



Structures Engineering Instructions (SEI)

Distribution: Structures, Bureau Chiefs, Chief of Contract Admin., Consultants

Approved: Wayne B. Symonds, **Date:** 4/10/2017
Structures Program Manager

Subject: Distribution of GFRP Reinforced Deck Design Guidelines

Administrative Information:

Effective Date: This SEI shall be considered effective for the Structures Section from the date of approval.

Superseded SEI: None.

Exceptions: None.

Disposition of SEI Content: The technical information transmitted by this SEI will be in effect until it is superseded.

Purpose:

To adopt the GFRP Reinforced Deck Design Guidelines – First Edition SD1-2017.01.18. The guidelines will be accompanied by a design tool for the deck design of the interior bays of beam superstructures.

Technical Information:

The GFRP Reinforced Deck Design Guideline is written to supplement the *AASHTO LRFD Bridge Design Guide Specifications for GFRP-Reinforced Concrete Bridge Decks and Traffic Railings* document to provide additional guidance and/or clarification on specific sections – this is not meant to be a standalone guide. The following document is presented in the same general order as the AASHTO GFRP manual with references to specific AASHTO GFRP manual subsections where applicable.

Throughout this guide, the term “GFRP”, when followed by a section or table number, is used as a reference to the *AASHTO LRFD Bridge Design Guide Specifications for GFRP-Reinforced Concrete Bridge Decks and Traffic Railings, 1st Edition, November 2009*. The term “LRFD”, when followed by a section or table number, is used as a reference to the *AASHTO LRFD Bridge Design, 7th Edition*, with interims through 2016.

The manual and design tool will not be published in paper format, but will be available for download from the VTrans Website.

Implementation:

The content of this SEI will be implemented immediately.

Transmitted Materials:

GFRP Reinforced Deck Design Guidelines
Mathcadd Design Tool – Interior Deck Design - GFRP

2016

SD1-2017.01.18

GFRP Reinforced Deck Design Guidelines

By T.Y. Lin International

First Edition

GFRP Reinforced Deck Design Guidelines

FIRST EDITION

by

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for

VTrans, Structures Section

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SECTION 1: INTRODUCTION

1.1 SCOPE

This document is provided as a supplement to the *AASHTO LRFD Bridge Design Guide Specifications for GFRP-Reinforced Concrete Bridge Decks and Traffic Railings* document to provide additional guidance and/or clarification on specific sections – this is not meant to be a standalone guide. The following document is presented in the same general order as the AASHTO GFRP manual with references to specific AASHTO GFRP manual subsections where applicable.

Throughout this guide, the term “GFRP”, when followed by a section or table number, is used as a reference to the *AASHTO LRFD Bridge Design Guide Specifications for GFRP-Reinforced Concrete Bridge Decks and Traffic Railings, 1st Edition, November 2009*. The term “LRFD”, when followed by a section or table number, is used as a reference to the *AASHTO LRFD Bridge Design, 7th Edition*, with interims through 2016.

1.2 DOCUMENT PRECEDENCE

The content of this design guide supersedes the Structures Design Manual. The Structures Design Manual may be referred to for all other content not found in this guide

1.3 LIMITATIONS

The use of this guideline constitutes the acceptance of the following statement:

The content of this guide are provided for information only. In all cases, the Engineer must use his or her professional judgment when applying the information contained herein. Responsibility for any design resulting from the use of this guide rests solely on the Engineer and not the State of Vermont nor T.Y. Lin International.

AASHTO is a registered trademark of the American Association of State Highway Officials. All trademarks, manufacturer names, or product names cited in this guide are the property of their respective owners. They are used in this guide for identification and informational purposes only. The use of trademarks, manufacturer names, or product names does not represent endorsement of or affiliation with those companies or products in any way.

1.4 BACKGROUND

Glass fiber reinforced polymer (GFRP) reinforcement has emerged as an appropriate alternative to steel reinforcement in certain bridge applications. GFRP reinforcement is well suited for use in corrosive environments in low dead load situations.

