AASHTO GFRP - REINFORCED CONCRETE DESIGN

TRAINING COURSE

COMPANION WORKBOOK

PHOTO: Installation of first GFRP pile cage on A1A Flagger beach project, April 2019.

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PHOTO: GFRP cage assembly for the Halls River Bridge Project, April 2017.

1. INTRODUCTION TO FRP-RC & MATERIAL PROPERTIES

NOTES

1.1. Review Questions: Fundamentals

1.1.1) Where could GFRP reinforcement for concrete be most suitable? _____________.

a. Any concrete member susceptible to steel corrosion by chloride ions
b. Any concrete member requiring non-ferrous reinforcement due to electro-magnetic considerations
c. As an alternative to epoxy, galvanized, or stainless steel rebars
d. Applications requiring thermal non-conductivity
e. All the above

1.1.2) Which of the following is **not** applicable to GFRP rebars? ___________________.

a. Corrosion resistant
b. Ductility
c. Low thermal and electrical conductivity
d. Light weight
e. High strength to weight ratio

1.1.3) The E-Modulus of GFRP rebars when compared to steel is approximately ______________.

a. 3 to 4 times lower
b. Comparable
c. 2 to 3 times higher
d. 3 to 4 times higher

NOTES

1.1.4) The durability of GFRP rebars: ______________.

a. Has been proven through accelerated aging protocols and by samples extracted from bridges that have been in service for 15 years
b. Is unknown
c. Is lower than steel
d. a) and c) are true

1.1.5) The density of GFRP is about ___________________.

a. About 4 times lighter than steel
b. Similar to that of steel
c. About 4 times heavier than steel
d. Half that of steel

1.1.6) FRP has higher strength in the _________ direction to the fibers.

a. Transverse
b. Parallel

1.1.7) The tensile strength of GFRP reinforcing bars with bends should be at minimum ______________________.

a. The same as straight GFRP reinforcing bars
b. 40% of the straight GFRP reinforcing bars
c. 60% of the straight GFRP reinforcing bars
d. 140% of straight GFRP reinforcing bars

NOTES

1.1.8) The guaranteed tensile strength of an GFRP bar as provided by the manufacturer is: __________________.

a. The mean tensile strength of a sample of test specimens
b. The mean tensile strength of a sample of test specimens minus three standard deviations
c. The mean tensile strength of a sample of test specimens minus two standard deviations
d. None of the above

1.1.9.) At higher temperatures of approximately ______ GFRP bars begin to soften.

a. 60 ^{o}F
b. 212 ^{o}F
c. 1220 ^{o}F
d. 2500^{o}F

2. FLEXURE RESPONSE OF GFRP-RC

NOTES

2.1. Review Questions: Fundamentals

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

2.1.2) When designing structures with GFRP 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

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

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

a. True
b. False

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 GFRP is linear until failure
d. GFRP is completely bonded to concrete

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

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

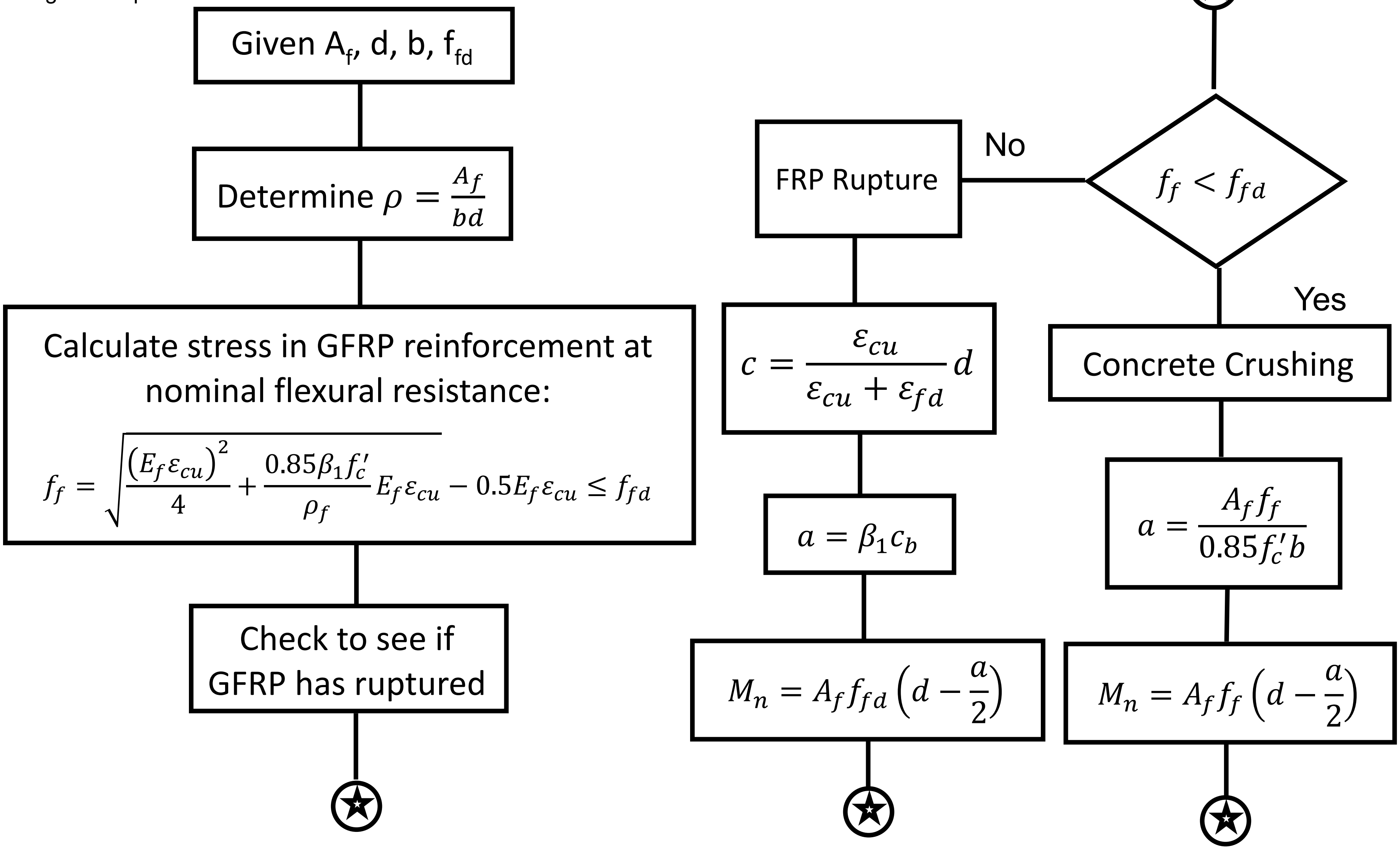
a. Higher than the ones used when designing steel RC
b. Lower than the ones used when designing steel RC
c. The same as the ones used when designing steel RC
d. Not defined yet

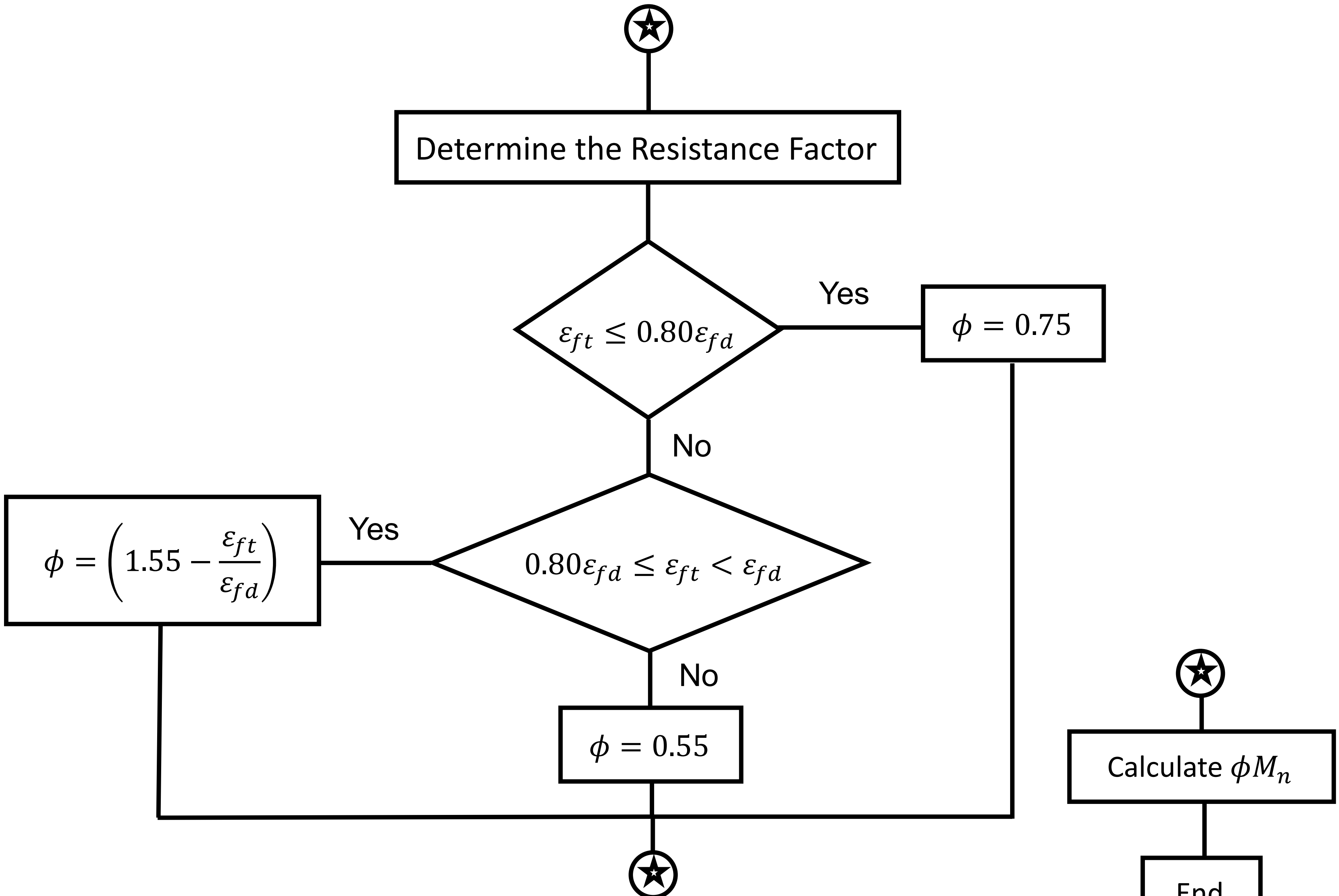
2.1.8) The purpose of shrinkage reinforcement is: ______________.

