LRFD Design Example

(from 2020)

Cast-in-Place Flat Slab Design with GFRP Reinforcement

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Preface

Given the document "LRFD Design Example #2", generated by HDR Engineering, Inc. for the Florida Department of Transportation (FDOT), the work of University of Miami, Department of Civil, Architectural and Environmental Engineering (CAE), is to:

- Edit the given example to account for properties of Glass Fiber Reinforced Polymer (GFRP) bars.
- Implement new algorithms to allow for the design of the GFRP-reinforced bridge superstucture (slab and traffic barrier) and substructure (pile cap).
- Update standards and references.
- Enrich the given example with notes, comments and drawings.

Coral Gables, FL
Januray 10, 2015

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The "LRFD Design Example #2" described above was modified for the Florida Department of Transportation (FDOT) to include the latest provisions in the AASHTO LRFD Bridge Design Guide Specifications for GFRP-Reinforced Concrete, 2nd Edition".

Coral Gables, FL
April 10, 2020

Antonio Nanni, Professor & Chair
Roberto Rodriguez, PhD Student

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Section 1: PROJECT INFORMATION

Section 2: SUPERSTRUCTURE DESIGN

About this Design Example

Description

This document provides guidance for the design of a cast-in-place flat slab bridge with GFRP-reinforcement.

The example includes the following component designs:

- GFRP-reinforced solid CIP slab design

The following assumptions have been incorporated in the example:

- Three span continuous @ 35'-0" each for a total of 105'-0" bridge length
- 30 degree skew
- No phased construction
- Two traffic railing barriers and one median barrier
- No sidewalks
- Permit vehicles are not considered
- Load rating is not addressed

Since this example is presented in a **Mathcad** document, a user can alter assumptions, constants, or equations to create a customized application.

Standards

- [AASHTO LRFD 2017] - AASHTO LRFD Bridge Design Specifications, 8th Edition, 2017
- [AASHTO GFRP 2018] - AASHTO LRFD Bridge Design Guide Specifications for GFRP-Reinforced Concrete Bridge Decks and Traffic Railings, 2nd Edition, 2018
- [FDOT 2020] - Florida Department of Transportation Standard Specifications for Road and Bridge Constructions, 2020
- [FDOT 2020] - Florida Department of Transportation Structures Design Guidelines, 2020
- [SDG 2020] - Structures Design Guidelines, Florida Department of Transportation, January 2020
- [FRPG 2020] - Florida Department of Transportation Fiber Reinforced Polymer Guidelines, Structures Manual Volume 4, January 2020

References

- [ACI 440.1R-15] - Guide for the Design and Construction of Structural Concrete Reinforced with FRP Bars, 2015
- [AC454 2014]-Acceptance Criteria for Glass Fiber–Reinforced Polymer (GFRP) Bars for Internal Reinforcement of Concrete and Masonry Members
- AISC Moment, Shears and Reactions for Continuous Highway Bridges, 1987
- F. Matta and A. Nanni, "Connection of Concrete Railing Post and Bridge Deck with Internal FRP Reinforcements", Journal of Bridge Engineering, ASCE, Vol.14, No.1, January/February 2009

Defined Units

All calculations in this electronic book use U.S. customary units. The user can take advantage of Mathcad's unit conversion capabilities to solve problems in MKS or CGS units. Although Mathcad has several built-in units, some common structural engineering units must be defined. For example, a lbf is a built-in Mathcad unit, but a kip or ton is not. Therefore, a kip and ton are globally defined as:

$kip \equiv 1000 \cdot lbf$ $\quad$ $ton \equiv 2000 \cdot lbf$

Definitions for some common structural engineering units:

$N \equiv newton$ $\quad$ $kN \equiv 1000 \cdot newton$

$plf \equiv \frac{lbf}{ft}$ $\quad$ $psf \equiv \frac{lbf}{ft^2}$ $\quad$ $pcf \equiv \frac{lbf}{ft^3}$ $\quad$ $psi \equiv \frac{lbf}{in^2}$ $\quad$ $MPa \equiv 1 \cdot 10^6 \cdot Pa$

$klf \equiv \frac{kip}{ft}$ $\quad$ $ksf \equiv \frac{kip}{ft^2}$ $\quad$ $ksi \equiv \frac{kip}{in^2}$ $\quad$ $°F \equiv 1 deg$ $\quad$ $GPa \equiv 1 \cdot 10^9 \cdot Pa$

Note: Anytime that a symbol is shown as variable symbol. $xxx := 1$ $xxx := 1$ it is not an error, but it indicates a repetition of

A variable with yellow highlight represents an input from engineer.

Notice

The materials in this document are only for general information purposes. This document is not a substitute for competent professional assistance. Anyone using this material does so at his or her own risk and assumes any resulting liability.

PROJECT INFORMATION

General Notes

Design Method................. Load and Resistance Factor Design (LRFD) except that Prestressed Piles have been designed for Service Load.

Design Loading................. HL-93 Truck

Future Wearing Surface.. Design provides allowance for 15 psf

Earthquake........................ Seismic provisions for minimum bridge support length only **[SDG 2.3.1]**.

Concrete............................

Class	Minimum 28-day Compressive Strength (psi)	Location
II	f'c = 3400	Traffic Barriers
II (Bridge Slab)	f'c = 4500	CIP Flat Slab
IV	f'c = 5500	CIP Substructure
V (Special)	f'c = 6000	Concrete Piling

Environment...................... The superstructure is classified as extremely aggressive. The substructure is classfied as extremely aggressive.

GFRP reinforcement....... The reinforcing bars meet the requirements of AC454 by ICC-ES.

Concrete Clear Cover.....

Superstructure: Short bridge **[SDG 4.2.1]**

Top slab surfaces	1.5"	**[FRPG 2020 2.3]**
Traffic barrier	1.5"	**[FRPG 2020 2.3]**
All other surfaces	1.5"	**[FRPG 2020 2.3]**

Substructure: Short bridge **[SDG 4.2.1]**

External surfaces exposed	2" *	**[FRPG 2020 2.3]**
External surfaces cast against earth	4"	**[FRPG 2020 2.3]**
Prestressed piling	3"	**[FRPG 2020 2.3]**

* The concrete clear cover for pier caps not in contact with water and earth can be set to 1.5 in instead of 2 in [FRPG 2020 2.3] , being GFRP not subject to corrosion.

Concrete cover includes 0.5 in allowance for planing [SDG 4.2.2]

Dimensions........................ All dimensions are in feet or inches, except as noted.

Design Parameters

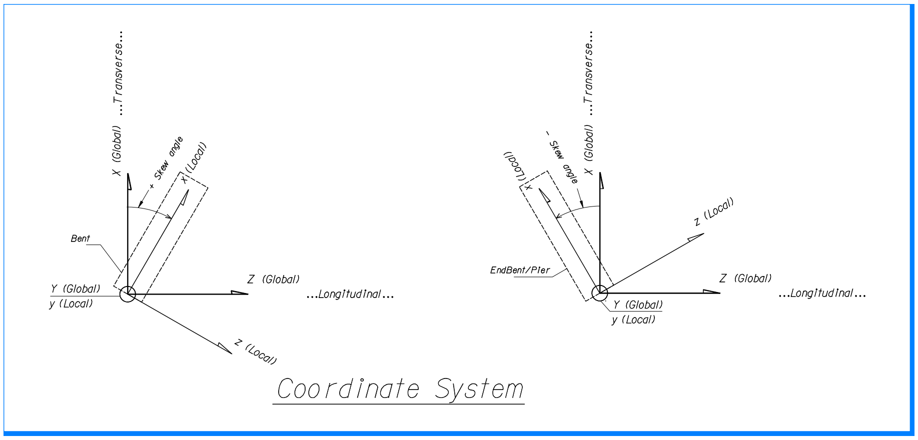
Description

This section provides the design input parameters necessary for the superstructure and substructure design.

Page	Contents

A. General Criteria

This section provides the general layout and input parameters for the bridge example.



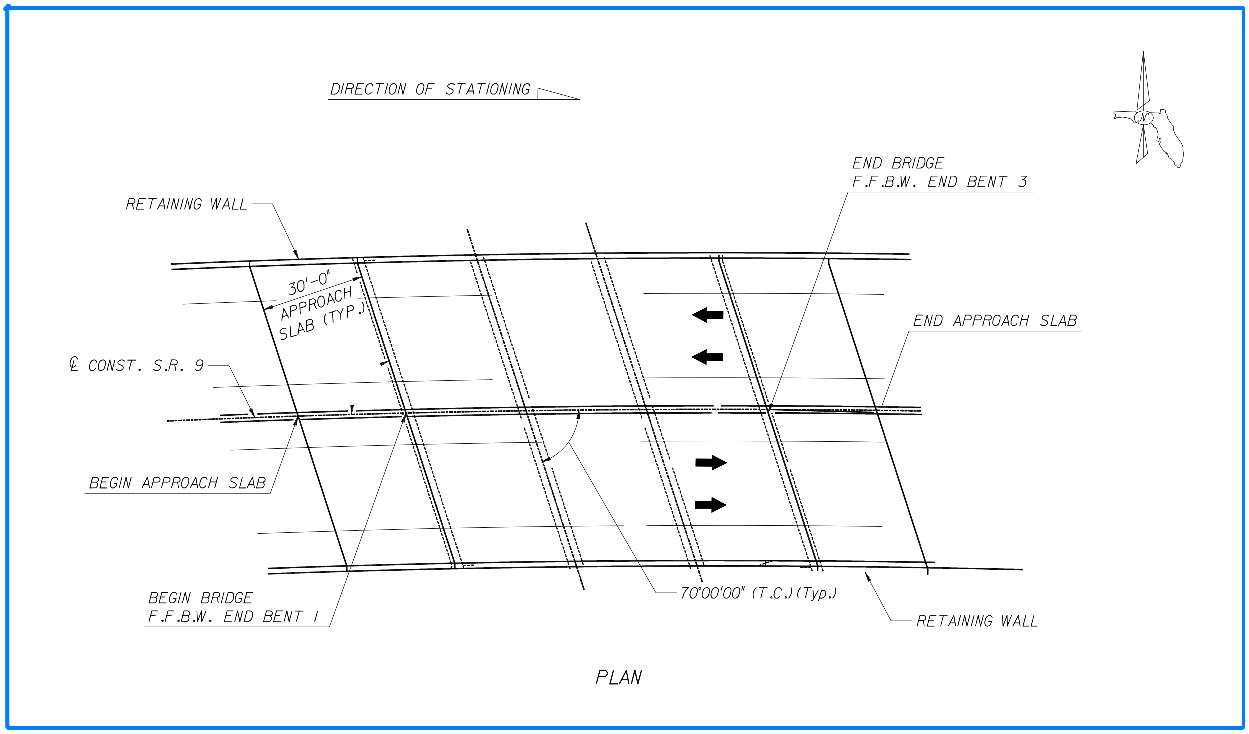
In addition, the bridge is also on a skew which is defined as:

Skew Angle........................ $Skew := -30deg$

A1. Bridge Geometry

Horizontal Profile

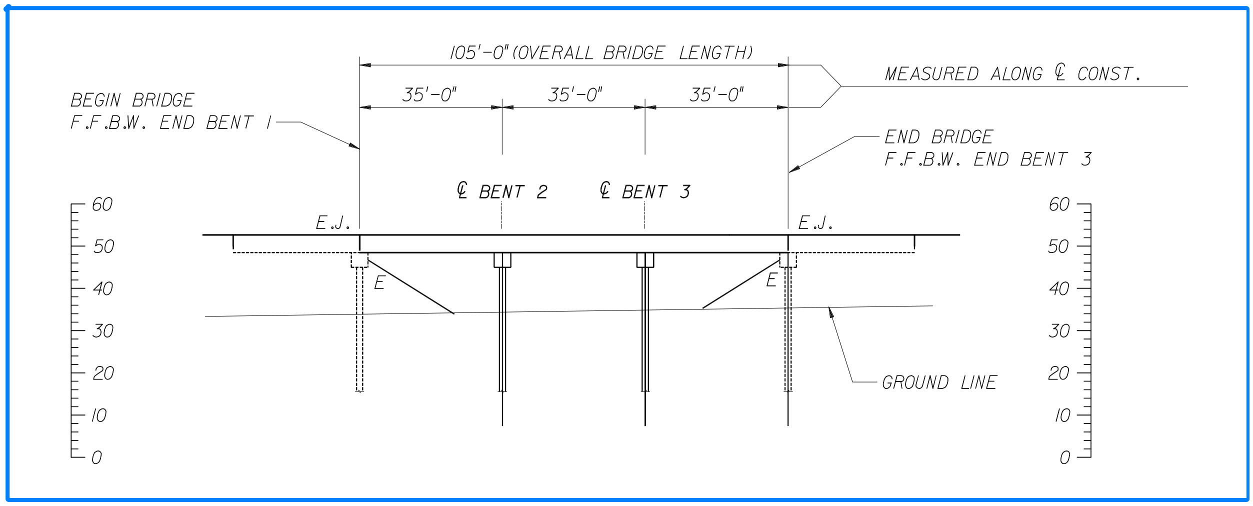
A slight horizontal curvature is shown in the plan view. For all component designs, the horizontal curvature will be taken as zero.