The unique properties and behavior of GFRP necessitate different design criteria and design approaches than normal steel reinforcement. GFRP is noncorrosive and can be produced with strengths greater than normal reinforcing steel; however it is also subject to creep under sustained loads, exhibits a brittle failure mode, is highly flexible, and is not effective in compression. Due to these limitations, the design of many situations is governed by serviceability or creep rupture.

SECTION 2: BRIDGE DECKS

2.1 SCOPE

This Section contains additional guidance to better link the concrete deck design provisions of the *AASHTO LRFD Bridge Design Specifications* and the *AASHTO LRFD Bridge Design Guide Specifications for GFRP-Reinforced Concrete Bridge Decks and Traffic Railings* to one another as well as to provide State guidance and preferences.

2.2 GENERAL CONSIDERATIONS

2.2.1 Use Location

GFRP reinforcement is regarded as Level III reinforcement (Exceptional Corrosion Resistance) as defined in the *VTrans Structures Design Manual* subsection 5.1.2.2 and SEI 12-001. Despite the Level III corrosion resistance, a GFRP reinforced concrete deck has less ductility and potentially more serviceability issues than a stainless-steel-reinforced concrete deck. For this reason, GFRP reinforcement may not be acceptable for use in all locations that require Level III reinforcement and the Designer shall coordinate the use of GFRP reinforcement with the VTrans project manager prior to design.

Additional restrictions and recommendations on use location due to bending and ductility issues can be found throughout this guide. Refer to Sections 2.5 and 3.

2.2.2 Deviations from Steel Reinforced Concrete Design

GFRP is a non-yielding, non-ductile material. Since the material does not exhibit plastic behavior, moment redistribution at continuous support locations should not be considered.

The compressive stress of GFRP reinforcement is often considerably lower than its tensile strength. Any GFRP reinforcement located in a compressive region shall be neglected from design calculations.

GFRP reinforcement shall not be used within lightweight concrete decks due to lack of research and performance record at the time of this publication.

2.2.3 Deck Drainage (*GFRP 2.5.1*)

Where deck drains are required, the use of FRP scuppers and downspouts should be incorporated for consistent corrosion resistance of elements within the concrete deck.

2.2.4 Stay-in-Place Formwork (*GFRP 2.5.5*)

The use of stay-in-place formwork should be considered on a project specific basis and is only allowed for use if noted on the Plans per Standard Specification Subsection 501.09.

2.3 MATERIAL PROPERTIES (GFRP 2.6)

2.3.1 General

Reinforcing Bars shall conform to the requirements of the latest version of VTrans “Special Provision (Reinforcing Bar, GFRP)”. See Appendix A for a copy of the special provision current at the time of the publication of this guide. A list of manufacturers of GFRP Reinforcing Bars for use on VTrans projects can be found within the Special Provision and additional guidance can be found in Appendix A of this Guide.

2.3.2 Tensile Strength and Strain (GFRP 2.6.1.2 & 4.6.1)

The nominal tensile strength and strain of GFRP reinforcement varies by manufacturer and bar size. Suggested nominal tensile strength values, based upon information available during the development of this guide, are provided in Table 2.3.2-1. The suggested nominal design strain value for use in design is 1.226%. The suggested design values are the minimum values among all known manufacturers active at the time of publication to provide increased availability and competition in bidding. A more complete summary of currently available tensile strengths and strains from approved manufacturers can be found in Section 4.2. However, due to potential changes to preapproved manufacturers and material property trends as the use of GFRP becomes more widespread, the designer should investigate current values at the beginning of each new design. Regardless of the suggested design values provided here, the nominal tensile strength and strain used in design should be the minimum value reported by at least three manufacturers. The selected minimum nominal tensile strength and strain value(s) shall be noted on the plans.