a. Distribute load
b. Improve development capacity of GFRP
c. Limit crack width
d. Reduce member thickness

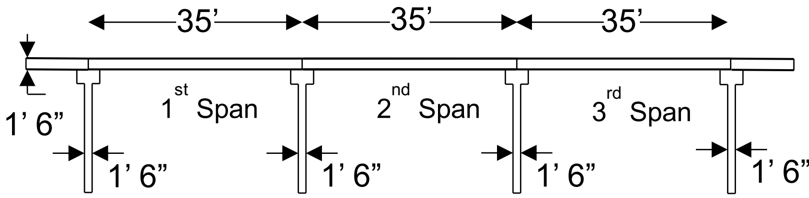
2.2. Design Example: Flat Slab

Refer to Annex A (Section A and B): LRFD Flat Slab Bridge Design Example

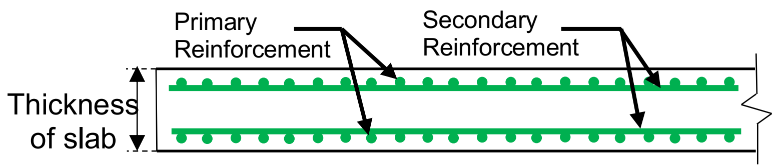



Determine the Resistance Factor
$\varepsilon_{ft} \leq 0.80\varepsilon_{fd}$
Yes
$\phi = 0.75$
No
$0.80\varepsilon_{fd} \leq \varepsilon_{ft} < \varepsilon_{fd}$
Yes
$\phi = \left(1.55 - \frac{\varepsilon_{ft}}{\varepsilon_{fd}}\right)$
No
$\phi = 0.55$
Calculate ϕM_n
End

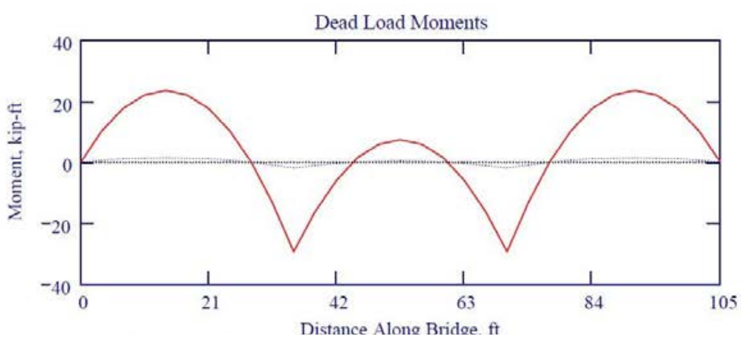
PART 1: DESIGN PARAMETERS



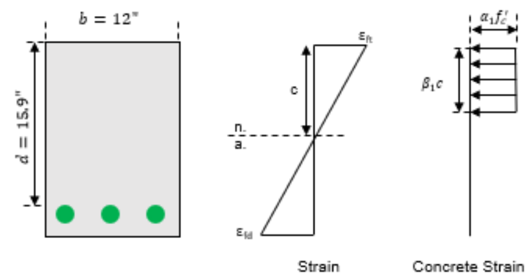
PART 2: MATERIAL PROPERTIES



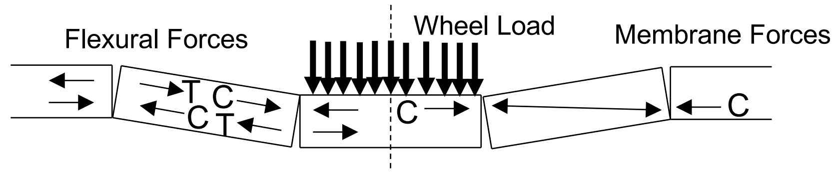
PART 3: FLAT SLAB DESIGN LOADS



FLAT SLAB DESIGN – TRADDITIONAL



FLAT SLAB DESIGN - EMPIRICAL



Adapted from Caltrans, 2015 Bridge Design Practice

PART 1: Design Parameters

A. General Criteria

Bridge Geometry

Skew Angle	-30 deg
Overall bridge length	105 ft
Bridge design span length	35 ft
Overall bridge width	89.1 ft

Number of Lanes

Roadway clear width	42 ft
Number of design traffic lanes per roadway	3

B. LRFD Criteria

Dynamic Load Allowance [AASHTO 2014, 3.6.2]

Impact factor for fatigue and fracture limit states	1+15/100
Impact factor for all other Limit states	1+33/100

Resistance Factors [AASHTO LRFD 2014 5.5.4.2]

Flexural and tension of Reinforced concrete	0.5 to 0.65 depending on reinf.
Shear and torsion of normal weight concrete	0.75

Span-to-Depth Ratios [AASHTO LRFD 2014 1.3.2]

Minimum slab thickness	18 in
Thickness of flat slab chosen	18 in
Slab width used for computation	12 in

C. FDOT Criteria

General

- The design life for bridge structure if 75 years
- Approach slabs are considered superstructure component
- Class II Concrete (Bridge Deck) will be used for all environmental classifications

Criteria for Deflection Only [SDG 2015, 1.2]

This provision for deflection only is not applicable, since no pedestrian loading is applied in this bridge design example.

Concrete and Environment [SDG 2015, 1.3]

The concrete cover for the slab is based on either the environmental classification [SDG 2015, 1.4]

Concrete clear cover for the slab	1.5 in
Concrete clear cover for the substructure not in contact with water	1.5 in

Minimum 28-day compressive strength of concrete components

II (Bridge Deck)	CIP Bridge Deck	4.5 ksi
IV	CIP Substructure	5.5 ksi
V (Special)	Concrete piling	6.0 ksi

Environmental Classifications [SDG 2015, 1.3]

The environment can be classified as either “Slight”, “Moderately”, or “Extremely” aggressive.

Environmental classification for superstructure	Extremely
Environmental classification for substructure	Extremely

D. Substructure

Bent 2 Geometry (Bent 3 similar)

Depth of intermediate bent cap	2.5 ft
Width of intermediate bent cap	3.5 ft
Length of intermediate bent cap	102.86 ft
Pile embedment depth	12 in
Pile size	18 in
Length of intermediate bent cap	11 ft
Length of edge bent cap	1.93 ft
Number of spans	9
Concrete clear cover	1.5 in

PART 2: Material Properties

A. Concrete Properties

28-day concrete compressive strength (Super)	4500 psi
28-day concrete compressive strength (Sub)	5500 psi

B. GFRP Reinforcement Properties

Primary Reinforcement

$BarSize_{slab.pr}$	10
$BarSpace_{slab.pr}$	4 in

Secondary Reinforcement

$BarSize_{slab.sec}$	6
$BarSpace_{slab.sec}$	8 in

Reinforcement Properties

E_f, tensile modulus of elasticity	6500 ksi
C_E, environmental reduction factor	0.7
C_b, bond reduction factor	0.83
C_c, creep rupture reduction factor	0.3

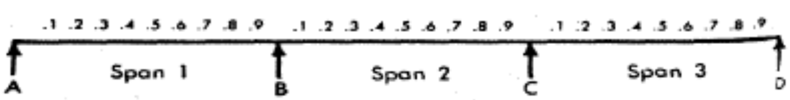
NOTES

PART 3: Flat Slab Design Loads

A. Dead Load Analysis

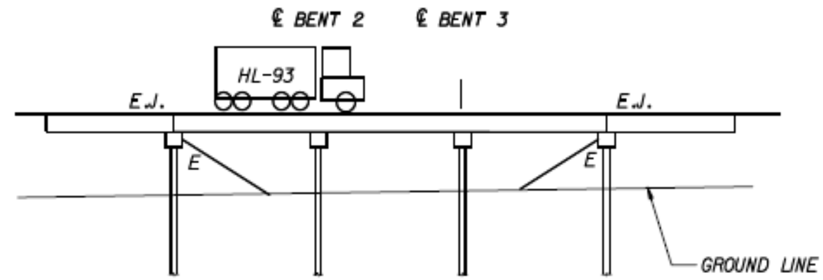
The influence line coordinates for a uniform load applied on a structure is utilized.