HORIZONTAL CURVE DATA

R = 3,800'

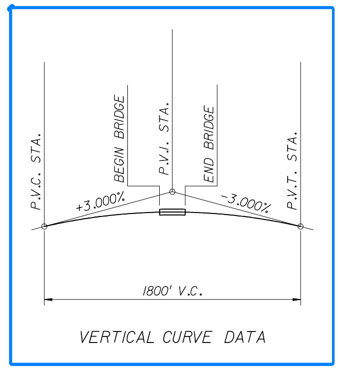
Vertical Profile

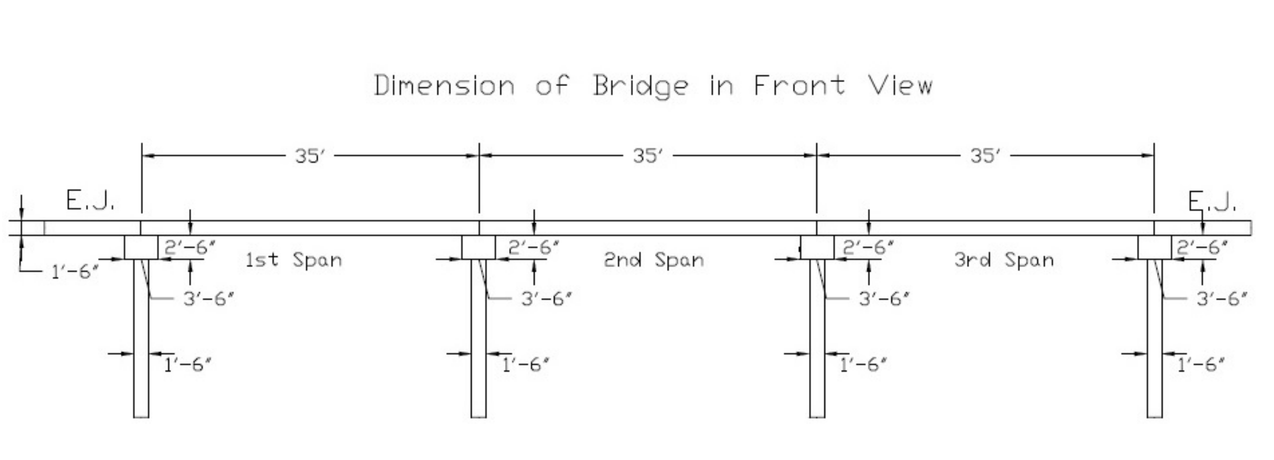


Overall bridge length............ $L_{bridge} \equiv 105 \cdot ft$

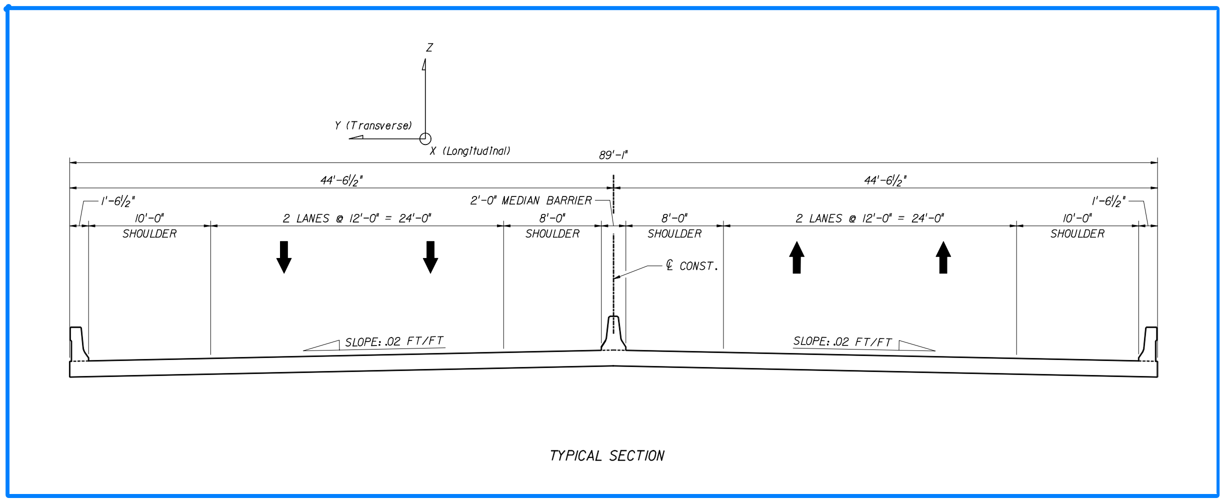
Bridge design span length...... $L_{span} := 35 \cdot ft$

*(**Note**: For unsymmetric spans, use average span length)*





Typical Cross-secton



Overall bridge width............ $W_{bridge} := 89.1 \cdot ft$

A2. Number of Lanes

Design Lanes

Current lane configurations show two striped lanes per roadway with a traffic median barrier separating the roadways. Using the roadway clear width between barriers, $Rdwy_{width}$, the number of design traffic lanes per roadway, N_{lanes}, can be calculated as:

Roadway clear width........... $Rdwy_{width} := 42 \cdot ft$

Number of design traffic lanes per roadway........................ $N_{lanes} := \text{floor}\left(\dfrac{Rdwy_{width}}{12 \cdot ft}\right)$

$N_{lanes} = 3$

A3. Concrete, Reinforcing and Prestressing Steel Properties

Unit weight of concrete......... $\gamma_{conc} := 145 \cdot pcf$

Modulus of elasticity for reinforcing steel.................. $E_s := 29000 \cdot ksi$

B. LRFD Criteria

The bridge components are designed in accordance with the following LRFD design criteria:

B1. Dynamic Load Allowance [AASHTO LRFD 2017, 3.6.2]

An impact factor will be applied to the static load of the design truck or tandem, except for centrifugal and braking forces.

Impact factor for fatigue and fracture limit states............... $IM_{fatigue} := 1 + \frac{15}{100}$

Impact factor for all other limit states................................. $IM := 1 + \frac{33}{100}$

B2. Resistance Factors [AASHTO GFRP 2009]

Flexure and tension of reinforced concrete............. ϕ_f= from 0.55 to 0.75 depending on the reinforcement ratio

[AASHTO GFRP 2009, 2.7.4.2]

Shear and torsion of normal weight concrete.................. $\phi_v := 0.75$ [AASHTO GFRP 2018, 2.5.5.2]

B3. Limit States [AASHTO LRFD 2017, 1.3.2]

The LRFD defines a limit state as a condition beyond which the bridge or component ceases to satisfy the provisions for which it was designed. There are four limit states prescribed by LRFD. These are as follows:

STRENGTH LIMIT STATE

Load combinations which ensures that strength and stability, both local and global, are provided to resist the specified load combinations that a bridge is expected to experience in its design life. Extensive distress and structural damage may occur under strength limit state, but overall structural integrity is expected to be maintained.

EXTREME EVENT LIMIT STATES

Load combinations which ensure the structural survival of a bridge during a major earthquake or flood, or when collided by a vessel, vehicle, or ice flow, possibly under scoured conditions. Extreme event limit states are considered to be unique occurrences whose return period may be significantly greater than the design life of the bridge.

SERVICE LIMIT STATE

Load combinations which place restrictions on stress, deformation, and crack width under regular service conditions.

FATIGUE LIMIT STATE

Load combinations which place restrictions on stress range as a result of a single design truck. It is intended to limit crack growth under repetitive loads during the design life of the bridge.

Table 3.4.1-1 - Load Combinations and Load Factors

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	1.00	—	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	1.00	1.00	1.00	0.50/1.20	γ_{TG}	γ_{SE}	—	—	—	—	—
Extreme Event I	1.00	γ_{EQ}	1.00	—	—	1.00	—	—	—	1.00	—	—	—	—
Extreme Event II	1.00	0.50	1.00	—	—	1.00	—	—	—	—	1.00	1.00	1.00	1.00
Service I	1.00	1.00	1.00	1.00	1.00	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	γ_{LL}	1.00	—	—	1.00	1.00/1.20	γ_{TG}	γ_{SE}	—	—	—	—	—
Service IV	1.00	—	1.00	1.00	—	1.00	1.00/1.20	—	1.00	—	—	—	—	—
Fatigue I— *LL, IM & CE* only	—	1.75	—	—	—	—	—	—	—	—	—	—	—	—
Fatigue II— *LL, IM & CE* only	—	0.80	—	—	—	—	—	—	—	—	—	—	—	—

Table 3.4.1-2 - Load factors for permanent loads, γ_p

Type of Load, Foundation Type, and Method Used to Calculate Downdrag		Load Factor Maximum	Minimum
DC: Component and Attachments		1.25	0.90
DC: Strength IV only		1.50	0.90
DD: Downdrag	Piles, α Tomlinson Method	1.40	0.25
	Piles, λ Method	1.05	0.30
	Drilled shafts, O'Neill and Reese (2010) Method	1.25	0.35
DW: Wearing Surfaces and Utilities		1.50	0.65
EH: Horizontal Earth Pressure			
• Active		1.50	0.90
• At-Rest		1.35	0.90
• *AEP* for anchored walls		1.35	N/A
EL: Locked-in Construction Stresses		1.00	1.00
EV: Vertical Earth Pressure			
• Overall Stability		1.00	N/A
• Retaining Walls and Abutments		1.35	1.00
• Rigid Buried Structure		1.30	0.90
• Rigid Frames		1.35	0.90
• Flexible Buried Structures			
○ Metal Box Culverts, Structural Plate Culverts with Deep Corrugations, and Fiberglass Culverts		1.50	0.90
○ Thermoplastic Culverts		1.30	0.90
○ All others		1.95	0.90
ES: Earth Surcharge		1.50	0.75

B4. Span-to-Depth Ratios in LRFD [AASHTO LRFD 2017, 2.5.2.6.3]

For continuous reinforced slabs with main reinforcement parallel to traffic

$$t_{min} = \frac{S + 10}{30} \geq 0.54 \cdot ft$$

Minimum slab thickness

$$t_{min} := \max\left(\frac{L_{span} + 10 \cdot ft}{30}, 0.54 \cdot ft\right)$$ $t_{min} = 18 \cdot in$

Thickness of flat slab chosen.. $t_{slab} := 18 \cdot in$

Slab width used for computation.................................. $b_{slab} := 12in$

C. FDOT Criteria

C1. Chapter 1 - General Requirements

General [SDG 2020, 1.1]

- The design life for bridge structures is 75 years.
- Approach slabs are considered superstructure component.
- Class II Concrete (Bridge Slab) will be used for all environmental classifications.

Criteria for Deflection only [SDG 2020, 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 2020, 1.3]

The concrete cover for the slab is based on either the environmental classification **[SDG 2020, 1.4]** or the type of bridge **[SDG 2020, 4.2.1]**.

Concrete clear cover for the slab.. $c_c := 1.5in$ **[FRPG 2020, 2.3]**

Concrete clear cover for substructure not in contact with water $c_c = 1.5 \cdot in$ **[FRPG 2020, 2.3]**

Minimum 28-day compressive strength of concrete components

Class		Location
II (Bridge Slab)	CIP Bridge Slab	$f_{c.slab} := 4.5 \cdot ksi$
IV	CIP Substructure	$f_{c.sub} := 5.5 \cdot ksi$
V (Special)	Concrete Piling	$f_{c.pile} := 6.0 \cdot ksi$

Environmental Classifications [SDG 2020, 1.3]

The environment can be classified as either "Slightly" , "Moderately" or "Extremely" agressive.

Environmental classification for superstructure.................. $Environment_{super} \equiv \text{"Extremely"}$

Environmental classification for substructure..................... $Environment_{sub} \equiv \text{"Extremely"}$

C2. Chapter 2 - Loads and Load Factors

Dead loads [SDG 2020, 2.2, 4.2]

Weight of future wearing surface

$$\rho_{fws} := \begin{cases} 15 \cdot psf & \text{if } L_{bridge} < 100ft \\ 0 \cdot psf & \text{otherwise} \end{cases} \qquad \rho_{fws} = 0 \cdot psf$$

Weight of sacrificial milling surface, using $t_{mill} = 0.5 \cdot in$

$$\rho_{mill} := t_{mill} \cdot \gamma_{conc} \qquad \rho_{mill} = 6.042 \cdot psf$$

(***Note***: *See Sect. C3 [SDG 4.2] for calculation of* t_{mill}*)*.

Seismic Provisions [SDG 2020, 2.3]

Seismic provisions for minimum bridge support length only.

Miscellaneous Loads [SDG 2020, Table 2.2-1]

ITEM	UNIT	LOAD
General		
Concrete, Counterweight (Plain)	Lb/cf	145
Concrete, Structural (Steel-RC/PC)	Lb/cf	150
Concrete, Structural (FRP-RC/PC)	Lb/cf	145
Future Wearing Surface	Lb/sf	15[1]
Soil; Compacted	Lb/cf	115
Stay-in-Place Metal Forms	Lb/sf	20[2]
Traffic Railings		
Rectangular Tube Retrofit (Index 460-490)	Lb/ft	30
42" Vertical Shape (Index 521-422)	Lb/ft	590
32" Vertical Shape (Index 521-423)	Lb/ft	385
36" Single-Slope Median (Index 521-426)	Lb/ft	645
36" Single-Slope (Index 521-427)	Lb/ft	430
42" Single-Slope (Index 521-428)	Lb/ft	580
Thrie-Beam Retrofit (Index 460-471, 460-475 & 460-476)	Lb/ft	40
Thrie-Beam Retrofit (Index 460-472, 460-473 & 460-474)	Lb/ft	30
Vertical Face Retrofit with 8" curb height (Index 521-480 to 521-483)	Lb/ft	270

Weight of traffic railing barrier... $w_{barrier} := 430 \cdot plf$

Weight of traffic railing median barrier.............................. $w_{median.bar} := 645 \cdot plf$

Barrier / Railing Distribution for Beam-Slab Bridges [SDG 2020, 2.8]

The traffic railing barriers and median barriers will be distributed equally over the full bridge cross-section.

C3. Chapter 4 - Superstructure Concrete

General [SDG 2020, 4.1]

Correction factor for Florida limerock coarse aggregate $\phi_{limerock} := 1.0$

Unit Weight of Florida limerock concrete $w_{c.limerock} := 145 \cdot pcf$

Yield strength of reinforcing steel $f_y := 60 \cdot ksi$

Note*: Epoxy coated reinforcing not allowed on FDOT projects.*

Modulus of elasticity for slab

$$E_{c.slab} := \phi_{limerock} \cdot \left(1820 \cdot \sqrt{f_{c.slab} \cdot ksi}\right) \qquad E_{c.slab} = 3861 \cdot ksi$$

Modulus of elasticity for substructure

$$E_{c.sub} := \phi_{limerock} \cdot \left(1820 \cdot \sqrt{f_{c.sub} \cdot ksi}\right) \qquad E_{c.sub} = 4268 \cdot ksi$$

Modulus of elasticity for piles

$$E_{c.pile} := \phi_{limerock} \cdot \left(1820 \cdot \sqrt{f_{c.pile} \cdot ksi}\right) \qquad E_{c.pile} = 4458 \cdot ksi$$

Concrete Slabs [SDG 2020, 4.2]

Bridge length definition

$$BridgeType := \begin{cases} \text{"Short"} & \text{if } L_{bridge} < 100ft \\ \text{"Long"} & \text{otherwise} \end{cases} \qquad BridgeType = \text{"Long"}$$

Thickness of sacrificial milling surface

$$t_{mill} \equiv \left(\begin{cases} 0 \cdot in & \text{if } L_{bridge} < 100ft \\ 0.5 \cdot in & \text{otherwise} \end{cases} \right) \qquad t_{mill} = 0.5 \cdot in$$

Slab thickness $t_{slab} = 18 \cdot in$

C4. Chapter 6 - Superstructure Components

Temperature Movement [SDG 2020, 6.3, Table 2.7.1-1]

Superstructure Material	Temperature Range (Degrees Fahrenheit)			
	Mean	High	Low	Range
Concrete Only	70	105	35	70
Concrete Deck on Steel Girder	70	110	30	80
Steel Only	70	120	30	90

The temperature values for "Concrete Only" in the preceding table apply to this example.

Temperature mean.. $t_{mean} := 70 \cdot °F$

Temperature high.. $t_{high} := 105 \cdot °F$

Temperature low.. $t_{low} := 35 \cdot °F$

Temperature rise

$\Delta t_{rise} := t_{high} - t_{mean}$ $\Delta t_{rise} = 35 \cdot °F$

Temperature fall

$\Delta t_{fall} := t_{mean} - t_{low}$ $\Delta t_{fall} = 35 \cdot °F$

Coefficient of thermal expansion
for normal weight concrete.. $\alpha_t := \dfrac{6 \cdot 10^{-6}}{°F}$
[AASHTO LRFD 2020, 5.4.2.2]
.