Table 2.3.2-1 – Suggested Nominal Tensile Strength, f_{fu} , Values for Design, ksi

Bar Size Designation	f_{fu} (ksi)
#3	105
#4	105
#5	100
#6	100
#7	95
#8	90

Note – Higher grade GFRP reinforcement may be available from some manufacturers. The use of higher grade reinforcement can be considered on a project-specific basis and shall require approval by the VTrans Structures Engineer if there are fewer than three vendors. The designer is cautioned to pay specific attention to bar sizes and reporting methods when utilizing higher grade materials to ensure the values and geometries are consistent with calibration of design code requirements and load and resistance factors.

The environmental reduction factor, C_E , shall be 0.70 for all decks regardless of wearing surface type.

2.3.3 Modulus of Elasticity (GFRP 2.6.1.3 & 4.6.2)

Unlike steel, the modulus of elasticity of GFRP reinforcement varies significantly by manufacturer and grade. Based upon information available at the time of publication of this guide, the suggested design

value for modulus of elasticity is 6,150 ksi for all bar sizes. However, due to potential changes to preapproved manufacturers and material property trends as the use of GFRP becomes more widespread, the designer should investigate current values at the beginning of each new design. Regardless of the suggested design value provided here, the modulus of elasticity used in design should be the minimum value reported by at least three manufacturers. The selected value shall be noted on the plans.

2.3.4 Concrete

Concrete shall be in accordance with *VTrans Structures Design Manual* subsection 5.1. Deck designs using GFRP should consider both the minimum required concrete strength and a maximum anticipated concrete strength. Due to the different failure modes possible, the maximum concrete strength may control design in some cases. The resistance factor used in the design of GFRP reinforced decks is highest during a concrete crushing failure and an increase in concrete strength may reduce the resistance factor at a more rapid rate than provide an increase in capacity.

2.4 DESIGN CONSIDERATIONS

2.4.1 Effect of Imposed Deformation (*GFRP 2.8.2*)

The effects of field flexing primary reinforcement shall be considered in design (e.g.: at a crown location). The tensile strength for product certification, f_{fu} , and tensile strain for product certification, ϵ_{fu} , shall be reduced by the amounts shown in Equations 1 and 2, respectively:

$$R_f = \frac{d_b}{2R} E_f \quad (2.4.1-1)$$

$$R_\epsilon = \frac{d_b}{2R} \quad (2.4.1-2)$$

where:

R_f = stress reduction due to field flexing (ksi)

R_ϵ = strain reduction due to field flexing

d_b = nominal bar diameter (in)

R = radius of the flexed bar (in)

E_f = modulus of elasticity of GFRP reinforcement (ksi)

See Section 2.5.6 for additional information. It is recognized that a flexed bar radius is not constant along its length and is difficult to approximate and control in the field. For design purposes, the following equation may be used to approximate a transversely flexed bar radius as a function of the crown rollover percent.

$$R = \frac{4in}{3(\theta_{flex})^2} \quad (2.4.1-3)$$

where:

θ_{flex} = algebraic difference in slope between sides of angle break, always taken as positive (rad)

2.4.2 Flexural Design Considerations

2.4.2.1 Creep Rupture Limit State (*GFRP 2.7.3*)

Creep rupture should be checked at maximum negative moment and maximum positive moment locations under Service I limit state, including live load. Creep rupture need only be checked in the primary direction and shall account for the imposed deformations noted in section 2.4.1.

2.4.2.2 Limits of Reinforcement (*GFRP 2.7.4.2*)

Where possible, the designer should specify a sufficient amount of reinforcement to meet or exceed $1.4\rho_{fb}$. Exceeding the balanced reinforcement ratio by this amount will provide increased resistance factors and simplified analysis.

2.4.2.3 Control of Cracking (*GFRP 2.9.3.4*)

The crack control Provisions of *GFRP 2.9.3.4* shall be satisfied during design. Although crack width does not directly affect corrosion of reinforcement, increased crack widths lead to reflective cracking in wearing surfaces and allow for water to infiltrate and freeze, thereby slowly deteriorating the concrete deck.