Bridge length	105 ft
Bridge width	89.1 ft
# of Traffic Barriers	2
# of Median Barriers	1
No. of spans	3
End span lengths	35 ft
Interior span lengths	35 ft
Concrete weight (DC)	0.150 kcf
Traffic railing barrier (DC)	0.418 klf
Median barrier (DC)	0.483 klf
Wearing surface and/or fws (DW)	0.015 ksf
Barriers and median (DC)	0.0148 ksf
Bridge slab (DC)	0.225 ksf
Additional Misc. loads (DC)	0.0 ksf
Components and Attachments	0.240 ksf
Span ratio	1.0

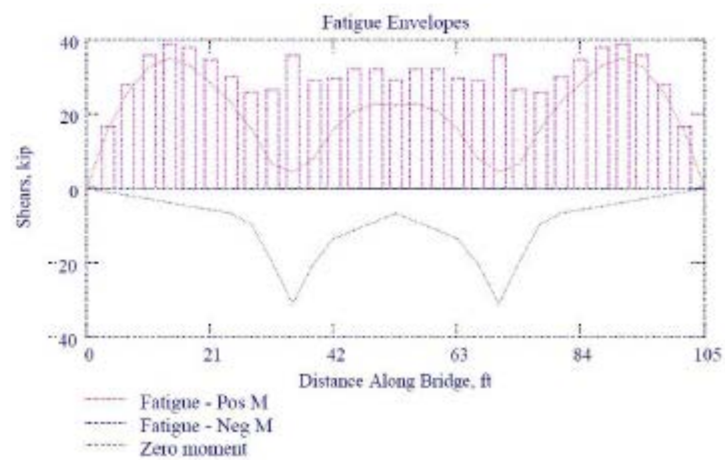


B. Live Load Analysis

The live load moments and shears are calculated with the FDOT Mathcad program “LRFD Live Load Generator, v2.1”



Results of the live load analysis are presented below:



Load Envelope

Design Example: Flat Slab – Traditional

NOTES

Calculate Negative Moment Capacity

$$\alpha_1 := \text{if}\left[f'_{c.super} \le 10ksi, 0.85, \max\left[0.75, 0.85 - 0.02\cdot\left(\frac{f'_{c.super}}{ksi} - 10\right)\right]\right] = 0.9$$

LRFD 5.6.2.2

$$\beta_1 := \begin{vmatrix} ans \leftarrow 0.85 \\ ans \leftarrow ans - \left(\frac{f'_{c.super} - 4\cdot ksi}{1\cdot ksi}\cdot 0.05\right) \text{ if } f'_{c.super} > 4\cdot ksi \\ ans \leftarrow 0.65 \text{ if } ans < 0.65 \\ ans \end{vmatrix} = 0.83$$

LRFD 5.6.2.2

Area of primary reinforcement per linear foot

$$A_{f1.slab} := Bar_{BarSize_{slab.pr},0}\cdot in^2\cdot\frac{1ft}{BarSpace_{slab.pr}} = 3.8\cdot in^2$$

Area of GFRP reinforcement per foot of negative moment

$$\rho_{f1.slab} := \frac{A_{f1.slab}}{b\cdot d_{f1.slab}} = 0.02001$$

FRP reinforcement ratio

$$f_{f1.slab} := \sqrt{\frac{(E_f\cdot\varepsilon_{cu})^2}{4} + \frac{0.85\cdot\beta_1\cdot f'_{c.super}}{\rho_{f1.slab}}\cdot E_f\cdot\varepsilon_{cu}} - 0.5\cdot E_f\cdot\varepsilon_{cu} = 46.6\cdot ksi$$

maximum tensile stress in the GFRP

f_f cannot exceed f_{fu}, therefore, must be taken as minimum of design tensile stress and calculated:

$$f_{f1.slab} := \min(f_{f1.slab}, f_{fd.slab.pr}) = 46.6\cdot ksi$$

Calculate the tensile strain and guaranteed design tensile strain

$$\varepsilon_{ft.slab.pr} := \frac{f_{f1.slab}}{E_f} = 0.00716$$

tensile strain

$$\varepsilon_{fd.slab.pr} := \frac{f_{fd.slab.pr}}{E_f} = 0.00833$$

guaranteed design tensile strain

NOTES

The failure mode depends on the amount of FRP reinforcement. If the computed FRP stress, f_f, is less than the design FRP stress, f_{fd}, then concrete crushing is the failure mode. If f_f is larger than design tensile strength, f_{fd}, then FRP rupture is the failure mode.

The stress-block is computed as per Eq. 2.6.3 whether $\varepsilon_{ft} \le \varepsilon_{fd}$ or $\varepsilon_{fd} > \varepsilon_{ft}$

$$a_{f1.slab} := \begin{cases} \beta_1 \cdot \dfrac{\varepsilon_{cu}}{\varepsilon_{cu} + \varepsilon_{fd.slab.pr}} \cdot d_{f1.slab} & \text{if } \varepsilon_{ft.slab.pr} \le \varepsilon_{fd.slab.pr} \\ \dfrac{A_{f1.slab} \cdot f_{f1.slab}}{0.85 \cdot f'_{c.super} \cdot b_{slab}} & \text{otherwise} \end{cases} = 3.5 \cdot in$$

[GFRP 2.6.3.2.2]

Locate axis depth c_b at balanced strain conditions

$$c_{f1.slab} := \frac{a_{f1.slab}}{\beta_1} = 4.2 \cdot in$$

[GFRP 2.6.7]

The nominal moment capacity is:

$$M_{n.1.slab} := \begin{cases} A_{f1.slab} \cdot f_{f1.slab} \cdot \left(d_{f1.slab} - \dfrac{a_{f1.slab}}{2} \right) & \text{if } f_{f1.slab} < f_{fd.slab.pr} \\ A_{f1.slab} \cdot f_{fd.slab.pr} \cdot \left(d_{f1.slab} - \dfrac{a_{f1.slab}}{2} \right) & \text{otherwise} \end{cases}$$

$$M_{n.1.slab} = 208.9 \cdot kip \cdot ft$$

Compute the resistance factor for flexural strength

$$\phi_{1.slab} := \begin{cases} 0.75 & \text{if } \varepsilon_{ft.slab.pr} \le 0.80 \cdot \varepsilon_{fd.slab.pr} \\ \left(1.55 - \dfrac{\varepsilon_{ft.slab.pr}}{\varepsilon_{fd.slab.pr}} \right) & \text{if } 0.80 \cdot \varepsilon_{fd.slab.pr} < \varepsilon_{ft.slab.pr} < \varepsilon_{fd.slab.pr} \\ 0.55 & \text{otherwise} \end{cases}$$

$$\phi_{1.slab} = 0.69$$

Design flexural resistance is computed as:

$$M_{r.1.slab} := \phi_{1.slab} \cdot M_{n.1.slab} = 144.1 \cdot kip \cdot ft$$

Check Primary Reinforcement Moment Capacity for Strength I

$$D/C{:}Moment_{1.slab} := \frac{M_{str1.neg}}{M_{r.1.slab}} = 0.65$$

2.3. Design Example: Creep Rupture

Refer to Annex A (Section D)

NOTES

D1. Data recall (section B of chapter 1.04)

$C_c = 0.3$ — *AASHTO GFRP 2.5.3*

$f_{f.creep} := C_c \cdot f_{fd.slab.pr} = 16.2 \cdot ksi$ — creep rupture limit stress

$Dia_{slab.pr} = 1.27 \cdot in$ — diameter of slab primary GFRP reinforcement

$Area_{slab.pr} = 1.27 \cdot in^2$ — area of slab primary GFRP reinforcement

$E_f = 6500 \cdot ksi$ — elastic modulus of slab primary GFRP reinforcement

$f_{f1.slab} = 46.6 \cdot ksi$ — tensile strength of slab primary reinforcement

$f_{fd.slab.pr} = 54.1 \cdot ksi$ — design strength of slab primary reinforcement

D2. Support

The stress level in the GFRP reinforcement for checking creep rupture failure is evaluated considering the total unfactored dead loads and a portion of the live load.

$M_{1.creep.slab} := M_{fatigue.neg} = 50.7 \cdot kip \cdot ft$

The load combination for creep rupture limit state includes moment due to dead load plus 0.2 times live load. The previous structural analysis did not calculate this limit state and a representative moment from the fatigue limit state is used for this example.

The tensile stress in GFRP is:

$$n := \frac{E_f}{E_{c.super}} = 1.6$$

$$k_{1.slab} := \sqrt{2 \cdot \rho_{f1.slab} + (\rho_{f1.slab} \cdot n)^2} - \rho_{f1.slab} \cdot n = 0.2$$

$$I_{cr1.slab} := \frac{b \cdot d_{f1.slab}^3}{3} \cdot k_{1.slab}^3 + n \cdot A_{f1.slab} \cdot (d_{f1.slab} - k_{1.slab} \cdot d_{f1.slab})^2 = 1108 \cdot in^4$$

$$f_{f1.creep} := \frac{n \cdot d_{f1.slab} \cdot (1 - k_{1.slab})}{I_{cr1.slab}} \cdot M_{1.creep.slab} = 11.3 \cdot ksi$$

NOTES

$Check_{creep.rupture.1} := \begin{cases} \text{"VERIFIED"} & \text{if } f_{f1.creep} \le C_c \cdot f_{fd.slab.pr} \\ \text{"NOT VERIFIED"} & \text{otherwise} \end{cases}$

$Check_{creep.rupture.1} = \text{"VERIFIED"}$

D3. Middle Span

$M_{2.creep.slab} := M_{fatigue.pos} = 46.7 \cdot kip \cdot ft$ bending moment due to dead load

Ratio of depth of neutral axis to reinforcement depth

$k_{2.slab} := \sqrt{2 \cdot \rho_{f2.slab} + (\rho_{f2.slab} \cdot n)^2} - \rho_{f1.slab} \cdot n = 0.2$

$I_{cr2.slab} := \frac{b \cdot d_{f2.slab}^3}{3} \cdot k_{2.slab}^3 + n \cdot A_{f2.slab} \cdot (d_{f2.slab} - k_{2.slab} \cdot d_{f2.slab})^2 = 1108 \cdot in^4$

$f_{f2.creep} := \frac{n \cdot d_{f2.slab} \cdot (1 - k_{2.slab})}{I_{cr2.slab}} \cdot M_{2.creep.slab} = 10.4 \cdot ksi$

$Check_{creep.rupture.2} := \begin{cases} \text{"VERIFIED"} & \text{if } f_{f2.creep} \le C_c \cdot f_{fd.slab.pr} \\ \text{"NOT VERIFIED"} & \text{otherwise} \end{cases}$

$Check_{creep.rupture.2} = \text{"VERIFIED"}$

2.4. Design Example: Minimum Flexural Reinforcement

Refer to Annex A (Section E)

The minimum flexural reinforcement requirement prevents a sudden failure upon exceeding ultimate loading.