Expansion Joints [SDG 2020, Table 6.4.1]

Expansion Joint Type	Maximum Open Width "W" (measured in the direction of travel at deck surface)
Hot Poured or Poured Joint without Backer Rod	3/4-inch
Poured Joint with Backer Rod	3-inches
Armored Elastomeric Strip Seal (Single gap)	Per ***LRFD*** [14.5.3.2]
Modular Joint (Multiple modular gaps)	Per ***LRFD*** [14.5.3.2]
Finger Joint	Per ***LRFD*** [14.5.3.2]

For new construction, use only the joint types listed in the preceding table. A typical joint for C.I.P. flat slab bridges is the silicone seal. (See FDOT Index 458-110 Standard Plans and Stanard Plans Instructions)

Maximum joint width.. $W_{max} := 3 \cdot in$

Minimum joint width at 70° F.............................. $W_{min} := 2 \cdot in$

Proposed joint width at 70° F.............................. $W := 2.5 \cdot in$

Movement [SDG 2020, 6.4.2]

For concrete structures, the movement is based on the greater of the following combinations **[AASHTO LRFD 2017, Table 3.4.1-1, Table 3.4.1-3]**:

Movement from factored combination of temperature, creep and shrinkage effects....

$$\Delta_{expansion} = \gamma_{TU} + \Delta_{temperature.rise}$$

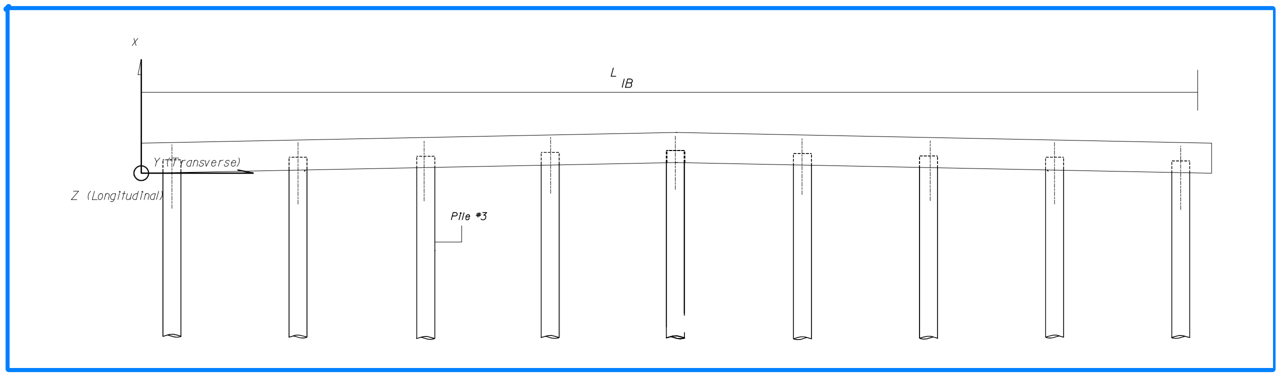
$$\Delta_{contraction} = \gamma_{TU} \cdot \Delta_{temperature.drop} + \gamma_{CR} \cdot \Delta_{creep} + \gamma_{SHR} \cdot \Delta_{shrinkage}$$

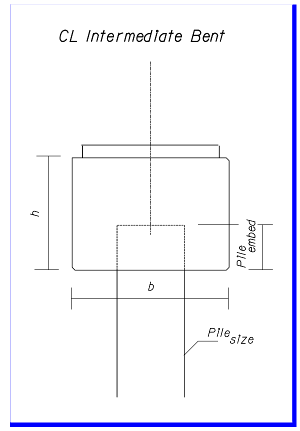
*(**Note:** A temperature rise with creep and shrinkage is not investigated since they have opposite effects)*

*(**Note**: For concrete structures, the temperature rise and fall ranges are the same)*

D. Substructure

D1. Bent 2 Geometry (Bent 3 similar)





Depth of intermediate bent cap..... $h_{cap} := 2.5 \cdot ft$

Width of intermediate bent cap..... $b_{cap} := 3.5 \cdot ft$

Length of intermediate bent cap.... $L := 102.86 \cdot ft$

Pile Embedment Depth................ $Pile_{embed} := 12 \cdot in$

Pile Size.................................... $Pile_{size} := 18 \cdot in$

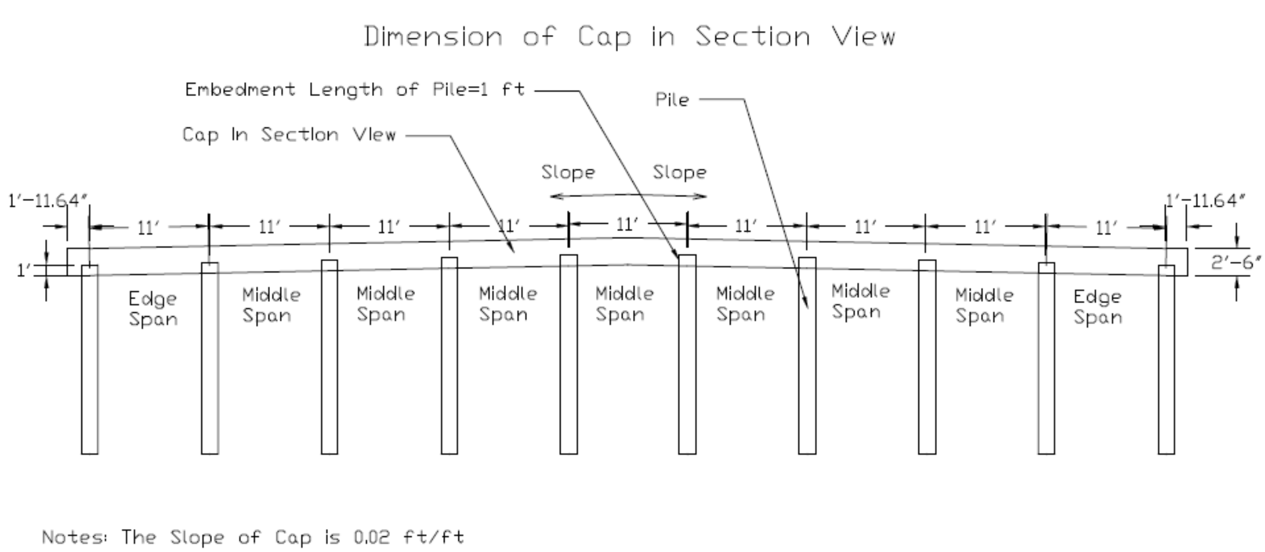
Length of intermediate bent cap $L_{cap} := 11ft$

Length of edge bent cap............. $L_{edge.cap} := 1.93ft$

Number of spans...................... $N_{cap} := 9$

Concrete clear cover.................. $c_c = 1.5 \cdot in$

(***Note***: *For this design example, only the intermediate bent will be evaluated*).



▶ Defined Units

Material properties

Description

This section provides the design input parameters necessary for the superstructure and substructure design.

Page Contents

A. Concrete properties

A1. Define superstructure concrete properties

28-day concrete compressive strength

note: minimum 3400psi

$f'_{c.super} := 4500psi$

Concrete tensile strength [AASHTO 5.4.2.6]

$$f_{r.super} := 0.24\sqrt{f'_{c.super}\cdot ksi} = 0.51\cdot ksi$$

Concrete ultimate strain [AASHTO GFRP 2018, 2.6.2.1]

$$\varepsilon_{cu} := 0.003$$

Unit weight of concrete

$$w_c := 145pcf$$

Correction factor for Florida limerock coarse aggregate

$$\Phi_{limerock} := 1.0$$

Concrete modulus of elasticity [AASHTO LRFD 5.4.2.4]

$$E_{c.super} := 120000\cdot\Phi_{limerock}\cdot\left(\frac{w_c}{pcf\cdot 1000}\right)^2\cdot\sqrt[3]{f'_{c.super}\cdot ksi^2} = 4165\cdot ksi$$

Stress-block coefficient [AASHTO LRFD 5.6.2.2]

$$\beta_{1.super} := \begin{cases} 0.85 & \text{if } f'_{c.super} = 4000psi \\ 1.05 - 0.05\cdot\dfrac{f'_{c.super}}{1000psi} & \text{if } 4000psi < f'_{c.super} < 8000psi \\ 0.65 & \text{otherwise} \end{cases} = 0.83$$

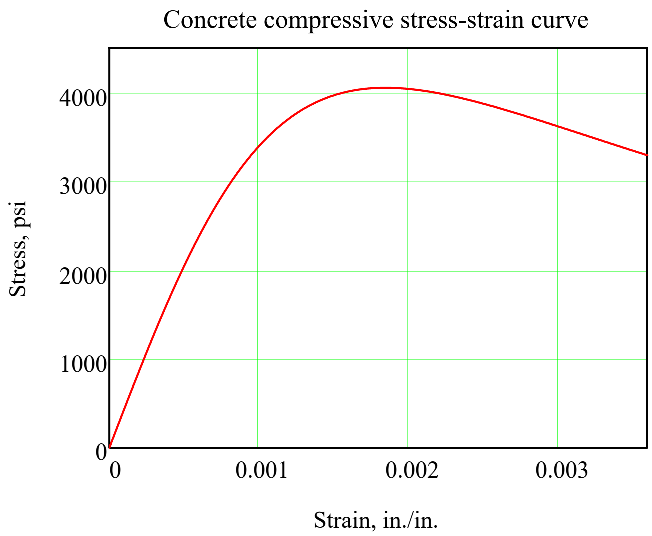
A2. Todeschini's model approximation for superstructure concrete

Compressive strain at peak: $\varepsilon_{c0.super} := \dfrac{1.71\cdot f'_{c.super}}{E_{c.super}} = 0.00185$

Compressive stress at peak: $\sigma''_{c.super} := 0.9\cdot\dfrac{f'_{c.super}}{psi}$

Stress-strain curve equation:

$$\sigma_{c.super}(\varepsilon_c) := \frac{2\cdot\sigma''_{c.super}\cdot\left(\frac{\varepsilon_c}{\varepsilon_{c0.super}}\right)}{1+\left(\frac{\varepsilon_c}{\varepsilon_{c0.super}}\right)^2}$$



A3. Define substructure concrete properties

28-day concrete compressive strength

note: minimum 5500psi

$f'_{c.sub} := 5500psi$

Concrete tensile strength [AASHTO 5.4.2.6]

$f_{r.sub} := 0.24\sqrt{f'_{c.sub}\cdot ksi} = 0.56\cdot ksi$

Concrete ultimate strain [AASHTO GFRP 2018, 2.6.2.1]

$\varepsilon_{cu} = 0.003$

Unit weight of concrete

$w_c = 145\cdot pcf$

Correction factor for Florida limerock coarse aggregate

$\Phi_{limerock} = 1$

Concrete modulus of elasticity

$$E_{c.sub} := 120000 \cdot \Phi_{limerock} \cdot \left(\frac{w_c}{pcf \cdot 1000}\right)^2 \cdot \sqrt[3]{f'_{c.sub} \cdot ksi^2} = 4454 \cdot ksi$$

Stress-block coefficient [AASHTO LRFD 5.6.2.2]

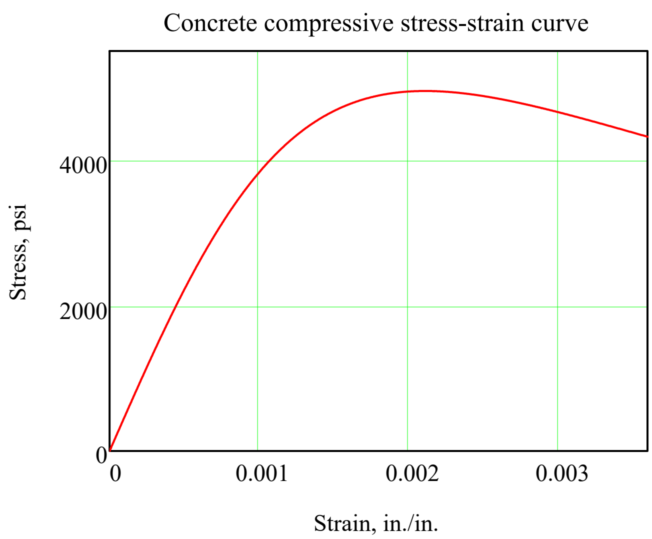
$$\beta_{1.sub} := \begin{cases} 0.85 & \text{if } f'_{c.sub} = 4000psi \\ 1.05 - 0.05 \cdot \dfrac{f'_{c.sub}}{1000psi} & \text{if } 4000psi < f'_{c.sub} < 8000psi \\ 0.65 & \text{otherwise} \end{cases} = 0.78$$

A4. Todeschini's model approximation for substructure concrete

Compressive strain at peak: $\varepsilon_{c0.sub} := \dfrac{1.71 \cdot f'_{c.sub}}{E_{c.sub}} = 0.00211$

Compressive stress at peak: $\sigma''_{c.sub} := 0.9 \cdot \dfrac{f'_{c.sub}}{psi}$

Stress-strain curve equation: $\sigma_{c.sub}(\varepsilon_c) := \dfrac{2 \cdot \sigma''_{c.sub} \cdot \left(\dfrac{\varepsilon_c}{\varepsilon_{c0.sub}}\right)}{1 + \left(\dfrac{\varepsilon_c}{\varepsilon_{c0.sub}}\right)^2}$



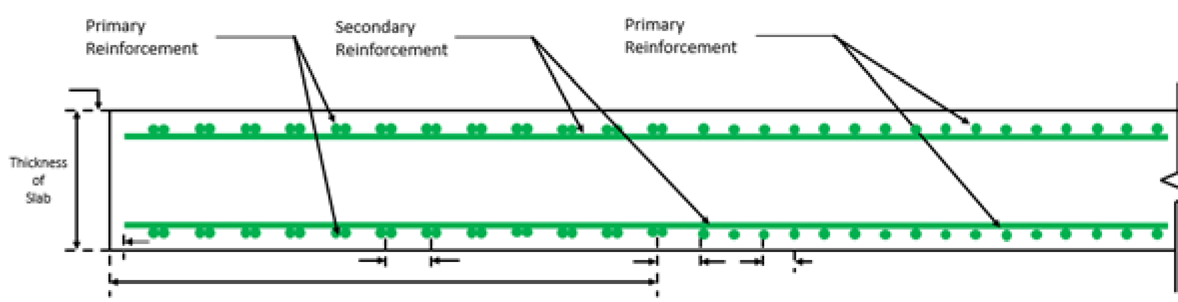
B. GFRP Reinforcement properties

Reinforcement type according to [FDOT 932].
Manufacturers provide material properties exceeding [FDOT 932] minimum requirements.
Guaranteed values provided by manufacturer are adopted for design purposes.