The bond dependent coefficient, k_b , may be taken as 1.0 and shall be noted on the plans with the other design values. Multiple manufacturers report the bond coefficient of their reinforcing bars to be equal to or less than 1.0, where 1.0 is equivalent to the bond strength of steel reinforcing bars to the surrounding concrete.

The *Canadian Highway Bridge Design Code* recommends a k_b of 0.8 for sand-coated GFRP bars and 1.0 for deformed GFRP bars. The use of a k_b value less than 1.0 will limit the number of suppliers but it may be considered on a project-specific basis with the approval of VTrans.

The *Canadian Highway Bridge Design Code* also allows two crack width limitations, dependent upon the environmental and exposure conditions. Consideration may be given on a project-specific basis to using a limiting crack width of 0.027 inches with approval of VTrans.

2.4.2.4 Deformation (*GFRP 2.9.3.5 and 2.7.2*)

The limiting deflection of concrete decks spanning between girders or supports shall be $L/800$, where L is the transverse center to center girder distance or bearing to bearing distance. The deflection check shall include the deflections from sustained loads, including creep, along with instantaneous deflections from transient loads. Long term deflection due to sustained dead loads may then be taken as the calculated instantaneous deflection from those loads multiplied by a coefficient of 3 or 4 in accordance with *GFRP 2.9.3.5*.

The AASHTO GFRP Guide allows the use of the gross moment of inertia in determining deformations if the service moment considered for deflection checks is below the cracking moment. However, a significant reduction to stiffness and corresponding increase in deformation occurs once the concrete section is cracked. In these situations, recognition should be given to the fact that heavier live loads not considered for deflection checks are likely to traverse the deck during the life of the bridge. Where the service moment is found to be less than the cracking moment, the Designer should use the effective moment

of inertia to determine deflections by setting M_a equal to M_{cr} , thereby reducing the gross moment of inertia by β_d .

2.4.3 Design for Shear Effects (GFRP 2.10)

Shear effects need not be considered in the design of decks utilizing the standard 8½" design deck thickness and clear cover noted in Section 2.5.1. The designer should investigate the need for shear reinforcement as deemed necessary by their own engineering judgement in accordance with GFRP 2.10.

2.4.4 Deck Overhang Design

Deck overhang design shall be in accordance with AASHTO LRFD Section 13. A Resistance Factor, ϕ , of 1.0 may be used in the design of the deck overhang under Extreme Event load combinations. Development of GFRP reinforcement beyond the critical overhang design section is difficult to achieve in many situations, regardless of the use of hooked ends. Select manufacturers fabricate headed or anchored end options that reduce development length; however, these are not recommended (see section 2.5.3).

In many cases the use of stainless steel reinforcement may be necessary in deck overhangs. If stainless steel reinforcement and GFRP reinforcement are both present in a design section, the GFRP reinforcement should be neglected due to the drastic differences in design methodology between the two reinforcement materials. See section 2.6.2 for guidance on lapping steel and GFRP reinforcement.

2.5 DETAILS OF REINFORCEMENT

2.5.1 Concrete Cover (GFRP 2.11.1 & 5.6.2.3)

Concrete cover to the top mat of GFRP reinforcement shall be 1½". Concrete cover to the top mat of GFRP reinforcement shall be a minimum of 1", however 1½" is preferred if the design and rebar spacing is not affected by the additional ½" of cover. The increased cover compared to GFRP 5.6.2.3 is to provide increased durability of the deck, to prevent splitting and cracking, decrease development lengths, and to account for concrete and reinforcement placement tolerances. Modification factors for water/cement ratios identified in LRFD 5.12.3 shall not be incorporated.

2.5.2 Bar Size (GFRP 4.5.4)

Bar sizes vary slightly by manufacturer in actual diameter and shape. However, all manufacturers provide bar sizes that generally conform to the nominal diameter and areas of standard steel reinforcement bars. Standard nominal bar sizes may be used in design and slight variations can be neglected.