$f_r := 0.24 \cdot \sqrt{f'_{c.super} \cdot (ksi)} = 0.51 \cdot ksi$ modulus of rupture *LRFD 5.4.2.6*

$S_r := \frac{t_{slab}^{2}}{6} \cdot b = 648 \cdot in^{3}$ section modulus at base for calculation of cracking moment using f_r

$M_{cr.slab} := 1.6 \cdot f_r \cdot S_r = 44 \cdot kip \cdot ft$ cracking moment of slab *GFRP LRFD 2.6.3.3*

$M_{min.slab} := \min(1.33 \cdot M_{str1.pos}, M_{cr.slab}) = 44 \cdot kip \cdot ft$ minimum required factored flexural resistance

$M_{r.slab} := \min(M_{r.1.slab}, M_{r.2.slab}) = 142.1 \cdot kip \cdot ft$ flexural capacity of slab

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

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

3. SHEAR RESPONSE OF GFRP-RC

NOTES

3.1. Review Questions: Fundamentals

3.1.1) When checking the capacity for shear of member designed with GFRP reinforcement, the maximum capacity requirement for steel RC still applies: ____________.

a. True
b. False

3.1.2) The shear strength of GFRP-RC members: ______________.

a. Is comparable to the shear strength of members reinforced with steel.
b. Is a lower than the shear strength of members reinforced with steel.
c. Is higher than to the shear strength of members reinforced steel bars.
d. Cannot be compared to the shear strength of members reinforced steel bars.

3.1.3) The required tail length of GFRP stirrups is at least equal to or more than: ___________________.

a. 4 times the bar diameter
b. 8 times the bar diameter
c. 12 times the bar diameter
d. 16 times the bar diameter

3.1.4) GFRP stirrups can be bent on site with EOR approval?

a. True
b. False

NOTES

3.1.5) The minimum bent radius allowed for a GFRP stirrup is generally __________. (Select all that apply)

a. Larger than required for steel, with a minimum of $r_b/d_b = 3$
   Can be equivalent to steel, if verified by manufacturer
b. Smaller than required for steel reinforcement due to lower elastic modulus
c. Dependent on field bending and cannot be prescribed

3.1.6) When designing GFRP shear reinforcement, the following shapes are possible to manufacturer: _________. (Select all that apply)

a. Two C's
b. Two U's
c. Closed stirrup, providing the tails overlap
d. L shapes for end hooks
e. Special bends for complex shapes

3.1.7) The maximum spacing of transverse GFRP reinforcement is generally _________.

a. 12 in.
b. 24 in.
c. 0.5d
d. Minimum value of 0.5d* or 24 in.

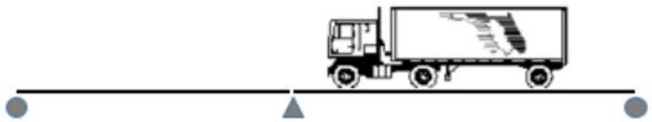
** Flexural reinforcement depth*

3.2. Design Example: Bent Cap

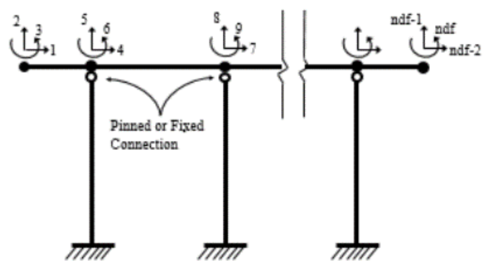
NOTES

Refer to Annex B consisting of three parts:

PART 1: LOAD GENERATOR

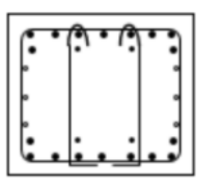


PART 2: FRAME ANALYSIS

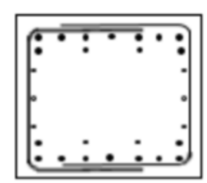


Bent Cap Analysis Model

PART 3: DESIGN & AASHTO CHECKS



Steel Rebar



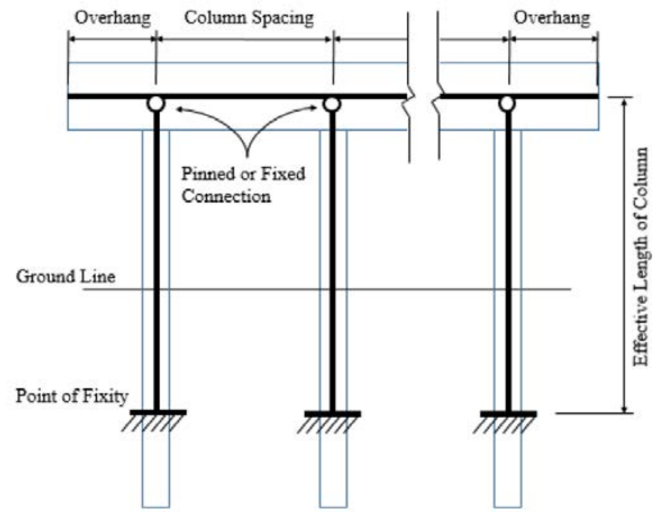
GFRP

PART 1: LOAD GENERATOR

Input Data

Superstructure

Beam properties	21.3x24.4 in
Beam weight	181 plf
Cap skew	0 Degrees
Average haunch thickness	1.14 in
Barrier height	32 inches
Barrier weight per SDG	420 lb/ft
Slab thickness (including sacrificial wearing)	8.5 in.
Length of back station span	37.17 feet
Length of ahead station span	37.17 feet
Total number of Beams in Typical Section	9
Centerline-to-centerline beam spacing	6.63 feet
Curb-to-curb roadway width	40 feet
Distance from coping to roadway edge	10.56 feet
Dead load of wearing surfaces and utilities	Exterior Beam 165 lb/ft Interior Beam 150 lb/ft
Additional dead load of structural components and nonstructural attachments (i.e. SIP forms)	Exterior Beam 20 lb/ft Interior Beam 20 lb/ft



Bent Cap Model

Substructure

Number of columns or piles	6
Effective length of columns	33.7 feet
Column spacing	10.38 feet
Column type	Square
Column Diameter/Width	18 inches
Cap Height	36 inches
Cap Width	48 inches
Cap Length	59.9 feet
Average pedestal height	4 inches
Pedestal Width	30 inches
Pedestal Length	34 inches
Beam bearing pad Length	22 inches
Min.28-day compressive strength for cap	5.5 ksi
Min.28-day compressive strength for columns	6 ksi
Correction factor for source of aggregate	1
Concrete unit weight for calculating Ec	0.145 kcf
Concrete unit weight for calculating dead loads	0.150 kcf

NOTES

Additional Input for Centrifugal Force (CE)

Radius of curvature of traffic lane	0 feet
Highway design speed	50 mph
Distribution Factor for CE load to intermediate bent cap	1

Additional Input for Braking Force (BR)

Distribution Factor for BR load to intermediate bent cap	1
Length of bridge for BR load calculation (length of lane load)	185.83 feet

Additional Input for Wind on Structure (WS)

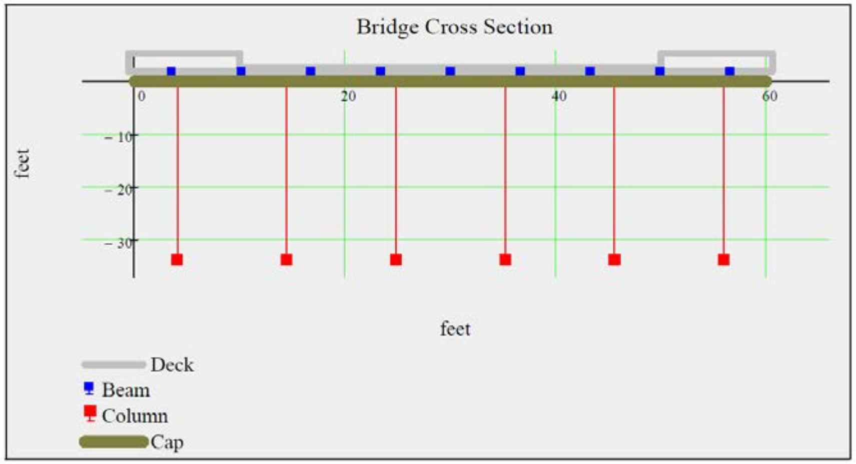
Low Member elevation	8.10 feet
Elevation of low ground or water level	0.06 feet
Design Wind Speed	130 mph
Total depth of superstructure (barrier, deck, haunch, beam and superelevation)	5.24 feet

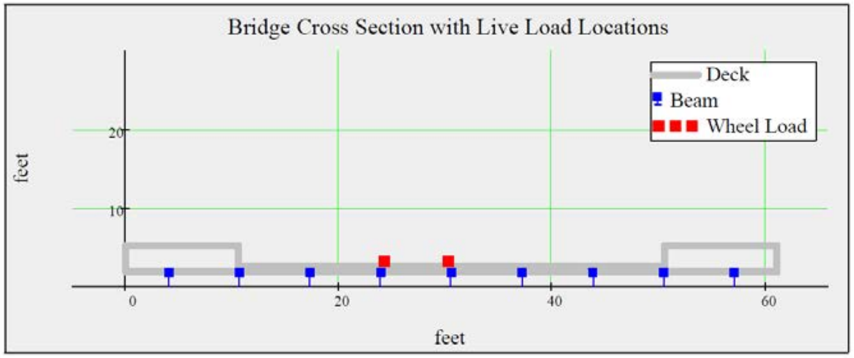
Additional Input for Water Load (WA)