B1. Define flat slab reinforcement

Primary Reinforcement

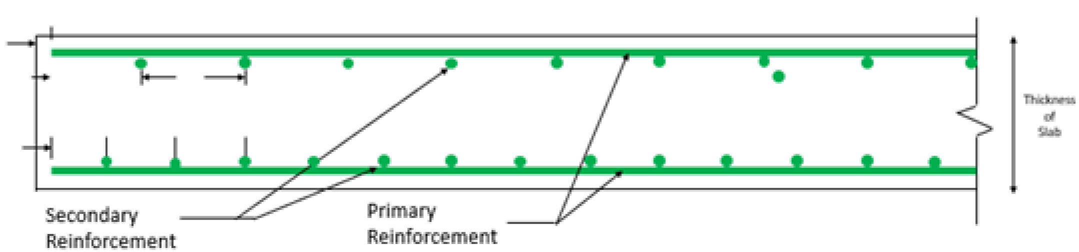
$BarSize_{slab.pr}$	size of primary reinforcement	10	
$BarSpace_{slab.pr}$	spacing of primary reinforcement	4	inches



Primary Reinforcement Section View

Secondary Reinforcement

$BarSize_{slab.sec}$	size of secondary reinforcement	6	
$BarSpace_{slab.sec}$	spacing of secondary reinforcement	8	inches



Secondary Reinforcement Section View

Reinforcement Properties

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

GFRP design references:

- *FDOT Structures Manual, Volume 4, 2020*
- *FDOT Specifications Section 932-3, January 2020*
- AASHTO Bridge Design Guide Specifications for GFRP Reinforced Concrete, 2nd Edition

▶ Initialize Data

▶ Reinforcing Bar Properties

Flat Slab Design Loads

References (links to other mathcad files)

Reference:C:\Users\rober\Google Drive (rxr1084@miami.edu)\FRP - Internal\09 FRP Course - all files\04 FDOT Course (8 hours)\6.

Description

This section provides the design loads for the flat slab superstructure

Page Contents

LRFD Criteria

STRENGTH I - Basic load combination relating to the normal vehicular use of the bridge without wind.

$WA = 0$ For superstructure design, water load and stream pressure are not applicable.

$FR = 0$ No friction forces.

$$\text{Strength1} = 1.25 \cdot DC + 1.50 \cdot DW + 1.75 \cdot LL + 0.50 \cdot (TU + CR + SH)$$

STRENGTH II - Load combination relating to the use of the bridge by Owner-specified special design vehicles, evaluation permit vehicles, or both without wind.

"Permit vehicles are not evaluated in this design example"

SERVICE I - Load combination relating to the normal operational use of the bridge with a 55 MPH wind and all loads taken at their nominal values.

$BR, WL = 0$ For superstructure design, braking forces and wind on live load are not applicable.

$CR, SH = 0$ Creep and shrinkage is not evaluated in this design example.

$$\text{Service1} = 1.0 \cdot DC + 1.0 \cdot DW + 1.0 \cdot LL$$

CREEP RUPTURE - $\text{CreepRupture} = 1.0DC + 1.0DW + 0.2LL$

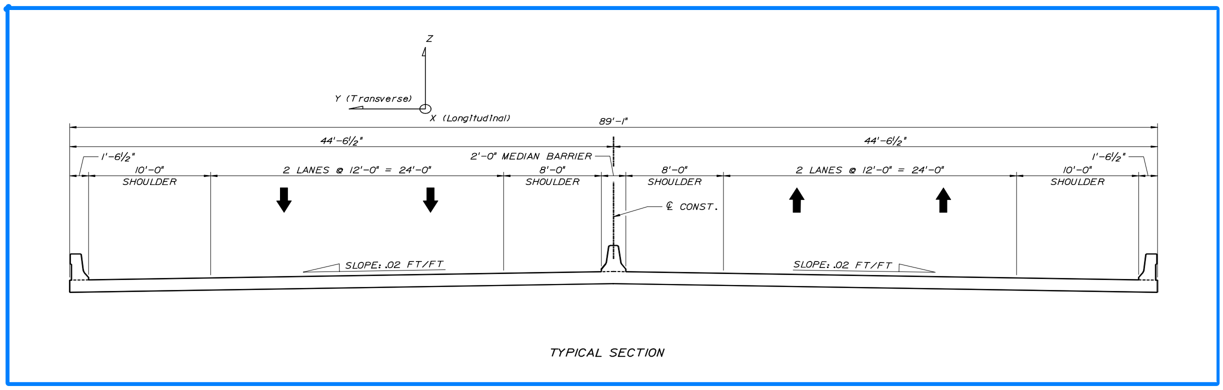
FATIGUE - Fatigue load combination relating to repetitive gravitational vehicular live load under a single design truck.

$$\text{Fatigue} = 0.8 \cdot LL$$

Note:

- **AASHTO LRFD 2017 C4.6.2.1.6** states that "past practice has been not to check shear in typical decks... It is not the intent to check shear in every deck." In addition, **AASHTO LRFD 2017 5.12.2.1** states that for cast-in-place slab superstructures designed for moment in conformance with **AASHTO LRFD 2017 4.6.2.3**, may be considered satisfactory for shear.
- For this design example, shear will not be investigated. From previous past experience, if the slab thickness is chosen according to satisfy LRFD minimum thickness requirements as per the slab to depth ratios and designed utilizing the distribution strips, shear will not control. If special vehicles are used in the design, shear may need to be investigated.

A. Input Variables



Bridge design span length........................ $L_{span} = 35\ \text{ft}$

Thickness of superstructure slab............. $t_{slab} = 18 \cdot \text{in}$

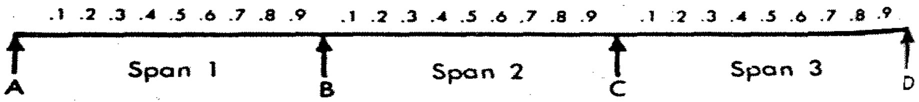
Milling surface thickness........................ $t_{mill} = 0.5 \cdot \text{in}$

Dynamic Load Allowance....................... $IM = 1.33$

Bridge skew.. $Skew = -30 \cdot \text{deg}$

B. Dead Load Analysis

There are numerous programs and charts that can be used to calculate the dead load moments on this type of structure. For the dead load calculation, the influence line coordinates for a uniform load applied on the structure is utilized. The influence coordinates are based on AISC's Moments, Shears and Reactions for Continuous Highway Bridges, published 1966.



	Bridge Length =	105	ft	
	Bridge Width =	89.1	ft	
	# of Traffic Barriers =	2	each	
	# of Median Barriers =	1	each	
	No. of spans =	3	each	
	End Span Lengths =	35.000	ft	
	Interior Span Lengths =	35.000		
	Concrete Weight (DC) =	0.145	kcf	
	Traffic Railing Barrier (DC) =	0.430	klf	
	Median Barrier (DC) =	0.645	klf	
	Wearing Surface and/or fws (DW) =	0.015	ksf	
	Barriers & Median (DC) =	0.0170	ksf = [(2 x 0.430 klf) + (1 x 0.645 klf)] / 89.1 ft = 0.017 ksf	
18 in = Thickness	Bridge Slab (DC) =	0.228	ksf = 18 in. / 12) x 0.145 kcf = 0.2175 ksf	
	Additional Misc Loads (DC)	0.000		
	Components & Attachments (DC) =	0.234	ksf = 0.017 ksf + 0.2175 ksf+ 0 ksf = 0.2344 ksf	

span ratio = 1.00
Use tables 1.0 and 1.1
(From "Moments, Shears and Reactions for Continuous Highway Bridges" published by AISC, 1966)

Pt.	AISC Table	Influence Line Coordinates 1.0	Influence Line Coordinates 1.1	DC MOMENTS (FT-KIP/FT)	DW MOMENTS (FT-KIP/FT)	DC SHEARS (KIP/FT)	DW SHEARS (KIP/FT)
0	A	0.0000	0.0000	0.0	0.0	3.4	0.2
1	0.1	0.0350	0.0340	10.3	0.6	2.5	0.2
2	0.2	0.0600	0.0580	17.6	1.1	1.7	0.1
3	0.3	0.0750	0.0720	22.0	1.4	0.8	0.1
4	0.4	0.0800	0.0760	23.5	1.5	0.0	0.0
5	0.5	0.0750	0.0700	22.0	1.4	-0.8	-0.1
6	0.6	0.0600	0.0540	17.6	1.1	-1.7	-0.1
7	0.7	0.0350	0.0280	10.3	0.6	-2.5	-0.2
8	0.8	0.0000	-0.0080	0.0	0.0	-3.4	-0.2
9	0.9	-0.0450	-0.0540	-13.2	-0.8	-4.2	-0.3
10	B	-0.1000	-0.1100	-29.4	-1.8	-5.0	-0.3
	B	-0.1000	-0.1100	-29.4	-1.8	4.2	0.3
11	1.1	-0.0550	-0.0555	-16.2	-1.0	3.1	0.2
12	1.2	-0.0200	-0.0132	-5.9	-0.4	2.1	0.1
13	1.3	0.0050	0.0171	1.5	0.1	1.0	0.1
14	1.4	0.0200	0.0352	5.9	0.4	0.0	0.0
15	1.5	0.0250	0.0413	7.3	0.5	0.0	0.0
16	1.6	0.0200	0.0352	5.9	0.4	-0.7	0.0
17	1.7	0.0050	0.0171	1.5	0.1	-1.4	-0.1
18	1.8	-0.0200	-0.0132	-5.9	-0.4	-2.1	-0.1
19	1.9	-0.0550	-0.0555	-16.2	-1.0	-2.8	-0.2
	C	-0.1000	-0.1100	-29.4	-1.8	-4.2	-0.3
20	C	-0.1000	-0.1100	-29.4	-1.8	5.0	0.3
21	2.1	-0.0450	-0.0540	-13.2	-0.8	4.2	0.3
22	2.2	0.0000	-0.0080	0.0	0.0	3.4	0.2
23	2.3	0.0350	0.0280	10.3	0.6	2.5	0.2
24	2.4	0.0600	0.0540	17.6	1.1	1.7	0.1
25	2.5	0.0750	0.0700	22.0	1.4	0.8	0.1
26	2.6	0.0800	0.0760	23.5	1.5	0.0	0.0
27	2.7	0.0750	0.0720	22.0	1.4	-0.8	-0.1
28	2.8	0.0600	0.0580	17.6	1.1	-1.7	-0.1
29	2.9	0.0350	0.0340	10.3	0.6	-2.5	-0.2
30	D	0.0000	0.0000	0.0	0.0	-3.4	-0.2

C. Approximate Methods of Analysis - Slabs [AASHTO LRFD 2014, 4.6.2]

C1. Equivalent Strip Widths for Slab-type Bridges [AASHTO LRFD 2017, 4.6.2.3]

The superstructure is designed on a per foot basis longitudinally. However, in order to distribute the live loads, equivalent strips of flat slab widths are calculated. The moment and shear effects of a single HL-93 vehicle or multiple vehicles are divided by the appropriate equivalent strip width. The equivalent strips account for the transverse distribution of LRFD wheel loads. This section is only applicable for spans greater than **15 feet**.

One design lane

The equivalent width of longitudinal strips per lane for both shear and moment with one lane loaded:

$$E = 10 + 5.0\sqrt{L_1 \cdot W_1}$$

where

L_1, modified span length taken equal to the lesser of the actual span or 60 feet.................................... $L_1 := \min(L_{span}, 60.0 \cdot ft)$

$L_1 = 35\,ft$

W_1, modified edge to edge width of bridge taken as the lesser of the actual width, W_{bridge}, or 30 feet for single lane loading.. $W_1 := \min(W_{bridge}, 30.0 \cdot ft)$

$W_1 = 30\,ft$

The equivalent distribution width for one lane loaded is given as.. $E_{onelane} := \left(10 + 5.0\sqrt{\frac{L_1}{ft}\frac{W_1}{ft}}\right) \cdot in$

$E_{onelane} = 14.3 \cdot ft$ or $E_{onelane} = 14.3\,ft$

Two or more design lanes

The equivalent width of longitudinal strips per lane for both shear and moment with more than one lane loaded:

$$E = 84 + 1.44\sqrt{L_1 \cdot W_1} \le \frac{12.0W}{N_L}$$

where

L_1, modified span length.. $L_1 = 35\,ft$

W_1, modified edge to edge width of bridge taken as the lesser of the actual width, W_{bridge}, or 60 feet for multilane loading.. $W_1 := \min(W_{bridge}, 60.0 \cdot ft)$

$W_1 = 60\,ft$

Since the bridge is crowned and the full width of the bridge is used in the equivalent distribution width equation, the number of design lanes should include both roadways. Therefore, number of design lanes..... $N_L := 2 \cdot N_{lanes}$

$N_L = 6$

The equivalent distribution width for more than one lane loaded is given as.................................

$$E_{TwoLane} := \min\left[\left(84 + 1.44\sqrt{\frac{L_1}{ft}\frac{W_1}{ft}}\right), \frac{12.0\left(\frac{W_{bridge}}{ft}\right)}{N_L}\right]\cdot in$$

$E_{TwoLane} = 150.0\cdot in$ or $E_{TwoLane} = 12.5\,ft$

The design strip width to use would be the one that causes the maximum effects. In this case, it would be the minimum value of the two...

$$E := \min(E_{onelane}, E_{TwoLane})$$

$E = 150.0\cdot in$ or $E = 12.5\,ft$

Skew modification

For skewed bridges, the longitudinal force effects (moments only) **may** be reduced by a factor r..............

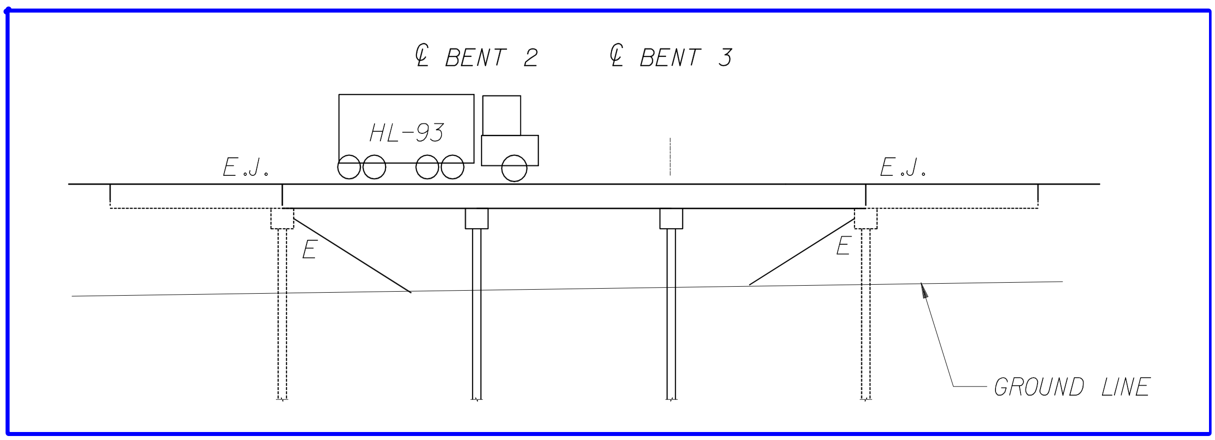
$$r := \min(1.05 - 0.25\cdot\tan(|Skew|), 1.00)$$

$r = 0.91$

*(**Note**: For this design example, the skew modification will not be applied in order to design for more conservative moment values)*

C2. Live Load Analysis

Determine the live load moments and shears due to one HL-93 vehicle on the continuous flat slab structure. The design live loads will consists of the HL-93 vehicle moments, divided by the appropriate equivalent strip widths. This will result in a design live load per foot width of flat slab.



