.slab} = 1.6 f_r S_r = 44.0k - ft$$

Slab cracking moment

$$M_{min.slab} = \min\left(1.33 M_{str1.pos}, M_{cr.slab}\right) = 44.0k - ft$$

Minimum required factored flexural resistance

$$M_{r.slab} = \min\left(M_{r.pos}, M_{r.neg}\right) = 142.1k - ft$$

Flexural capacity of slab

$$CheckMinReinf_{slab} = if\left(M_{r.slab} \geq M_{min.slab}, \text{"OK","No Good"}\right)$$

$$\boxed{CheckMinReinf_{slab} = \text{"}OK\text{"}}$$

AASHTO GFRP-Reinforced Concrete Design Training Course



FDOT

UNIVERSITY OF MIAMI

Center for Integration of Composites into Infrastructure