100-year event: parallel to the bent-cap	0 kip
100-year event: perpendicular to the bent-cap	0 kip
500-year event: parallel to the bent-cap	0 kip
500-year event: perpendicular to the bent-cap	0 kip

Additional Input for Force Effect due to Uniform Temperature (TU)

TU load in the longitudinal direction of the bridge	0 kip


Bridge Cross Section
0
20
40
60
– 10
– 20
– 30
feet
feet
Deck
Beam
Column
Cap

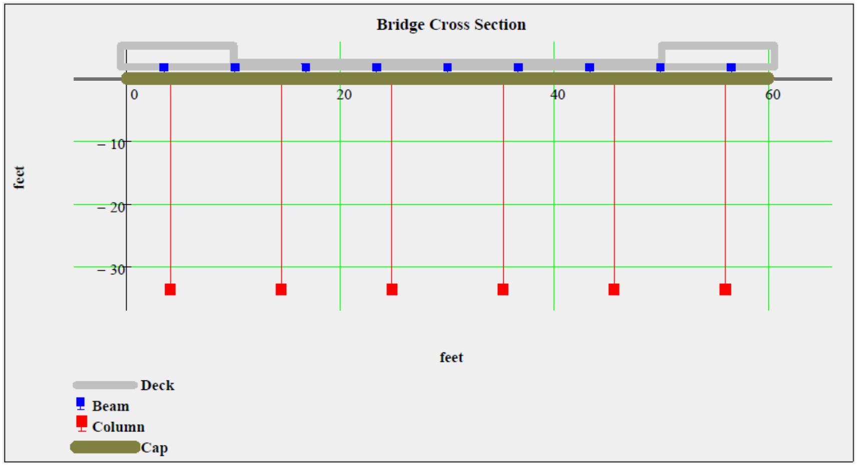

Bridge Cross Section with Live Load Locations
Deck
Beam
Wheel Load
20
10
0
20
40
60
feet
feet

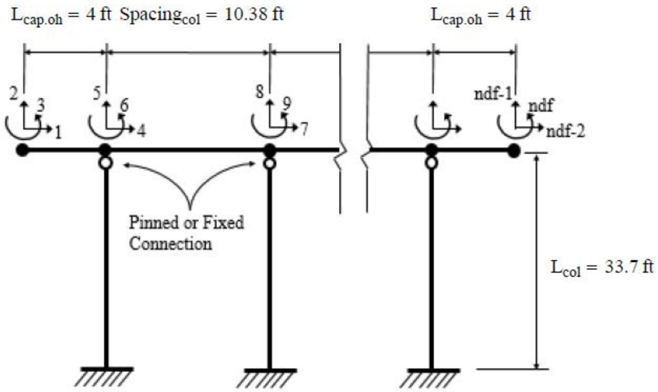
NOTES

PART 2: FRAME ANALYSIS

NOTES

Connection of columns to bent cap (Fixed or Pinned)	Fixed
Beam load	Distributed



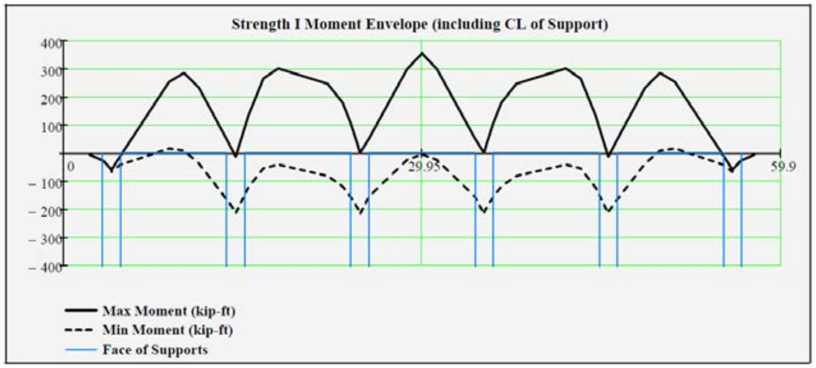


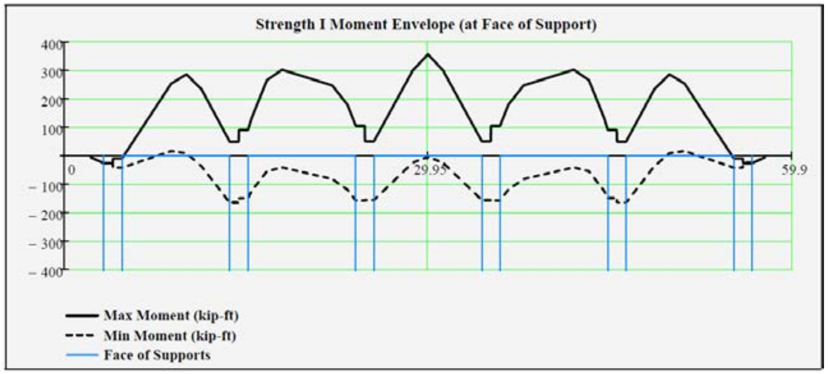
Bent Cap Analysis Model

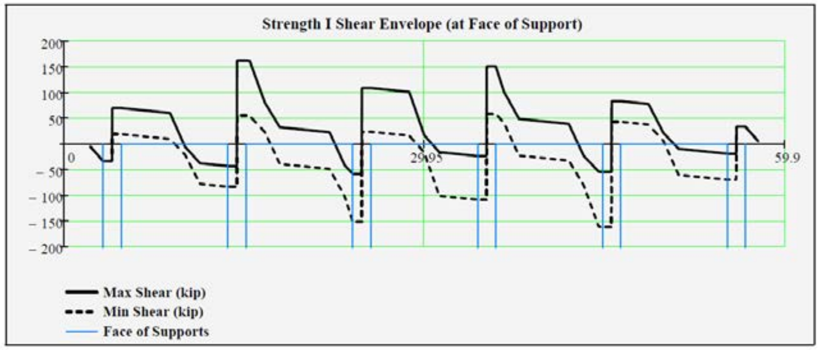
Summary Results

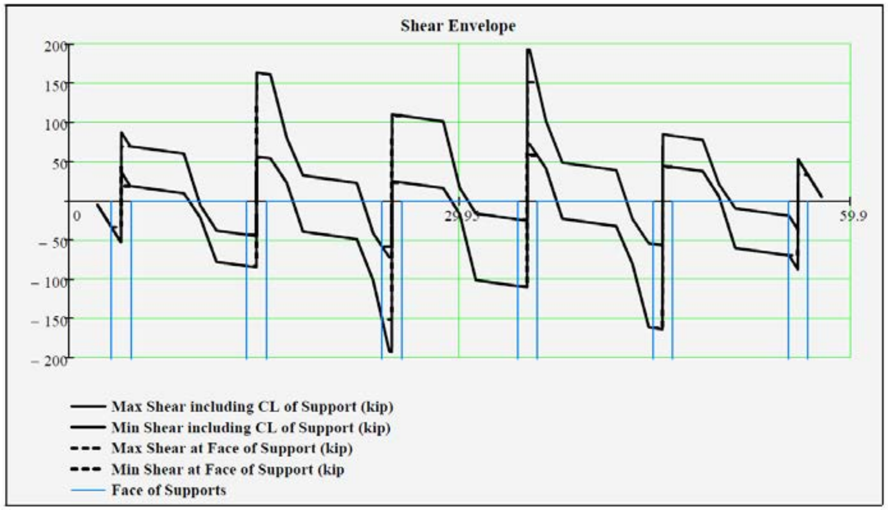
NOTES

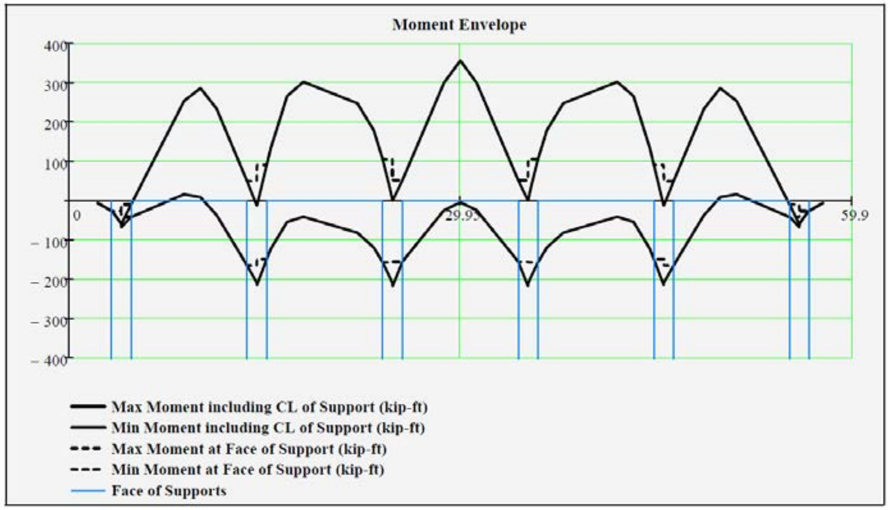
Limit State Strength I (max vertical load)