In order to calculate the live load moments and shears, the FDOT MathCad program "LRFD Live Load Generator, English, v2.1".

Read Live Load results from files generated by FDOT Program

The files generated by the program are as follows: ("service1.txt"   "fatigue.txt"). These files are output files that can be used to transfer information from one file to another via read and write commands in MathCad.

The files can be view by clicking on the following icons:

To data is read from the file created by FDOT MathCad program "LRFD Live Load Generator" program.

The values for Strength I can be obtained by multiplying by the appropriate load case factor. The values of Live Load for the HL-93 loads are as follows:

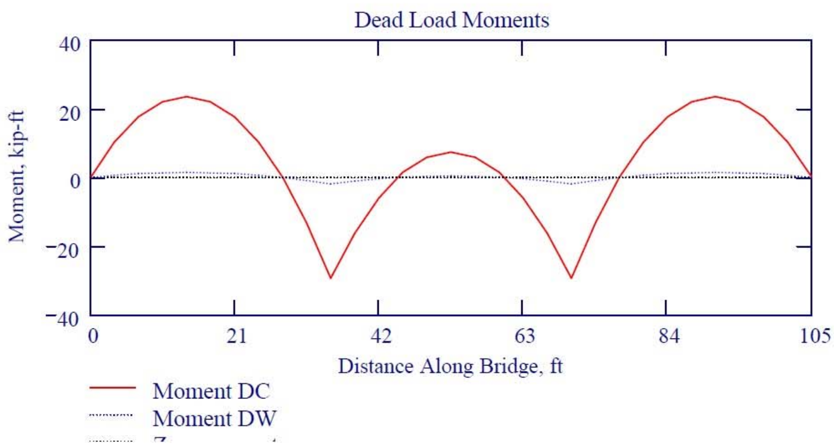
HL-93 Live Load Envelopes

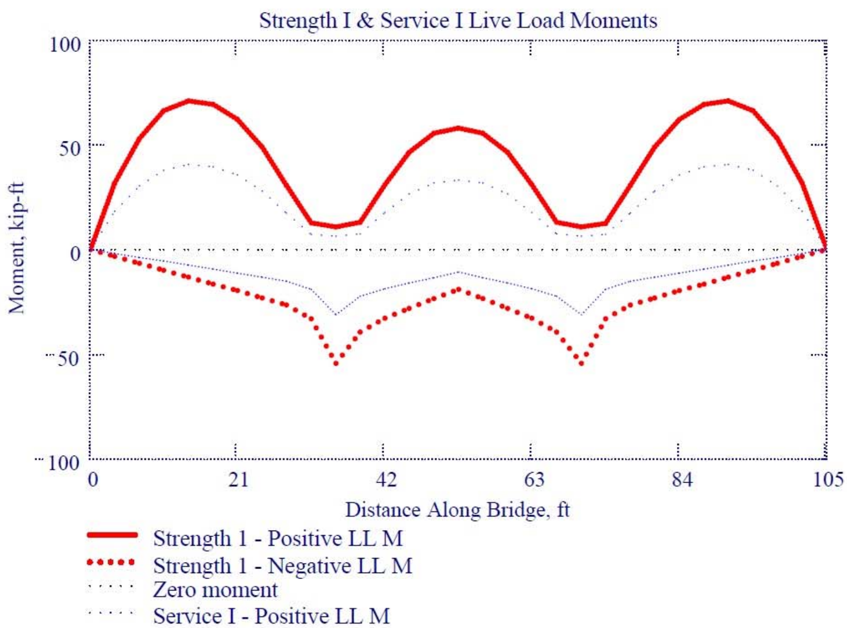
	(10th points)	Service I		Strength I		Fatigue		
Pt.	"X" distance	+M	-M	+M	-M	+M	-M	M_{Range}
0	0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
1	3.5	220.9	-23.0	386.6	-40.2	92.7	-5.8	98.5
2	7	369.4	-46.0	646.5	-80.4	156.0	-11.6	167.5
3	10.5	460.8	-69.0	806.4	-120.7	195.5	-17.3	212.8
4	14	495.0	-92.1	866.3	-161.1	209.2	-23.1	232.3
5	17.5	482.8	-115.0	844.9	-201.3	198.4	-28.9	227.3
6	21	433.1	-137.7	757.9	-241.0	171.1	-34.7	205.8
7	24.5	340.6	-161.5	596.1	-282.6	138.0	-40.5	178.5
8	28	213.3	-184.5	373.3	-322.9	94.9	-59.1	154.0
9	31.5	88.1	-232.9	154.2	-407.6	39.8	-117.9	157.7
10	35	76.1	-383.5	133.2	-671.1	27.0	-186.9	213.8
11	38.5	89.5	-275.7	156.7	-482.5	48.7	-122.2	170.8
12	42	215.3	-228.7	376.8	-400.2	95.6	-81.2	176.8
13	45.5	322.4	-196.6	564.2	-344.1	124.3	-67.5	191.8
14	49	386.1	-165.5	675.7	-289.6	136.6	-54.0	190.5
15	52.5	403.4	-133.9	706.0	-234.3	134.4	-40.5	174.9
16	56	386.1	-165.5	675.7	-289.6	136.6	-54.0	190.5
17	59.5	322.4	-196.6	564.2	-344.1	124.3	-67.5	191.8
18	63	215.3	-228.7	376.8	-400.2	95.6	-81.2	176.8
19	66.5	90.1	-275.7	157.6	-482.5	48.7	-122.2	170.8
20	70	76.1	-383.0	133.2	-670.3	27.0	-186.9	213.8
21	73.5	87.5	-232.9	153.1	-407.6	39.8	-117.9	157.7
22	77	213.3	-184.5	373.3	-322.9	94.9	-59.1	154.0
23	80.5	340.6	-161.5	596.1	-282.6	138.0	-40.5	178.5
24	84	433.1	-137.7	757.9	-241.0	171.1	-34.7	205.8
25	87.5	482.8	-115.0	844.9	-201.3	198.4	-28.9	227.3
26	91	495.0	-92.1	866.3	-161.1	209.2	-23.1	232.3
27	94.5	460.8	-69.0	806.4	-120.7	195.5	-17.3	212.8
28	98	369.4	-46.0	646.5	-80.4	156.0	-11.6	167.5
29	101.5	220.9	-23.0	386.6	-40.2	92.7	-5.8	98.5
30	105	0.0	0.0	0.0	0.0	0.0	0.0	0.0

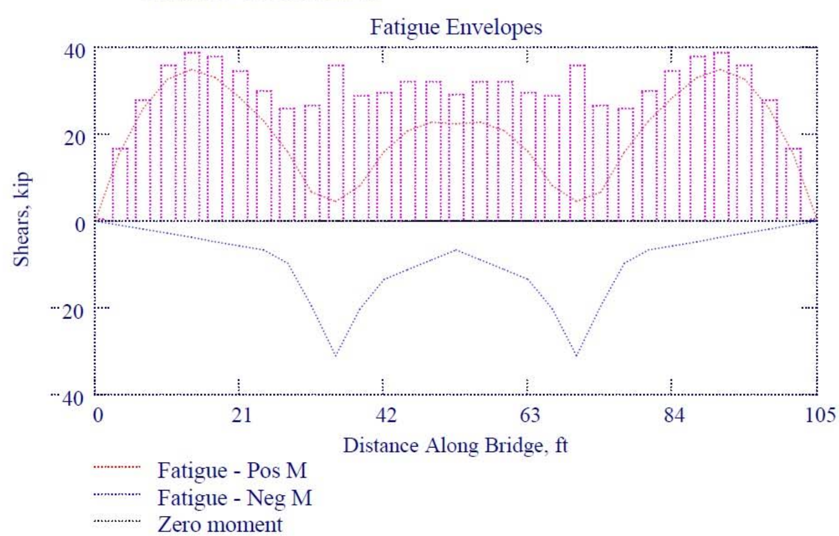
The design values can be obtained by dividing the moments by the distribution width, $E = 12.5 \cdot ft$; for fatigue, $E_{onelane} = 14.3\ ft$

		Design Live Load Envelopes				E =	12,5 ft	
						$E_{fatigue}$ =	14,3 ft	
	(10th points)	Service I		Strength I		Fatigue		
Joint	"X" distance	+M	-M	+M	-M	+M	-M	M_{Range}
0	0	0,0	0,0	0,0	0,0	0,0	0,0	0,0
1	3,5	17,7	-1,8	30,9	-3,2	6,4	-0,4	6,8
2	7	29,6	-3,7	51,7	-6,4	10,8	-0,8	11,6
3	10,5	36,9	-5,5	64,5	-9,7	13,6	-1,2	14,8
4	14	39,6	-7,4	69,3	-12,9	14,5	-1,6	16,1
5	17,5	38,6	-9,2	67,6	-16,1	13,8	-2,0	15,8
6	21	34,6	-11,0	60,6	-19,3	11,9	-2,4	14,3
7	24,5	27,2	-12,9	47,7	-22,6	9,6	-2,8	12,4
8	28	17,1	-14,8	29,9	-25,8	6,6	-4,1	10,7
9	31,5	7,1	-18,6	12,3	-32,6	2,8	-8,2	10,9
10	35	6,1	-30,7	10,7	-53,7	1,9	-13,0	14,8
11	38,5	7,2	-22,1	12,5	-38,6	3,4	-8,5	11,8
12	42	17,2	-18,3	30,1	-32,0	6,6	-5,6	12,3
13	45,5	25,8	-15,7	45,1	-27,5	8,6	-4,7	13,3
14	49	30,9	-13,2	54,1	-23,2	9,5	-3,7	13,2
15	52,5	32,3	-10,7	56,5	-18,7	9,3	-2,8	12,1
16	56	30,9	-13,2	54,1	-23,2	9,5	-3,7	13,2
17	59,5	25,8	-15,7	45,1	-27,5	8,6	-4,7	13,3
18	63	17,2	-18,3	30,1	-32,0	6,6	-5,6	12,3
19	66,5	7,2	-22,1	12,6	-38,6	3,4	-8,5	11,8
20	70	6,1	-30,6	10,7	-53,6	1,9	-13,0	14,8
21	73,5	7,0	-18,6	12,2	-32,6	2,8	-8,2	10,9
22	77	17,1	-14,8	29,9	-25,8	6,6	-4,1	10,7
23	80,5	27,2	-12,9	47,7	-22,6	9,6	-2,8	12,4
24	84	34,6	-11,0	60,6	-19,3	11,9	-2,4	14,3
25	87,5	38,6	-9,2	67,6	-16,1	13,8	-2,0	15,8
26	91	39,6	-7,4	69,3	-12,9	14,5	-1,6	16,1
27	94,5	36,9	-5,5	64,5	-9,7	13,6	-1,2	14,8
28	98	29,6	-3,7	51,7	-6,4	10,8	-0,8	11,6
29	101,5	17,7	-1,8	30,9	-3,2	6,4	-0,4	6,8
30	105	0,0	0,0	0,0	0,0	0,0	0,0	0,0

$i := 0 .. rows(X) - 1$


Dead Load Moments
Moment, kip-ft
40
20
0
−20
−40
0
21
42
63
84
105
Distance Along Bridge, ft
Moment DC
Moment DW


Strength I & Service I Live Load Moments
Moment, kip-ft
100
50
0
−50
−100
0
21
42
63
84
105
Distance Along Bridge, ft
Strength 1 - Positive LL M
Strength 1 - Negative LL M
Zero moment
Service I - Positive LL M


Fatigue Envelopes
Shears, kip
40
20
0
−20
−40
0
21
42
63
84
105
Distance Along Bridge, ft
Fatigue - Pos M
Fatigue - Neg M
Zero moment

C3. Limit State Moments and Shears

The service and strength limit states used to design the section are calculated as follows:

Limit State Design Loads

		Service I		Strength I		Fatigue			
	(10th points)	1.0DC + 1.0DW + 1.0LL		1.25DC + 1.50DW + 1.75LL		1.0DC + 1.0DW + 1.5LL MRange = 0.75LL ; $-M_{min}$ = 0.75LL			
Pt.	**"X" dist**	**+M**	**-M**	**+M**	**-M**	**+M**	**-M**	**M_{Range}**	**$-M_{min}$**
0	0	0,0	0,0	0,0	0,0	0,0	0,0	0,0	0,0
1	3,5	28,6	9,1	44,7	10,6	20,6	10,3	6,8	-0,4
2	7	48,3	15,1	75,4	17,3	35,0	17,5	11,6	-0,8
3	10,5	60,3	17,9	94,1	20,0	43,8	21,6	14,8	-1,2
4	14	64,6	17,6	100,9	18,7	46,7	22,6	16,1	-1,6
5	17,5	62,0	14,2	97,2	13,5	44,1	20,4	15,8	-2,0
6	21	53,4	7,7	84,3	4,4	36,5	15,1	14,3	-2,4
7	24,5	38,2	-2,0	61,5	-8,8	25,3	6,7	12,4	-2,8
8	28	17,1	-14,8	29,9	-25,8	9,9	-6,1	10,7	-4,1
9	31,5	-7,0	-32,7	-5,4	-50,4	-9,9	-26,3	10,9	-8,2
10	35	-25,1	-61,9	-28,8	-93,2	-28,4	-50,7	14,8	-13,0
11	38,5	-10,0	-39,2	-9,2	-60,3	-12,1	-29,9	11,8	-8,5
12	42	11,0	-24,5	22,2	-39,9	3,7	-14,7	12,3	-5,6
13	45,5	27,4	-14,2	47,1	-25,6	14,5	-5,5	13,3	-4,7
14	49	37,1	-7,0	61,9	-15,3	20,5	0,6	13,2	-3,7
15	52,5	40,1	-2,9	66,3	-8,9	21,8	3,6	12,1	-2,8
16	56	37,1	-7,0	61,9	-15,3	20,5	0,6	13,2	-3,7
17	59,5	27,4	-14,2	47,1	-25,6	14,5	-5,5	13,3	-4,7
18	63	11,0	-24,5	22,2	-39,9	3,7	-14,7	12,3	-5,6
19	66,5	-10,0	-39,2	-9,1	-60,3	-12,1	-29,9	11,8	-8,5
20	70	-25,1	-61,9	-28,8	-93,1	-28,4	-50,7	14,8	-13,0
21	73,5	-7,0	-32,7	-5,5	-50,4	-9,9	-26,3	10,9	-8,2
22	77	17,1	-14,8	29,9	-25,8	9,9	-6,1	10,7	-4,1
23	80,5	38,2	-2,0	61,5	-8,8	25,3	6,7	12,4	-2,8
24	84	53,4	7,7	84,3	4,4	36,5	15,1	14,3	-2,4
25	87,5	62,0	14,2	97,2	13,5	44,1	20,4	15,8	-2,0
26	91	64,6	17,6	100,9	18,7	46,7	22,6	16,1	-1,6
27	94,5	60,3	17,9	94,1	20,0	43,8	21,6	14,8	-1,2
28	98	48,3	15,1	75,4	17,3	35,0	17,5	11,6	-0,8
29	101,5	28,6	9,1	44,7	10,6	20,6	10,3	6,8	-0,4
30	105	0,0	0,0	0,0	0,0	0,0	0,0	0,0	0,0
	Maximum negative Moments =		-61,9		-93,2	*46,7*		*16,1*	*-1,6*
	Maximum positive Moments =	64,6		100,9			*-50,7*	*14,8*	*-13,0*

<–Maximum positive moment and corresponding fatigue values

<–Maximum negative moment and corresponding fatigue values

▶ Defined Units

Handbook Design Examples\2-3. Slab (Empirical & Traditional)\GFRPSlabv2.01\1.03.Design.Parameters.xmcd(R)

GFRP-Reinforced Flat Slab Design - Traditional

References (links to other Mathcad files)

Reference:C:\Users\rober\Google Drive (rxr1084@miami.edu)\FRP - Internal\09 FRP Course - all files\04 FDOT Course (8 hours)\6.