The designer should attempt to limit bar sizes to #6 or less since bars larger than #6 may not be available from all manufacturers. In addition, actual tensile strength generally decreases with an increase in bar size, and cover and spacing requirements are increasingly difficult to satisfy with increased bar sizes.

2.5.3 Hooks, Bends, and Anchors (GFRP 2.11.2, 2.12.2.3, & 2.12.3)

Bar bends of GFRP reinforcement can only be produced in specific configurations and with limited lengths. Most manufacturers can only bend bars in one direction (i.e. clockwise or counterclockwise) and the longest leg on the bent bar cannot be more than about 6 feet. Bends may also require increased concrete cover (GFRP 5.6.4), depending on the bar size and location of the bend. Due to GFRP reinforcement

fabrication process, GFRP bars are bent immediately after pultrusion and prior to resin hardening using jigs to ensure proper geometry. Specific limitations on individual shapes may be obtained directly from manufacturers. A link to one manufacturer's bending limitations may be found in the References [V-ROD Bend Guideline]. In general, short and relatively simple bent bars may be detailed using GFRP, but complex or longer bent bars must be detailed using stainless steel reinforcement. The Designer is encouraged to consider the use of bent GFRP bars in corrosion prone areas such as edge beams at expansion joints and bars passing from the deck into an integral abutment (see Figure 2.5.3-1).

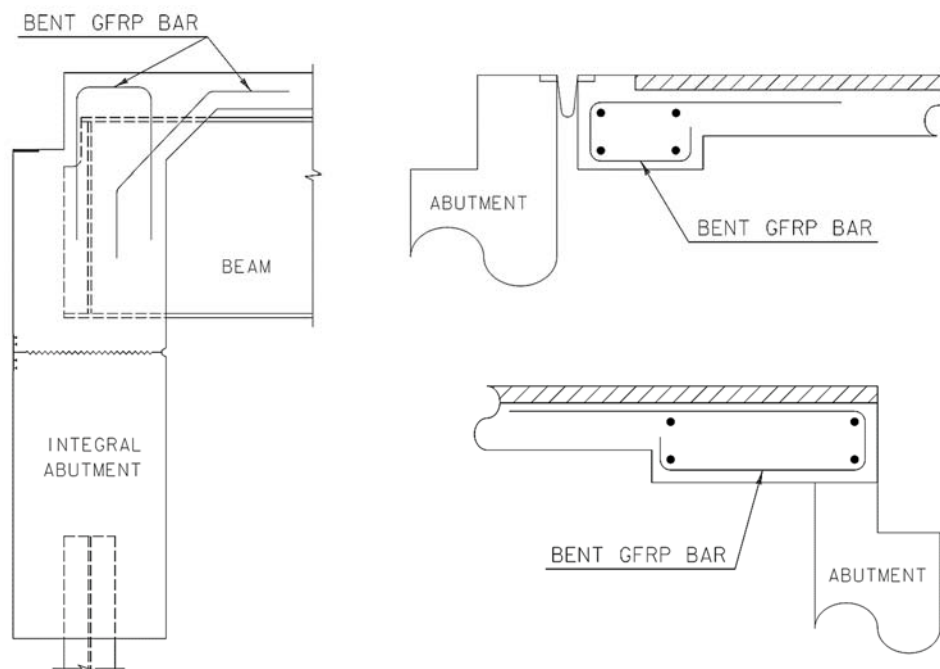


Figure 2.5.3-1 Suggested locations to consider the use of bent GFRP reinforcing bars.

Bar bends result in localized fiber buckling and redirection of fibers within the reinforcement, which reduces the bar strength at the bend location. Unlike steel reinforcement, the introduction of a hook at the end of GFRP reinforcement provides little reduction in development length. Basic development lengths for GFRP reinforcement with hooked ends are typically about twice that of an uncoated steel bar. Current GFRP Provisions also do not include reduction to hooked bar development lengths for side cover, end cover, or demand-to-capacity ratios.