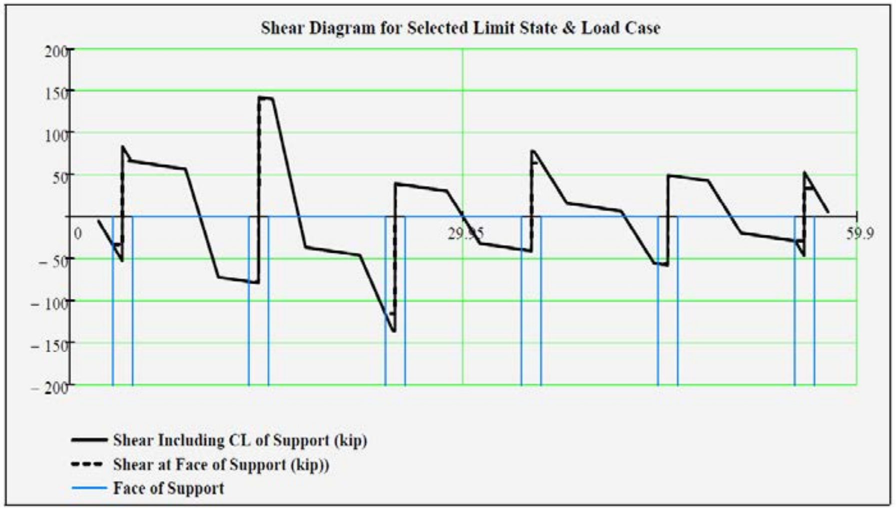


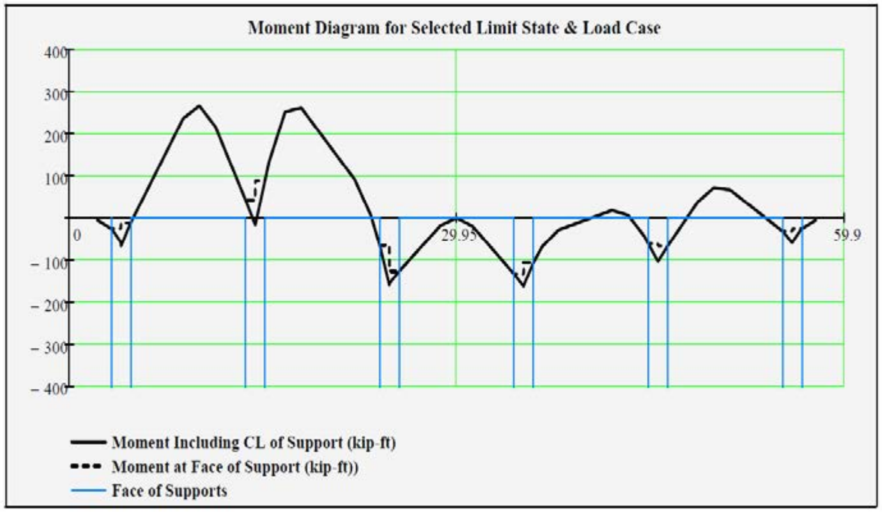

Shear Envelope
200
150
100
50
0
29.95
59.9
– 50
– 100
– 150
– 200
Max Shear including CL of Support (kip)
Min Shear including CL of Support (kip)
Max Shear at Face of Support (kip)
Min Shear at Face of Support (kip
Face of Supports


Moment Envelope
400
300
200
100
0
29.95
59.9
– 100
– 200
– 300
– 400
Max Moment including CL of Support (kip-ft)
Min Moment including CL of Support (kip-ft)
Max Moment at Face of Support (kip-ft)
Min Moment at Face of Support (kip-ft)
Face of Supports

NOTES

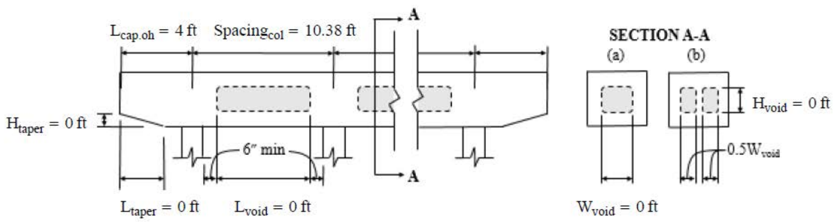
NOTES


Shear Diagram for Selected Limit State & Load Case
200
150
100
50
0
29.95
59.9
− 50
− 100
− 150
− 200
Shear Including CL of Support (kip)
Shear at Face of Support (kip))
Face of Support


Moment Diagram for Selected Limit State & Load Case
400
300
200
100
0
29.95
59.9
− 100
− 200
− 300
− 400
Moment Including CL of Support (kip-ft)
Moment at Face of Support (kip-ft))
Face of Supports

PART 3: GFRP DESIGN & AASHTO CHECKS

NOTES

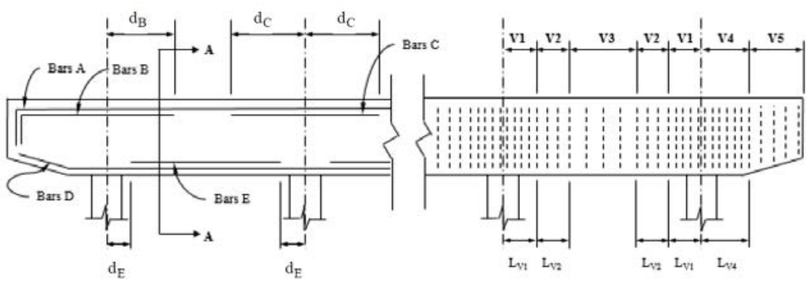


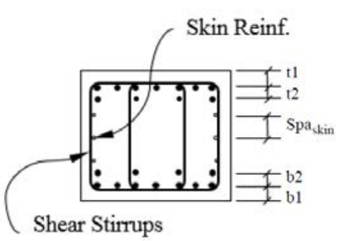
-Design Loads - Moments and Shears (Torques not considered)
-Critical section for shear design should be at face of support.

GFRP Material and Design Properties

Environmental reduction factor (C_E)	0.7
Tensile modulus of elasticity (E_f)	6500 ksi

Flexural reinforcement





SECTION A-A

Top Reinforcement (Negative Moment)

Size of top reinforcing bars (A, B & C)	8
Distance from c.g. of 1st layer bars to cap top face (t_1)	4 in.
Distance from c.g. of 2nd layer to 1st layer bars (t_2)	0

Bottom Reinforcement (Positive Moment)

Size of bottom reinforcing bars (D & E)	8
Distance from 1st layer bars to cap bottom face (b_1)	4 in.
Distance from c.g. of 2nd layer to 1st layer bars (b_2)	0

Bars A: Continuous Top Reinforcement

Number of bars placed in 1st Layer	8
Number of bars placed in 2nd Layer	0

Bars B: Supplemental Top Reinforcement over Exterior Columns

Length of Bars B beyond CL of Exterior Column (d_B)	15 in.
Number of bars placed in 1st Layer	4
Number of bars placed in 2nd Layer	0

Bars C: Supplemental Top Reinforcement Centered on CL of Interior Columns

Length of Bars C beyond CL of Interior Column (d_C)	15 in.
Number of bars placed in 1st Layer	4
Number of bars placed in 2nd Layer	0

Bars D: Continuous Bottom Reinforcement

Number of bars placed in 1st Layer	8
Number of bars placed in 2nd Layer	0

Bars E: Supplemental Bottom Reinforcement Centered on Interior Spans

Distance from CL of column to end of Bars E (d_E)	15 in.
Number of bars placed in 1st Layer	4
Number of bars placed in 2nd Layer	0

Spacing of Flexural Reinforcement

Concrete cover on the two sides	3 in.

Shear Reinforcement

Zone V1

Size of stirrup bar	5
No. of bar legs	5
Spacing	5 in.
Length of Zone V1 (L_{v1})	36 in.

Zone V2

Size of stirrup bar	5
No. of bar legs	5
Spacing	5 in.
Length of Zone V2 (L_{v2})	36 in.

Zone V3

Size of stirrup bar	5
No. of bar legs	5
Spacing	5 in.

Zone V4 (Cap overhang)

Size of stirrup bar	5
No. of bar legs	5
Spacing	5 in.
Length of Zone V4 (L_{v4})	36 in.

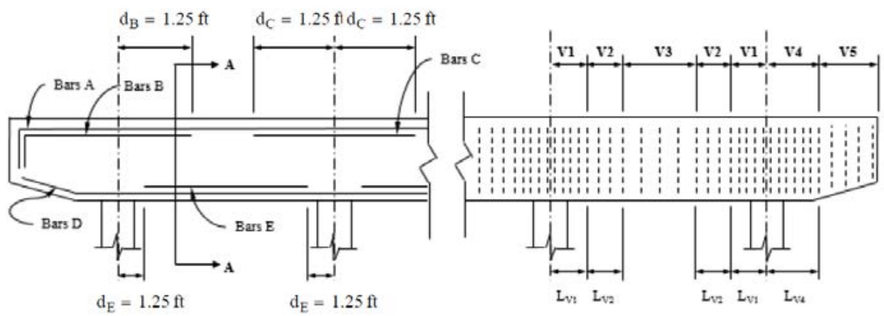
Zone V5 (Cap overhang)

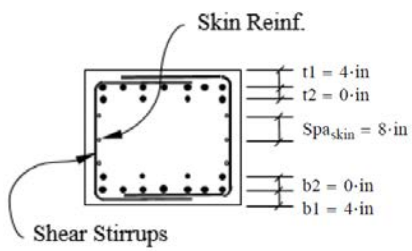
Size of stirrup bar	5
No. of bar legs	5
Spacing	5 in.

Skin Reinforcement

NOTES

Size of bar	5
Number of bars on each side face	2
Spacing of skin reinforcement	8 in.