Reference:C:\Users\rober\Google Drive (rxr1084@miami.edu)\FRP - Internal\09 FRP Course - all files\04 FDOT Course (8 hours)\6.

Reference:C:\Users\rober\Google Drive (rxr1084@miami.edu)\FRP - Internal\09 FRP Course - all files\04 FDOT Course (8 hours)\6.

Description

This section provides the design for the GFRP-reinforced flat slab superstructure using Traditional Design.

Page Contents

A. Input Variables

Maximum positive moment and corresponding fatigue values

Strength	$M_{str1.pos} := 100.9ft \cdot kip$
Service	$M_{ser1.pos} := 64.6ft \cdot kip$
Fatigue	$M_{fatigue.pos} := 46.7ft \cdot kip$
	$M_{rang.pos} := 16.1ft \cdot kip$
	$M_{min.pos} := -1.6ft \cdot kip$
Live Load Only	$M_{sPos} := 39.6ft \cdot kip$

Maximum negative moment and corresponding fatigue values

Strength	$M_{str1.neg} := 93.2ft \cdot kip$
Service	$M_{ser1.neg} := 61.9ft \cdot kip$
Fatigue	$M_{fatigue.neg} := 50.7ft \cdot kip$
	$M_{rang.neg} := 14.8ft \cdot kip$
	$M_{min.neg} := -13ft \cdot kip$
Live Load Only	$M_{sNeg} := 30.1ft \cdot kip$

DC represents the dead load of components & attachment, and DW represents dead load of wearing surface.

$DC := 0.23klf$

$DW := 0.015klf$

For negative moment between points of contraflexure under a uniform load on all spans, and reaction at interior piers only, 90 percent of the effect of two design trucks spaced a minimum of 50.0 ft between the lead axle of one truck and the rear axle of the other truck, combined with 90 percent of the effect of the design lane load. The distance between the 32.0-kip axles of each truck shall be taken as 14.0 ft. The two design trucks shall be placed in adjacent spans to produce maximum force effects. Flat slab is assumed to support all wheels of truck.

[AASHTO LRFD 2017, 3.6.1.3]

Lane Load LL_{lane} $\quad LL_{lane} := 0.64 \frac{kip}{ft}$ [AASHTO LRFD 2017, 3.6.1.2.4]

Truck Load $LL_{truck\text{-}axis@i}$, i can be 1, 2 and 3

$LL_{TruckAxisAt1} := 8kip$

$LL_{TruckAxisAt2} := 32kip$ [AASHTO LRFD 2017, 3.6.1.2.2]

$LL_{TruckAxisAt3} := 32kip$

Note: The distance between truck axis 1 and 2 is 14 ft and the distance between axis 2 and 3 can range from 14 ft to 30 ft. However, [AASHTO LRFD 2017, 3.6.1.3.1] states the distance between the 32.0-kip axles of one truck shall be taken as 14.0 ft in order to obtain the maximum load effect.

$d_{axisbtw1and2} := 14ft$ Distance between axis 1 and 2 [AASHTO LRFD 2017, 3.6.1.3.1]

$d_{axisbtw2and3} := 14ft$ Distance between axis 2 and 3

Based on influence line analysis of 3-span continuous beam, the maximum shear will occur at right or left sides of support B based on Table 3.0A in AISC Moments Shears and Reactions for Continuous Highway Bridge.

$V_{rightB} := 15.8kip$

$V_{leftB} := -6.8kip$

$V_{rightC} := -10kip$

$V_{max} := \max\left(\left|V_{rightB}\right|, \left|V_{leftB}\right|, \left|V_{rightC}\right|\right) = 15.8 \cdot kip$

Therefore, the ultimate shear in d=[15.875in + half of cap width (42 in)] from right of support B V_{rightB_d}

$$V_{rightB\_d} := \frac{\left(L_{span} - 15.88in - \frac{42in}{2}\right)\cdot\left(15.8kip + \left|V_{rightC}\right|\right)}{L_{span}} - \left|V_{rightC}\right| = 13.5 \cdot kip$$

B. Design of Primary Reinforcement

B1.Data recall (section B of chapter 1.04)

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

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

$E_f = 6500 \cdot ksi$ Modulus of elasticity of slab primary GFRP reinforcement

$f_{fu.slab.pr} = 77.3 \cdot ksi$ Tensile strength of slab primary reinforcement for product certification as reported by GFRP manufacturer [AASHTO GFRP 2018, 2.4.2.1]

$f_{fd.slab.pr} = 54.1 \cdot ksi$ Design strength of slab primary reinforcement considering reduction for service environment [AASHTO GFRP 2018, 2.4.2.1-1]

B2. Check primary reinforcement spacing

Check Primary Reinforcement Spacing

$BarSize_{slab.pr} = 10$ size of primary reinforcement

largest aggregate size

$BarSpace_{slab.pr} = 4 \cdot in$ primary reinforcement spacing $a_g := 0.75$

The minimum reinforcement spacing

$$MinSpacing_{slab.pr} := \max\left(1.5 \cdot Dia_{slab.pr}, 1.5 \cdot a_g \cdot in, 1.5in\right) = 1.9 \cdot in$$

The effective reinforcement depth

$$d_{f1.slab} := t_{slab} - c_c - \frac{Dia_{slab.pr}}{2} = 15.9 \cdot in$$

The maximum reinforcement spacing

$$MaxSpacing_{slab.pr} := \min\left(1.5 \cdot d_{f1.slab}, 18in\right) = 18 \cdot in$$

$$CheckSpacingReinf_{slab.pr} := \left| \begin{array}{l} \text{"NG, minimum clear distance not satisfied"} \quad \text{if } BarSpace_{slab.pr} < MinSpacing_{slab.pr} \\ \text{"NG, maximum spacing exceeded"} \quad \text{if } BarSpace_{slab.pr} > MaxSpacing_{slab.pr} \\ \text{"OK"} \quad \text{otherwise} \end{array} \right.$$

$CheckSpacingReinf_{slab.pr} = \text{"OK"}$

B3. Negative moment region - flexural strength at support

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

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

$b := 12in$ equivalent strip of one foot

Calculate Negative Moment Capacity

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

LRFD 5.6.2.2

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

LRFD 5.6.2.2

Area of primary reinforcement per linear foot

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

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

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

maximum tensile stress in the GFRP

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

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

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

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 $\varepsilon_{fd} > \varepsilon_{ft}$

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

[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:

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

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

Compute the resistance factor for flexural strength

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

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

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

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

B4. Development Length at Support

At least 1/3 of the total tension reinforcement provided for negative moment at a support shall have an embedment length $L_{d.neg.min}$ beyond the point of inflection as follows:

[AASHTO GFRP, 2.9.7.3.2]

$$L_{d.neg.min} := \max\left(d_{f1.slab}, 12 \cdot Dia_{slab.pr}, 0.0625 \cdot L_{span}\right) = 2.2 \cdot ft$$

1/3 of reinforcement has an embedment length beyond the point of inflection that is chosen to be 2.5 ft.

Based on the bending moment envelope, the negative moment extends about 12 ft from the support. Therefore, 1/3 of reinforcement for negative moment should have a length 17 ft, distributed 7 ft from the left support B and 10 ft from the right support B. (assuming B is the interior support between the first and second spans.) At support C, the length is also about 17 ft distributed symmetrically (assuming C is the interior support containing the second and third spans).

The remaining 2/3 of reinforcement for negative moment are distributed along the entire span of bridge.

Lap splices are covered at end of the flexural design (section B7).

B5. Positive Moment Region - Flexural Strength at Middle Span

$Dia_{slab.pr} = 1.3 \cdot in$ — diameter of slab primary GFRP reinforcement

$Area_{slab.pr} = 1.3 \cdot in^2$ — area of slab primary GFRP reinforcement

$b = 12 \cdot in$ — equivalent strip of one foot

$A_{f2.slab} := A_{f1.slab} = 3.8 \cdot in^2$ — area of GFRP reinforcement per foot width for positive moment

$d_{f2.slab} := d_{f1.slab} = 15.9 \cdot in$ — the effective reinforcement depth

Calculate Positive Moment Capacity

$\alpha_1 = 0.9$	bar location modification factor	***LRFD 5.6.2.2***
$\beta_1 = 0.83$	concrete compressive strength factor	***LRFD 5.6.2.2***

Area of primary reinforcement per linear foot

$A_{f2.slab} = 3.8 \cdot in^2$

Area of GFRP reinforcement per foot of negative moment

$$\rho_{f2.slab} := \frac{A_{f2.slab}}{b \cdot d_{f2.slab}} = 0.02001$$

FRP reinforcement ratio

$$f_{f2.slab} := \sqrt{\frac{\left(E_f \cdot \varepsilon_{cu}\right)^2}{4} + \frac{0.85 \cdot \beta_1 \cdot f'_{c.super}}{\rho_{f1.slab}} \cdot E_f \cdot \varepsilon_{cu}} - 0.5 \cdot E_f \cdot \varepsilon_{cu} = 46.6 \cdot ksi$$

maximum tensile stress in the GFRP

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

$$f_{f2.slab} := \min\left(f_{f2.slab}, f_{fd.slab.pr}\right) = 46.6 \cdot ksi$$

Calculate the tensile strain and guaranteed design tensile strain

$$\varepsilon_{ft2.slab.pr} := \frac{f_{f2.slab}}{E_f} = 0.00716$$

tensile strain

$$\varepsilon_{fd2.slab.pr} := \frac{f_{fd.slab.pr}}{E_f} = 0.00833$$

guaranteed design tensile strain

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

$$a_{f2.slab} := \begin{cases} \dfrac{A_{f1.slab} \cdot f_{f1.slab}}{0.85 \cdot f'_{c.super} \cdot b_{slab}} & \text{if } f_{f2.slab} < f_{fd.slab.pr} \\ \beta_1 \cdot \dfrac{\varepsilon_{cu}}{\varepsilon_{cu} + \varepsilon_{fd.slab.pr}} \cdot d_{f2.slab} & \text{otherwise} \end{cases} = 3.9 \cdot in$$

Locate axis depth c_b at balanced strain conditions

$$c_{f2.slab} := \frac{a_{f2.slab}}{\beta_1} = 4.7 \cdot in$$

AASHTO GFRP

The nominal moment capacity is:

$$M_{n.2.slab} := \begin{cases} A_{f2.slab} \cdot f_{f2.slab} \cdot \left(d_{f2.slab} - \frac{a_{f2.slab}}{2} \right) & \text{if } f_{f2.slab} < f_{fd.slab.pr} \\ A_{f2.slab} \cdot f_{fd.slab.pr} \cdot \left(d_{f2.slab} - \frac{a_{f2.slab}}{2} \right) & \text{otherwise} \end{cases}$$

$$M_{n.2.slab} = 205.9 \cdot kip \cdot ft$$

Compute the resistance factor for flexural strength

$$\phi_{2.slab} := \begin{cases} 0.75 & \text{if } \varepsilon_{ft.slab.pr} \le 0.80 \cdot \varepsilon_{fd2.slab.pr} \\ \left(1.55 - \frac{\varepsilon_{ft.slab.pr}}{\varepsilon_{fd2.slab.pr}} \right) & \text{if } 0.80 \cdot \varepsilon_{fd2.slab.pr} < \varepsilon_{ft2.slab.pr} < \varepsilon_{fd2.slab.pr} \\ 0.55 & \text{otherwise} \end{cases}$$

$$\phi_{2.slab} = 0.69$$

Design flexural resistance is computed as:

$$M_{r.2.slab} := \phi_{2.slab} \cdot M_{n.2.slab} = 142.1 \cdot kip \cdot ft$$

Check Primary Reinforcement Moment Capacity for Strength I

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

B6. Development length at middle of span

According to [AASHTO GFRP, 2.9.7.3.1], reinforcement should extend not less than the development length, Ldpos beyond the point at which it is no longer required to resist flexure and no more than 50% should be terminated at any section.

$$L_{d.pos} := \max\left(t_{slab}, 15 \cdot Dia_{slab.pr}, \frac{L_{span}}{20} \right) = 21 \cdot in$$ [AASHTO GFRP 2.9.7.3.2]

Therefore, the selected development length $L_{d.pos.sl}$ is chosen to be 2 ft, which is larger than the required $L_{d.pos}$

$$L_{d.pos.sl} := 2ft$$

1/3 of reinforcement has an embedment length beyond the point of inflection that is chosen to be 2 ft.

Based on the bending moment envelope, the positive bending moment in the first and third span extends 18 ft, and along the second span for 11 ft. Therefore, 1/3 of the reinforcement will have a length of 22 ft for the first and third span, and 15 ft for the second span.

The remaining 2/3 of reinforcement is distributed continuously along the entire span of bridge.

Lap splices are covered at end of flexural design (Section B7).

B7. Reinforcement splices

The tension lap splice length (L_{sp}) should satisfy AASHTO GFRP 2018 2.9.7.6 and 2.9.7.4.1-1.

Development length for deformed bars in tension is defined as $L_{.d.tension}$

Bar location modification factor α, takes the value of 1 except for bars with more than 12 in of concrete cast below for which a value of 1.5 shall be adopted. α is 1.5 for negative moment reinforcement while α=1 for positive moment.

$$\alpha_{neg.slab} := 1.5$$

$$\alpha_{pos.slab} := 1$$

bar modification factor, shall be set to 1.0 except for bars with more than 12 in. of concrete cast below reinforcement, for which a value of 1.5 shall be adopted.