Select manufacturers offer formed anchors to reduce development lengths. Anchor size, shape, and general performance vary by manufacturer and there is currently no standard method of testing their adequacy. Additionally, the requirement of headed bars within Contract Documents may eliminate some manufacturers from competing since not all offer similar anchor systems. Until test standardization or sufficient system performance records are available, it is recommended formed anchors not be used.

2.5.4 Minimum Negative Flexural Concrete Deck Reinforcement

The provisions of ASHTO LRFD subsection 6.10.1.7 shall directly apply to GFRP reinforced concrete decks without adjustment for differences in reinforcement elastic modulus values. Cracking over interior supports of continuous spans is most heavily influenced by primary girder stiffness and resulting curvature

under loading patterns and not by the type of reinforcement within the deck. Providing one percent reinforcement will help distribute the inevitable cracks regardless of differences in reinforcement material stiffness. [Frosch, 1999]

2.5.5 Field Bending and Cutting (*GFRP 5.6.4 & 5.8*)

Due to the brittle nature of GFRP as a material, reinforcing bars cannot be field bent or straightened without permanent damage and loss of bar capacity.

Elastic bending or flexing of straight bars (e.g.: to meet a curved horizontal alignment or traverse a roadway crown) is acceptable, but may require additional investigation by the designer. Field flexing of GFRP reinforcement will result in locked in stresses and therefore a reduction to the allowable capacity of the reinforcing bar for subsequent construction phases and design investigations. See Section 2.4.1 for additional guidance. No investigations or reductions in design strength are necessary for field flexing secondary reinforcement (e.g.: distribution reinforcement or shrinkage and temperature reinforcement). In all cases, the designer should ensure the field flexed bars have adequate length and/or tie-downs to maintain their shape through deck casting.

Field cutting of GFRP reinforcement should be avoided where possible. Field cutting can lead to frayed ends and wicking of moisture, thereby reducing service life of the bar through delamination. Therefore, the designer should avoid detailing GFRP reinforcement within the plans as “field cut.”

2.6 DEVELOPMENT AND SPLICES OF REINFORCEMENT

2.6.1 Development of Bars in Tension (*GFRP 2.12.2.1*)

Similar to recent revisions to steel development lengths in AASHTO, the development of GFRP reinforcement is a function of concrete strength, bar spacing and/or clear cover, and the ultimate stress demand within the bar. Notice that the reinforcement effective strength used in equation 2.12.2.1-1 is the actual stress present in the reinforcement at the design limit state, not the design strength of the GFRP material. Because of this, there is no excess reinforcement reduction factor for development length.

The July 2013 Errata to the GFRP Guide Specifications stated that the coefficient 31.6 in equation 2.12.2.1-1 should be removed – this is incorrect and the 2013 Errata should be ignored. The equation within the Errata was incorrectly changed to match ACI 440.1R reported equations which utilize units of psi [ACI 440.1R-15]. When using strengths of reinforcement and concrete denominated in ksi, the 31.6 coefficient should remain as this represents the fraction $1000 \div \sqrt{1000}$.

2.6.2 Lap Splices (*GFRP 2.12.4*)

Lap splices for GFRP reinforcement shall be in accordance with GFRP 2.12.4. Where possible, lap splices between adjacent bars or top and bottom mats of reinforcement should be staggered to ease congestion and better meet minimum spacing requirements.

When using a combination of steel and GFRP reinforcement, such as for the design of deck overhangs, lap splice lengths shall be the greater of either that for steel in accordance with AASHTO 5.11.5.3 or GFRP 2.12.4.

REFERENCES

Frosch, R. J. 1999. “Another Look at Cracking and Crack Control in Reinforced Concrete,” *ACI Structural Journal*. American Concrete Institute, Farmington Hills, MI, May –June. pp. 437-442.

Canadian Highway Bridge Design Code, CAN/CSA-S6-06 with Interims through 2013, Section 16 – Fibre-reinforced structures.