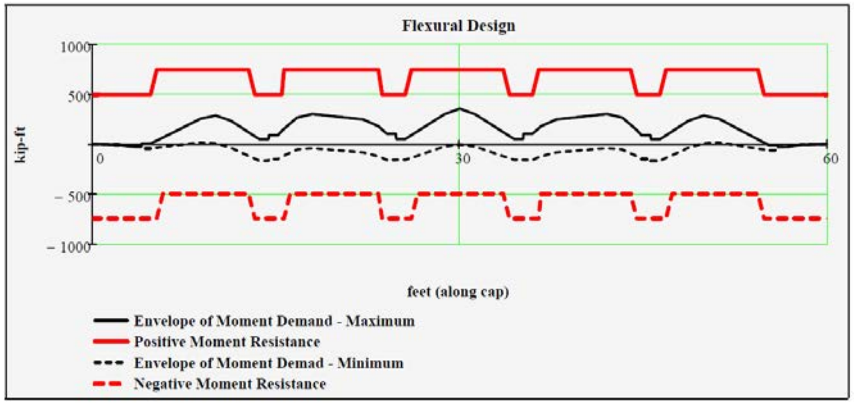




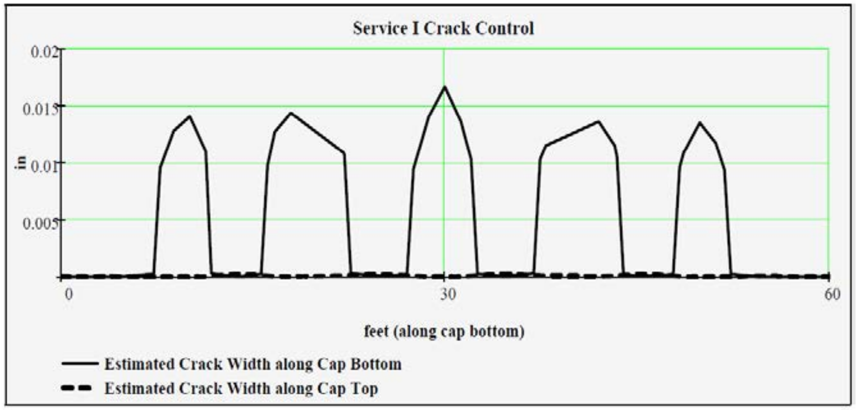
SECTION A-A

Flexural Design [AASHTO BDS for GFRP 2.6]

NOTES

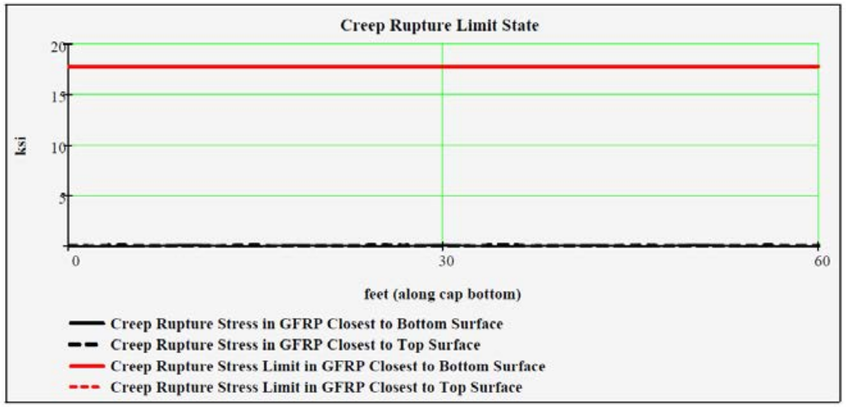


Crack Control [AASHTO BDS for GFRP 2.6.7]

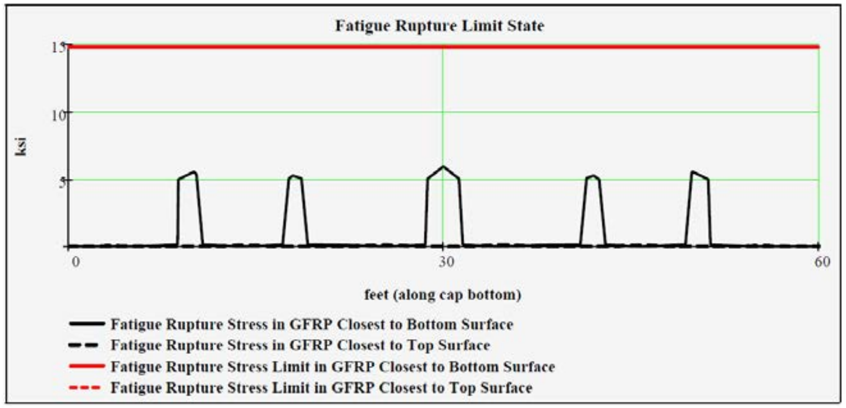


Creep Rupture Limit State [AASHTO BDS for GFRP 2.5.3]

NOTES



Fatigue Limit State [AASHTO BDS for GFRP 2.5.4]



Shear Design [AASHTO BDS for GFRP 2.7]

NOTES

Nominal Shear Resistance of the Concrete

$V_c = 0.0316\beta\sqrt{f_c'}b_v d_v$

β=5 (for concrete sections not subjected to axial tension Article 2.7.2.4)

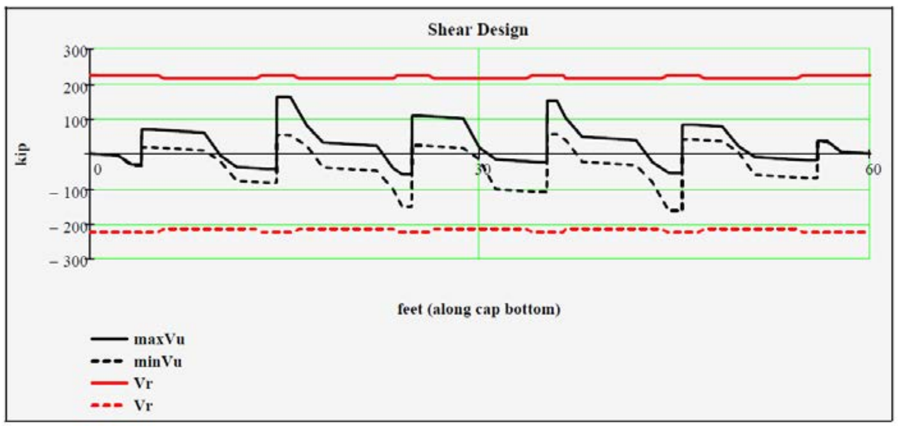
Max (V_c) = 64.75 kip

Min (V_c) = 53.52 kip

Shear Resistance by Transverse Reinforcement

$V_u \leq V_r$

$DCR = \frac{V_u}{V_r} = 0.72$



Check Spacing of stirrups

$S \leq Min\{0.5d\ ,\ 24in\}$

Summary of LRFD and SDG Checks

NOTES

Save Data

Positive Moment

$Check_{Mr.pos}$ = "OK"

$max(DCR_{M.pos}) = 0.48$

$Check_{minAf.bot}$ = "OK"

$Check_{crack.control.bot}$ = "OK"

$max(CrackW_{SerI.bot}) = 0.017 \cdot in$

$Check_{creep.bot}$ = "OK"

$max(f_{f.SL.bot}) = 0.11 \cdot ksi$

$Check_{fatigue.bot}$ = "OK"

$max(f_{f.Fat.bot}) = 5.94 \cdot ksi$

$C_f \cdot f_{fd.pos} = 14.8 \cdot ksi$

Negative Moment

$Check_{Mr.neg}$ = "OK"

$max(DCR_{M.neg}) = 0.24$ *The maximum demand to capacity ratio*

$Check_{minAf.top}$ = "OK"

$Check_{crack.control.top}$ = "OK"

$max(CrackW_{SerI.top}) = 0.000 \cdot in$ *The maximum crack width*

$Check_{creep.top}$ = "OK"

$max(f_{f.SL.top}) = 0.08 \cdot ksi$ *The maximum stress under sustained load (DL+0.2LL)*

$Check_{fatigue.top}$ = "OK"

$max(f_{f.Fat.top}) = 0.11 \cdot ksi$ *The maximum stress under fatigue (DL+1.5LL fatigue)*

$C_f \cdot f_{fd.neg} = 14.8 \cdot ksi$ *Fatigue stress limit*

Shear Checks

$Check_{V.r}$ = "OK"

$max(DCR_V) = 0.72$ *The maximum demand to capacity ratio*

$Check_{A.v.min}$ = "OK"

$Check_{shear.spa}$ = "OK"

$CriticalSpa_{reqd.shear} = 14.5 \cdot in$ *The allowable spacing for shear reinforcement at the most critical cap section*

Skin Reinforcement

$Check_{AreaSkinReinf}$ = "Skin Reinf Not Required"

$A_{skin.reqd} = 0.00 \cdot in^2$

$Check_{SpaSkinReinf}$ = "Skin Reinf Not Required"

$Spa_{skin.reqd} = 0.00 \cdot in$

Shrinkage and Temperature Reinforcement

$Check_{AreaShrinkReinf}$ = "OK"

$A_{shrink.reqd} = 0.44 \cdot \frac{in^2}{ft}$

$Check_{SpaShrinkReinf}$ = "OK"