The calculation for lap splices for negative moment and positive moment is:

[AASHTO GFRP 2.9.7.4.1-1]

$$L_{d.tension.neg.slab} := \max\left[Dia_{slab.pr} \cdot \frac{\left(31.6 \cdot \alpha_{neg.slab} \cdot \dfrac{f_{f1.slab}}{\sqrt{f'_{c.super} \cdot ksi}} - 340\right)}{13.6 + \dfrac{c_c}{d_{f1.slab}}}, 20 \cdot Dia_{slab.pr}\right] = 64.9 \cdot in$$

$$L_{d.tension.pos.slab} := \max\left[Dia_{slab.pr} \cdot \frac{\left(31.6 \cdot \alpha_{pos.slab} \cdot \dfrac{f_{f2.slab}}{\sqrt{f'_{c.super} \cdot ksi}} - 340\right)}{13.6 + \dfrac{c_c}{d_{f2.slab}}}, 20 \cdot Dia_{slab.pr}\right] = 32.8 \cdot in$$

[AASHTO 2.9.7.6]

$$L_{sp.neg.req} := \max\left(12in, 1.3 \cdot L_{d.tension.neg.slab}\right) = 84.4 \cdot in$$

$$L_{sp.pos.req} := \max\left(12in, 1.3 \cdot L_{d.tension.pos.slab}\right) = 42.6 \cdot in$$

Therefore, lap splice length $L_{sp.sl}$ selected is:

For negative moment region $L_{sp.neg.sl.slab} := 92in$

For positive moment region $L_{sp.pos.sl.slab} := 48in$

C. Shear Verification

Effective shear depth *LRFD 5.7.2.8* *GFRP LRFD 2.7.2.8*

$$d_v := d_{f1.slab} - \frac{a_{f1.slab}}{2} = 14.1 \cdot in$$

$$d_v := \max\left(d_v, 0.9 \cdot d_{f1.slab}, 0.72 \cdot t_{slab}\right) = 14.3 \cdot in$$

The nominal shear resistance provided by concrete, V_c

Shear Capacity *LRFD 5.7.3.4.2* *GFRP LRFD 2.7.3.6*

$$f\beta and\theta\left(M_u, V_u, d_v, A_s\right) := \left| \begin{array}{l} \varepsilon \leftarrow \dfrac{\max\left(\dfrac{|M_u|}{d_v}, |V_u|\right) + |V_u|}{E_f \cdot A_s} \\ \varepsilon \leftarrow \min(0.006, \varepsilon) \\ \theta \leftarrow 29 + 3500 \cdot \varepsilon \\ s_x \leftarrow \dfrac{d_v}{in} \\ s_{xe} \leftarrow s_x \cdot \dfrac{1.38}{a_g + 0.63} \\ s_{xe} \leftarrow \max\left(12, \min\left(s_{xe}, 80\right)\right) \\ \beta \leftarrow \dfrac{4.8}{1 + 750 \cdot \varepsilon} \dfrac{51}{39 + s_{xe}} \\ (\beta \ \ \theta)^T \end{array} \right.$$

function for β and θ using general procedure when sections do not contain at least the minimum amount of reinforcement

$$\beta := f\beta and\theta\left(M_{str1.pos}, V_{rightB\_d}, d_v, A_{f1.slab}\right)_0 = 1.2$$

$$\theta := f\beta and\theta\left(M_{str1.pos}, V_{rightB\_d}, d_v, A_{f1.slab}\right)_1 = 42.9$$

$$V_c := 0.0316 \cdot \beta \cdot \sqrt{f'_{c.super} \cdot ksi} \cdot b \cdot d_v = 13.3 \cdot kip$$ *[AASHTO GFRP 2.10.2.2]*

$$V_n := 0.5 \cdot V_c = 6.6 \cdot kip$$ *[AASHTO GFRP 2.7.4.2]*

Resistance factor ϕ_v for shear is: $\phi_v = 0.75$

Recall ultimate shear $V_{rightB\_d} = 13.5 \cdot kip$

$V_r := \phi_v \cdot V_n = 5 \cdot kip$

$$CheckShear := \begin{cases} \text{"Verified, no stirrup required"} & \text{if } V_{rightB\_d} \le V_c \\ \text{"Redesign"} & \text{otherwise} \end{cases}$$

$CheckShear = \text{"Redesign"}$

Note:

- **AASHTO LRFD 2014 C4.6.2.1.6** states that "past practice has been not to check shear in typical decks... It is not the intent to check shear in every deck." In addition, **AASHTO LRFD 2017 5.12.2.1** states that for cast-in-place slab superstructures designed for moment in conformance with **AASHTO LRFD 2017 4.6.2.3**, may be considered satisfactory for shear.

- For this design shear will not be investigated. From previous past experience, if the slab thickness is chosen according to satisfy LRFD minimum thickness requirements as per the slab to depth ratios and designed utilizing the distribution strips, shear will not control.

- If a bridge deck conforms to AASHTO minimum thickness requirements, the **AASHTO GFRP 2018 2.7** does not explicitly require a shear check. Thus, you are not mandated to perform the shear strength check for the bridge deck. The primary reason for not requiring the shear check in conventional reinforced decks is the arching action. Antonio Nanni, PhD, PE, Inaugural Senior Scholar, Member ACI 440.1R-15 committee.

D. Creep Rupture

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:

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

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

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

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

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

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

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

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

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

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

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

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

E. Minimum Flexural Reinforcement

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*

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

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

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

F. Longitudinal Spacing Requirements

To control the width of cracking, AASHTO GFRP limits the spacing between longitudinal reinforcement

GFRP LRFD 2.6.7

$E_f = 6500 \cdot ksi$

$C_b = 0.83$ bond reduction factor

$w := 0.028in$ maximum crack width limit

The maximum spacing for crack control

$$MaxBarSpace_{slab.pr} := \min\left(1.15 \cdot \frac{C_b \cdot E_f \cdot w}{f_{f1.creep}} - 2.5 \cdot c_c, 0.92 \cdot \frac{C_b \cdot E_f \cdot w}{f_{f1.creep}}\right) = 11.7 \cdot in$$

$$CheckBarSpace := if\left(BarSpace_{slab.pr} \le MaxBarSpace_{slab.pr}, \text{"OK"}, \text{"No Good"}\right)$$

$$CheckBarSpace = \text{"OK"}$$

$$d_c := t_{slab} - d_{f1.slab} = 2.1 \cdot in$$

$$c_c := \min\left(d_c, c_c\right) = 1.5 \cdot in$$

$$\zeta := \frac{t_{slab} - k_{1.slab} \cdot d_{f1.slab}}{d_{f1.slab} - k_{1.slab} \cdot d_{f1.slab}} = 1.2$$

$$GFRPMaxd_c := \frac{C_b \cdot E_f \cdot w}{2 \cdot f_{f1.creep} \cdot \zeta} = 5.8 \cdot in$$

$$CheckGFRPd_c := if\left(d_c \le GFRPMaxd_c, \text{"OK"}, \text{"No Good"}\right)$$

$$CheckGFRPd_c = \text{"OK"}$$

$$CheckCrack_{slab} := if\left[(CheckBarSpace = \text{"No Good"}) + \left(CheckGFRPd_c = \text{"No Good"}\right), \text{"No Good"}, \text{"OK"}\right]$$

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

G. Secondary Reinforcement

$Dia_{slab.sec} = 0.8 \cdot in$ — diameter of secondary GFRP reinforcement

$Area_{slab.sec} = 0.4 \cdot in^2$ — area of secondary GFRP reinforcement

$E_f = 6500 \cdot ksi$ — modulus of elasticity of slab secondary GFRP reinforcement

$f_{fu.slab.sec} = 93 \cdot ksi$ — tensile strength of slab secondary reinforcement

$f_{fd.slab.sec} = 65.1 \cdot ksi$ — design strength of secondary reinforcement considering reduction for service environment [AASHTO GFRP 2.4.2.1]

Distribution Reinforcement

Reinforcement shall be placed in the secondary direction at the bottom of the slab as a percentage of the primary reinforcement for positive moment as follows:

[AASHTO GFRP 2.10.2.1]

For primary reinforcement parallel to traffic:

Recall $A_{f2.slab} = 3.8 \cdot in^2$

The required secondary reinforcement $A_{sec.req}$

$$A_{sec.req.slab} := \min\left(50\%, \frac{100\%}{\sqrt{\frac{L_{span}}{ft}}}\right) \cdot A_{f2.slab} = 0.64 \cdot in^2$$

The design width in transverse direction is:

$b_{trans} := 12in$

The required number of #6 for secondary reinforcement $N_{sec.req}$

Recall $Dia_{slab.sec} = 0.8 \cdot in$ $Area_{slab.sec} = 0.4 \cdot in^2$

$$N_{sec.req.slab} := \frac{A_{sec.req.slab}}{Area_{slab.sec}} = 1.5$$

The required spacing of #6 reinforcement $S_{sec.req}$

$$S_{sec.req.slab} := \frac{b_{trans}}{N_{sec.req.slab}} = 8.2 \cdot in$$

Spacing for $A_{sec.req}$ is Ssec.req, considering the maximum spacing requirement

[AASHTO GFRP 2.7.2.7]

$$S_{sec.max.slab} := \min(0.5 \cdot t_{slab}, 24in, S_{sec.req.slab}) = 8.2 \cdot in$$

Spacing for minimum spacing [AASHTO 2.11.3]

$$S_{sec.min.slab} := \min(1.5 \cdot Dia_{slab.sec}, 1.5in) = 1.1 \cdot in$$

$$CheckSecondarySpacing := \begin{cases} \text{"VERIFIED"} & \text{if } S_{sec.min.slab} \le BarSpace_{slab.sec} \le S_{sec.max.slab} \\ \text{"NOT VERIFIED"} & \text{otherwise} \end{cases}$$

$$CheckSecondarySpacing = \text{"VERIFIED"}$$

H. Shrinkage and Temperature Reinforcement

The ratio of GFRP reinforcement and temperature reinforcement area to gross concrete area $\rho_{f.st}$

[AASHTO GFRP 2.9.3]

$$\rho_{f.st} := \min\left(0.0036, \max\left(\frac{3132ksi \cdot ksi}{E_f \cdot f_{fd.slab.sec}}, 0.0014\right)\right) = 0.0036$$

minimum area of temperature reinforcing required

The design width in the transverse direction

$$b_{trans} = 12 \cdot in$$

$$A_{g.trans} := b_{trans} \cdot t_{slab} = 216 \cdot in^2$$

GFRP shrinkage and temperature reinforcement A_{st}

$$AreaTemp_{slab.pr.req} := \rho_{f.st} \cdot A_{g.trans} = 0.78 \cdot in^2$$

The number of required reinforcement for shrinkage and temperature area A_{st}

$BarSize_{slab.sec} = 6$ $BarSpace_{slab.sec} = 8 \cdot in$

The shrinkage and temperature reinforcement, provided on top and bottom of the beam require:

$$AreaTemp_{slab.pr} := 2 \cdot \left(Bar_{BarSize_{slab.sec}, 0} \cdot in^2 \cdot \frac{1\,ft}{BarSpace_{slab.sec}} \right) = 1.32 \cdot in^2$$

$$CheckTempReinf_{slab.pr} := \begin{array}{|l} \text{"Insufficient Area Provided"} \quad \text{if } AreaTemp_{slab.pr} < AreaTemp_{slab.pr.req} \\ \text{"Spacing exceeds 3*thickness"} \quad \text{if } BarSpace_{slab.pr} > 3 \cdot t_{slab} \\ \text{"Spacing exceeds 12 inches"} \quad \text{if } BarSpace_{slab.pr} > 12 \cdot in \\ \text{"OK"} \quad \text{otherwise} \end{array}$$

$CheckTempReinf_{slab.pr} = \text{"OK"}$

Temperature and shrinkage requirements can be met with a #6 GFRP bars @ 12" top and bottom. Secondary reinforcing requirements are met with a #6 @ 6" bottom. Thus, the combined configuration is #6 @ 12" top and #6 @ 6" bottom.

I. Deflection Verification

Preliminary Calculations

$$\Delta_{lim.slab} := \frac{L_{span}}{800} = 0.5 \cdot in$$

[AASHTO BDS 2.5.2.6.2]

The gross moment of inertia is:

$$I_{g.slab} := \frac{b_{slab} \cdot t_{slab}^{3}}{12} = 5832 \cdot in^4$$

The negative cracking moment is:

$$M_{cr.Neg.slab} := \frac{f_{r.super} \cdot I_{g.slab}}{t_{slab} - c_{f1.slab} - c_c} = 20.1 \cdot kip \cdot ft$$

The positive cracking moment is:

$$M_{cr.Pos.slab} := \frac{f_{r.super} \cdot I_{g.slab}}{t_{slab} - c_{f2.slab} - c_c} = 20.9 \cdot kip \cdot ft$$

The negative cracking moment is:

$$M_{crNeg.slab} := \frac{f_{r.super} \cdot I_{g.slab}}{t_{slab} - c_{f1.slab} - c_c} = 20.1 \cdot kip \cdot ft$$

The positive cracking moment is:

$$M_{crPos.slab} := \frac{f_{r.super} \cdot I_{g.slab}}{t_{slab} - c_{f2.slab} - c_c} = 20.9 \cdot kip \cdot ft$$

Cracked Moment of Inertia

The cracked moment of inertia, I_{cr}, is computed as follows:

- Case 1: Mid-span

$$I_{cr1.slab} = 1108 \cdot in^4$$

- Case 2: Internal Support

$$I_{cr2.slab} = 1108 \cdot in^4$$

Effective Moment of Inertia

The effective moment of inertia, I_e, is computed using AASHTO LRFD 2014 Eq. 5.7.3.6.2.-1.

The maximum positive bending moment for exterior span due to service loads is:

$$M_{ser1.pos} = 64.6 \cdot kip \cdot ft$$

The value of I_e at midspan is:

Recall $I_{g.slab} = 5832 \cdot in^4$

The effective moment of inertia $I_{e2.slab}$ at where the maximum positive moment due to service load is: [AASHTO GFRP 2018, 2.6.3.4.2-1]

$$\gamma_d := 1.72 - 0.72 \cdot \left(\frac{M_{crPos.slab}}{M_{ser1.pos}} \right) = 1.5$$

parameter to account for the variation in stiffness along the length of the member

$$I_{e2.slab} := \min\left[I_{g.slab}, \frac{I_{cr2.slab}}{1 - \gamma_d \cdot \left(\frac{M_{crPos.slab}}{M_{ser1.pos}} \right)^2 \cdot \left(1 - \frac{I_{cr2.slab}}{I_{g.slab}} \right)} \right] = 1269 \cdot in^4$$

effective moment of inertia

Maximum Deflection

The maximum allowable deflection is:

$$\Delta_{lim.slab} = 0.5 \cdot in$$

The thickness of slab satisfies the minimum requirement of AASHTO LRFD 2017 Bridge Design Specification Table 2.5.2.6.3-1. The instantaneous deflection to be used for the calculation of the long-time deflection is based on the magnification factor.

Considering the bridge is continuous and simply supported, the exterior span can be assumed to be at pinned at one end and fixed at other. Therefore, according to deflection formula of uniformly loaded fixed-pinned beam.