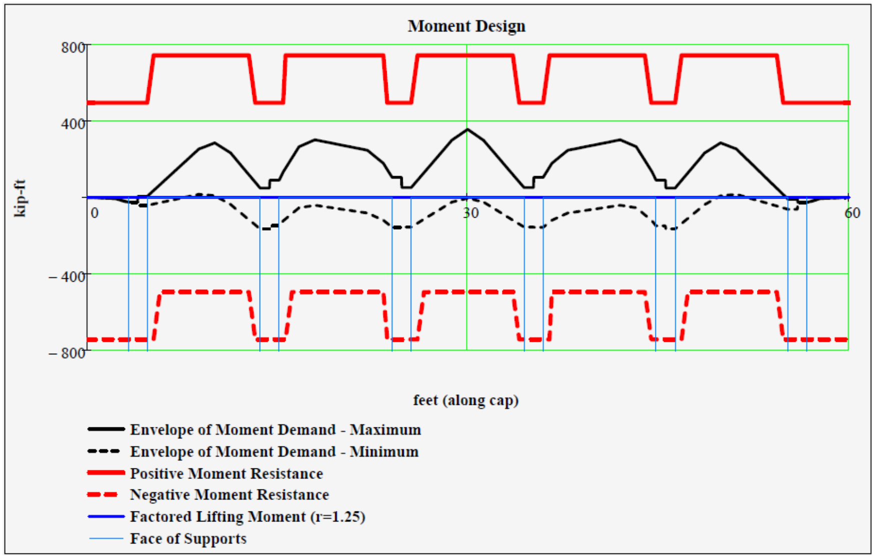
$Spa_{shrink.reqd} = 12.00 \cdot in$

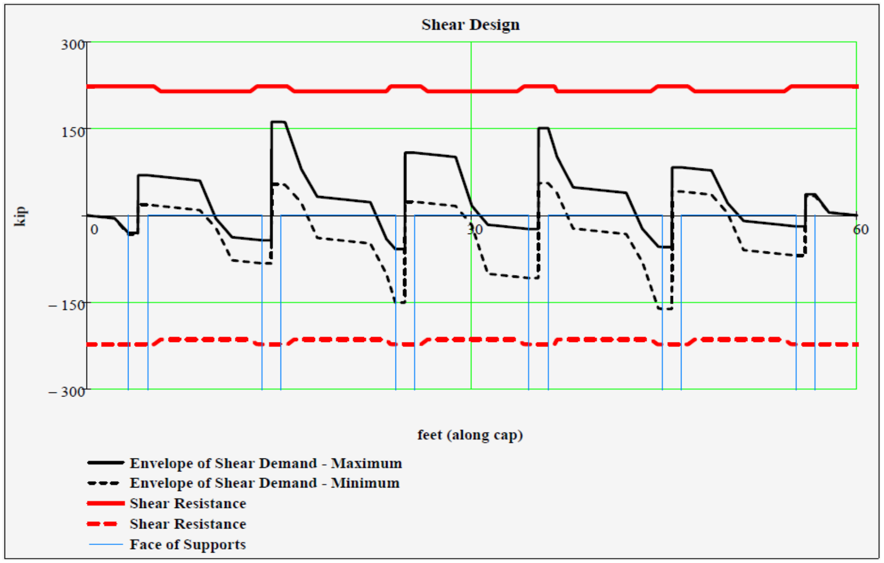
Lifting Checks

$Check_{M.u.lifting}$ = "Not Applicable"

Mass concrete requirements

$SDG_{3.9}$ = "Use regular concrete provisions"


Moment Design
800
400
0
30
60
– 400
– 800
kip-ft
feet (along cap)
Envelope of Moment Demand - Maximum
Envelope of Moment Demand - Minimum
Positive Moment Resistance
Negative Moment Resistance
Factored Lifting Moment (r=1.25)
Face of Supports


Shear Design
300
150
0
30
60
– 150
– 300
kip
feet (along cap)
Envelope of Shear Demand - Maximum
Envelope of Shear Demand - Minimum
Shear Resistance
Shear Resistance
Face of Supports

NOTES

4. AXIAL RESPONSE OF GFRP-RC

NOTES

Refer to Annex C: LRFD Design P-M Diagram of GFRP Reinforced Pile Example.

4.1. Review Questions: Fundamentals

4.1.1) The compressive capacity of GFRP reinforcement _____________.

a. Is used to improve ductility
b. Is used to satisfy maximum strain requirements of AASTHO
c. Is only considered for low loads
d. Can be used in design up until a concrete strain of 0.003

4.1.2) Compared to steel reinforced columns, the transverse spacing requirements for GFRP reinforced columns are the same.

a. True
b. False

4.1.3) Compared to steel reinforced columns, the ultimate tensile capacity is reduced.

a. True
b. False

NOTES

4.1.4) When constructing a moment-interaction diagram for a GFRP reinforced column, the balance points refers to: _______________.

a. The point at which the GFRP reinforcement yields and concrete crushes
b. The point at which the GFRP reinforcement ruptures and concrete crushes
c. The point at which the GFRP reinforcement yields before concrete crushes
d. The point at which the concrete crushes, but the GFRP reinforcement has not ruptured.

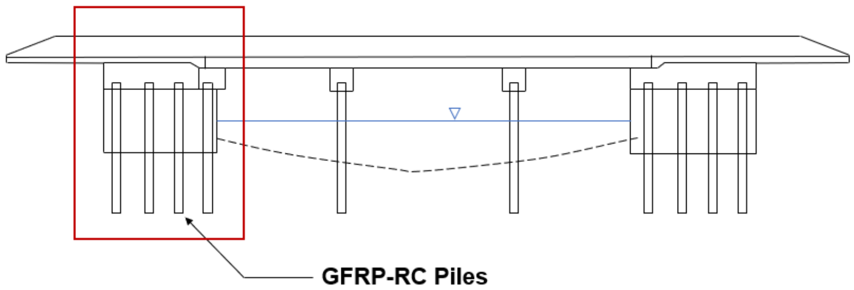
4.1.5) When designing a slender GFRP reinforced column, special considerations should be made to determining slenderness ratio and EI?

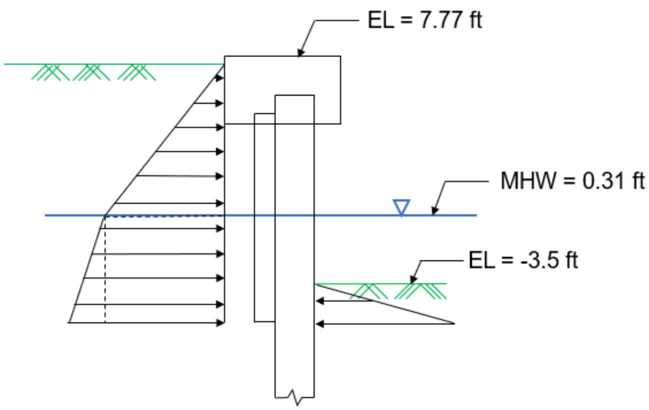
a. True
b. False

4.2. Design Example: Solider Pile in Wing Wall

NOTES

Consider the following example:





Cross Section

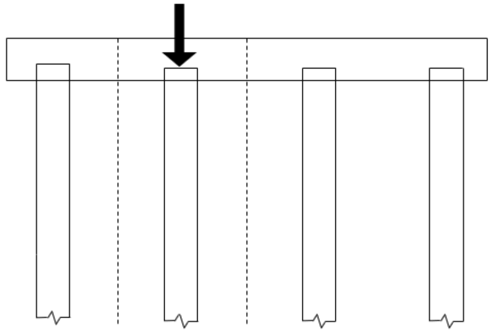
<u>Soil Properties</u>

$\gamma = 110\ pcf$

$\phi = 30\ degrees$

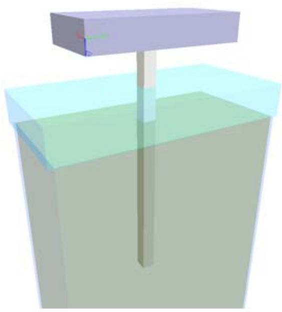
$c = 0\ ksf$

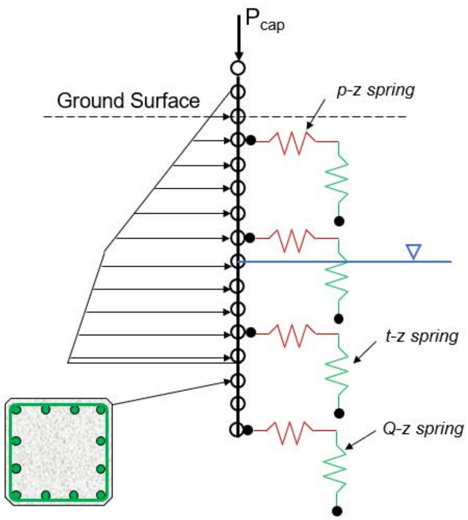
$$P_{cap} = Pile\ Spacing \times Cap_{Height} \times Cap_{Width} \times \gamma_c$$



$$Distributed\ Force = Resultant\ Earth\ Pressure \times Pile\ Spacing$$

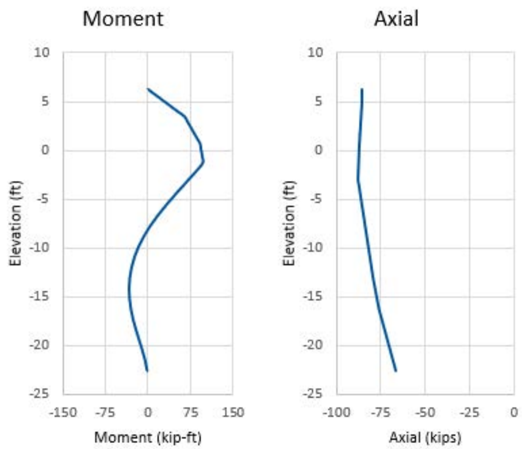
Laterally Loaded Pile Model





NOTES

NOTES



Max:	97.4 kip-ft	87.7 kip
Min:	-33.4 kip-ft	-63.0 kip