Maximum positive moment due to live load for the exterior span:

$$M_{sPos} = 39.6 \cdot kip \cdot ft$$

The maximum instantaneous deflection under live loads is:

$$\Delta_{SL.slab.ins} := \frac{8}{185} \cdot \frac{M_{sPos} \cdot L_{span}^{2}}{E_{c.super} \cdot I_{e2.slab}} = 0.1\,ft$$

The magnification factor for long-term deflection under live loads is taken directly from AASHTO LRFD 2017 Section 5.6.3.5.2. Here the presence of compression reinforcement ($A'_f = 2/3 A_{f1.slab}$) is considered even if such reinforcement is not taken into account for strength calculation.

[AASHTO LRFD 2017, 5.6.3.5.2]

$$factor_{lt} := \left| \begin{array}{l} \max\left(1.6, 3 - 1.2 \cdot \dfrac{\frac{2}{3} \cdot A_{f1.slab}}{A_{f2.slab}}\right) \text{ if } I_{e2.slab} < I_{g.slab} \\ 4 \text{ if } I_{e2.slab} = I_{g.slab} \end{array} \right. = 2.2$$

$$\Delta_{SL.slab.lt} := factor_{lt} \cdot \Delta_{SL.slab.ins} = 1.5 \cdot in$$

$$CheckSlabInstananeousDeflection := \left| \begin{array}{l} \text{"VERIFIED"} \text{ if } \Delta_{SL.slab.ins} \le \Delta_{lim.slab} \\ \text{"SERVICEABILITY SUGGESTIONS IS NOT MET"} \text{ otherwise} \end{array} \right.$$

CheckSlabInstananeousDeflection = "SERVICEABILITY SUGGESTIONS IS NOT MET"

$$CheckSlabLongTermDeflection := \left| \begin{array}{l} \text{"VERIFIED"} \text{ if } \Delta_{SL.slab.lt} \le \Delta_{lim.slab} \\ \text{"SERVICEABILITY SUGGESTIONS IS NOT MET"} \text{ otherwise} \end{array} \right.$$

CheckSlabLongTermDeflection = "SERVICEABILITY SUGGESTIONS IS NOT MET"

Even through the instance and long-time deflections are higher than the maximum allowable deflection, the design is considered satisfactory as the effect of parapets and edge beam are disregarded. More sophisticated tools could be considered for the computation of deflections.

M. Summary of Reinforcement Provided and Detailing

Primary Reinforcement

$BarSize_{slab.pr} = 10$ — Bar number of primary reinforcement (top and bottom)

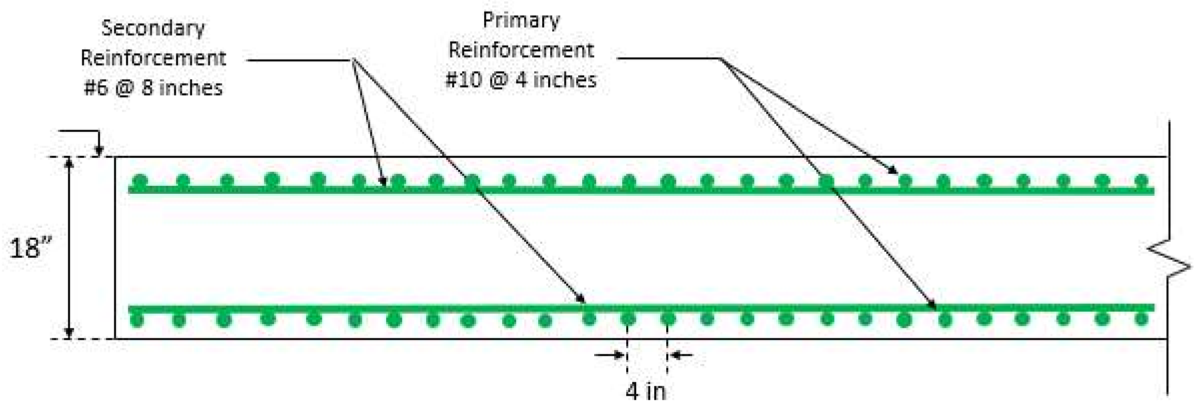
$BarSpace_{slab.pr} = 4 \cdot in$ — Bar spacing of primary reinforcement

$L_{d.neg.min} = 2.2\ ft$ — Development length for negative moment region

$L_{d.pos.sl} = 2\ ft$ — Development length for positive moment region

$L_{sp.neg.sl.slab} = 92 \cdot in$ — Splice length for negative moment region

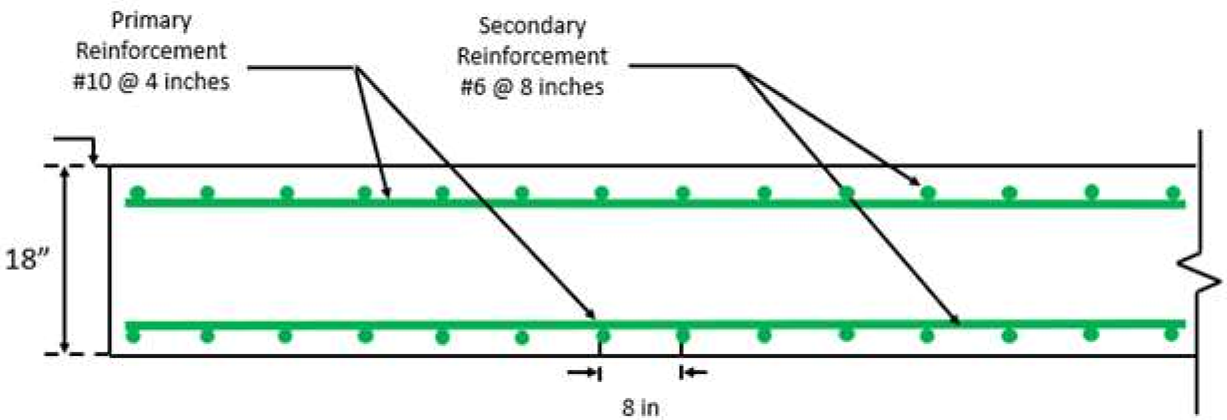
$L_{sp.pos.sl.slab} = 48 \cdot in$ — Splice length for positive moment region



Secondary Reinforcement

$BarSize_{slab.sec} = 6$ Bar number of secondary reinforcement (top and bottom)

$BarSpace_{slab.sec} = 8 \cdot in$ Bar spacing of secondary reinforcement



GFRP-Reinforced Deck Design - Empirical

References (links to other Mathcad files)

Reference:C:\Users\rober\Google Drive (rxr1084@miami.edu)\FRP - Internal\09 FRP Course - all files\04 FDOT Course (8 hours)\6.

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Description

This section provides the design for the GFRP-reinforced deck superstructure using Empirical Design.

Page Contents

A. Introduction

The empirical design method is based upon compressive membrane action (CMA) theory. The arching effect is made possible by cracking in the positive moment region of a deck, which causes the neutral axis to move upward in that portion of the deck. This results in a structural behavior similar to that of a concrete dome. As a result, the failure mechanism for such a member is punching shear.

To design such members according to the empirical method, AASHTO LRFD 9.7.2. specifies the requirements that must be met in order for such design to be valid. At present time, the FDOT has not issued guidance to for empirical design of bridge decks utilizing GFRP reinforcement. The requirements presented here are covered in AASHTO GFRP 2nd Edition without amendments in the SDG. Generally, the traditional method results in higher reinforcement ratios than empirical in the final state.

B. Design Conditions & Limitations

The design conditions that must be met to apply the empirical method are:

Changes in ***navy blue***

- Diaphragms must be used throughout the cross-section at lines of support.
- Supporting components must be made of steel and/or concrete.
- Deck must be fully cast-in-place and must be water cured.
- Deck must have a uniform depth except girder haunches and areas of local thickening.
- 6.0 ≤ effective length to design depth ratio ≤ 18.0
- Core depth of the deck ≥ ***3.5 inches***.
- Core depth of the deck ≥ 7.0 inches; excluding a sacrificial wearing surface.
- Overhang ≥ 5 x deck depth (without a continuous and composite barrier), or ≥ 3 x deck depth (with a continuous barrier)
- 28-day deck concrete strength $f'_c \geq 4.0$ ksi
- Deck is composite with the supporting structural components
- Minimum of two shear connectors at 24-inch spacing in negative moment region

C. Reinforcement Requirements

These reinforcement requirements are as follows:

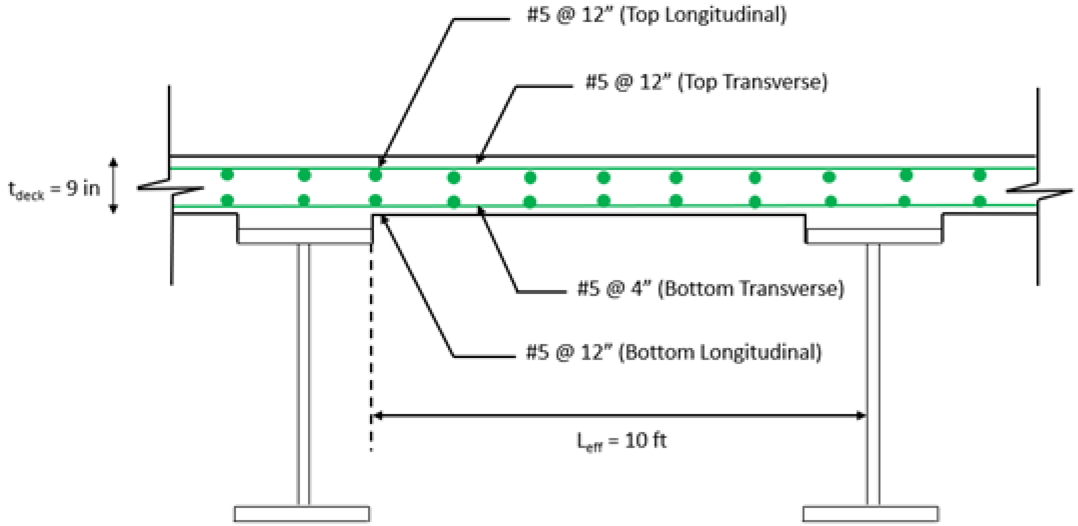
- Four layers of reinforcement (top in each direction and bottom in each direction)
- Area of each bottom layer of reinforcement placed in direction of effective length

$$\geq \frac{870 \cdot d}{E_f}$$

- Reinforcement ratio, $\rho_f \geq 0.0035$
- Spacing of reinforcement ≤ 12 inches

Note format adapted from FHWA manual LRFD for Highway Bridge Superstructures, intro summarized from FDOT BDV34-977-01

Consider the following example:



Bridge Deck Based on Empirical Design Method

$d := 9in - c_c - 0.5in - \frac{0.625in}{2} = 6.688 \cdot in$ design depth

The reinforcement can be checked as follows:

Top Transverse:

$$\text{if}\left(\frac{0.31in^2}{1ft \cdot d} > 0.0035, \text{"OK"}, \text{"NOT OKAY"}\right) = \text{"OK"}$$

Top Longitudinal:

$$\text{if}\left(\frac{0.31in^2}{1ft \cdot d} > 0.0035, \text{"OK"}, \text{"NOT OKAY"}\right) = \text{"OK"}$$

Bottom Longitudinal:

$$\text{if}\left(\frac{0.31in^2}{1ft \cdot d} > 0.0035, \text{"OK"}, \text{"NOT OKAY"}\right) = \text{"OK"}$$

Bottom Transverse:

$$\text{if}\left(0.31in^2 \cdot \frac{1ft}{4in} \cdot \frac{1}{ft} > \frac{870 \cdot \frac{d}{in}}{\frac{E_f}{ksi}} \cdot \frac{in^2}{ft}, \text{"OK"}, \text{"NOT OKAY"}\right) = \text{"OK"}$$

The following table summarizes the possible empirical deck designs utilizing a #5 bar for various effective deck lengths with typical deck thicknesses.

GFRP Empirical Method

Effective Span Length	Deck Thickness	Transverse Bars				Longitudinal Bars			
		Top Bars		Bottom Bars		Top Bars		Bottom Bars	
		Size	Spacing	Size	Spacing	Size	Spacing	Size	Spacing
[ft]	[in]	[-]	[in]	[-]	[in]	[-]	[in]	[-]	[in]
7.0	8.50	#5	12.00	#5	4.50	#5	12.00	#5	12.00
7.5	8.50	#5	12.00	#5	4.50	#5	12.00	#5	12.00
8.0	8.50	#5	12.00	#5	4.50	#5	12.00	#5	12.00
8.5	8.50	#5	12.00	#5	4.50	#5	12.00	#5	12.00
9.0	8.75	#5	12.00	#5	4.00	#5	12.00	#5	12.00
9.5	9.00	#5	12.00	#5	4.00	#5	12.00	#5	12.00
10.0	9.00	#5	12.00	#5	4.00	#5	12.00	#5	12.00
10.5	9.25	#5	12.00	#5	4.00	#5	12.00	#5	12.00
11.0	9.50	#5	12.00	#5	3.50	#5	12.00	#5	12.00
11.5	9.50	#5	12.00	#5	3.50	#5	12.00	#5	12.00
12.0	9.75	#5	12.00	#5	3.50	#5	12.00	#5	12.00
12.5	10.00	#5	12.00	#5	3.50	#5	12.00	#5	12.00
13.0	10.00	#5	12.00	#5	3.50	#5	12.00	#5	12.00
13.5	10.25	#5	11.75	#5	2.50	#5	11.75	#5	11.75

Considering the equivalent effecive span lengh and deck thicknesses, the following table can be developed for steel.

Steel Empirical Method

Effective Slab Length	Deck Thickness	Transverse Bars				Longitudinal Bars			
		Top Bars		Bottom Bars		Top Bars		Bottom Bars	
		Size	Spacing	Size	Spacing	Size	Spacing	Size	Spacing
[ft]	[in]	[-]	[in]	[-]	[in]	[-]	[in]	[-]	[in]
7.0	8.50	#4	12.00	#4	8.00	#4	12.00	#5	12.00
7.5	8.50	#4	12.00	#4	8.00	#4	12.00	#5	12.00
8.0	8.50	#4	12.00	#4	8.00	#4	12.00	#5	12.00
8.5	8.50	#4	12.00	#4	8.00	#4	12.00	#5	12.00
9.0	8.75	#4	12.00	#4	8.00	#4	12.00	#5	12.00
9.5	9.00	#4	12.00	#4	8.00	#4	12.00	#5	12.00
10.0	9.00	#4	12.00	#4	8.00	#4	12.00	#5	12.00
10.5	9.25	#4	12.00	#4	8.00	#4	12.00	#5	12.00
11.0	9.50	#4	12.00	#4	8.00	#4	12.00	#5	12.00
11.5	9.50	#4	12.00	#4	8.00	#4	12.00	#5	12.00
12.0	9.75	#4	12.00	#4	8.00	#4	12.00	#5	12.00
12.5	10.00	#4	12.00	#4	8.00	#4	12.00	#5	12.00
13.0	10.00	#4	12.00	#4	8.00	#4	12.00	#5	12.00
13.5	10.25	#4	12.00	#4	8.00	#4	12.00	#5	12.00