ACI 440.1R-15 Guide for the Design and Construction of Structural Concrete Reinforced with Fiber-Reinforced Polymer Bars, American Concrete Institute, Farmington Hills, MI, 2015.

V-ROD Bend Guideline, Pultrall Inc., Thetford Mines, Quebec, Canada, 2016.

<http://www.pultrall.com/dbfiles/Document/111201691450AMDOCen48.pdf>

SECTION 3: SPECIFICATIONS

3.1 GENERAL

The following subsections provide background and guidance used in the development of the current Special Provision and of the suggested design values presented within this Guide. Much of the information provided in this section is subject to change as a result of potential changes to manufacturing processes and availability.

3.2 MATERIAL PROPERTIES

3.2.1 Nominal Tensile Strength

Nominal tensile strength values reported by manufacturers vary by bar size, manufacturer, and grade. Table 4.2.1-1 summarizes values and availability of standard grade bars at the time of this publication. Some manufacturers may offer higher grade bars, but the designer is cautioned to pay specific attention to bar sizes and reporting methods when utilizing higher grade materials to ensure the values and geometries are consistent with calibration of design code requirements and load and resistance factors.

Table 4.2.1-1 – Nominal Tensile Strength Values, f_{tu} , ksi

Bar Size	Draft** ASTM Spec.	Fiberline ComBar	Pultrall V-ROD LM	Hughes Brothers Aslan 100	Pultron Composites MateenBar	Marshall Composite Technologies C-BAR	BP Composites TUF-BAR
#3	147	145	128	120	108	130	142
#4	108	145	145	110	108	105	122
#5	94	145	136	105	100	105	133
#6	93	145	136	100	100	101	128
#7	90	145	-	95	100	-	127
#8	85	145	139	90	94	-	118

**The second column in the table above shows the values from a Draft ASTM specification governing GFRP rebar that is currently in development. That ASTM standard is expected to be adopted sometime in 2017, and should help standardize GFRP rebar across the industry.

3.2.2 Nominal Tensile Strain

Nominal tensile strain values reported by respective manufacturers vary by bar size, manufacturer, and grade. Table 4.2.2-1 summarizes values and availability of standard grade bars at the time of this publication.

Table 4.2.2-1 – Nominal Tensile Strain Values, ϵ_{fu} , %

Bar Size	Fiberline ComBAR	Pultrall V-ROD LM	Hughes Brothers Aslan 100	Pultron Composites MateenBar	Marshall Composite Technologies C-BAR	BP Composites TUF-BAR
#3	1.67	2.07	1.79	1.226	2.00	2.3
#4	1.67	2.35	1.64	1.226	1.90	2.1
#5	1.67	2.21	1.57	1.226	1.95	2.1
#6	1.67	2.21	1.49	1.226	1.80	2.0
#7	1.67	-	1.42	1.226	-	1.9
#8	1.67	2.26	1.34	1.226	-	1.8

3.2.3 Modulus of Elasticity

Modulus of elasticity values reported by respective manufacturers vary by manufacturer and grade. Table 4.2.3-1 summarizes values and availability of standard grade bars at the time of this publication.

Table 4.2.2-1 – Nominal Tensile Strain Values, E_f , ksi

Bar Size	Fiberline ComBAR	Pultrall V-ROD LM	Hughes Brothers Aslan 100	Pultron Composites MateenBar	Marshall Composite Technologies C-BAR	BP Composites TUF-BAR
#3	8,700	6,150	6,700	7,680	7,600	7115
#4	8,700	6,150	6,700	7,680	6,600	6608
#5	8,700	6,150	6,700	7,390	7,500	7079
#6	8,700	6,150	6,700	7,390	7,100	7395
#7	8,700	-	6,700	7,390	-	7195
#8	8,700	6,150	6,700	7,390	-	7547

REFERENCES

V-ROD, Pultrall Inc., www.vrodcanada.com

Aslan 100, Hughes Brothers, Inc., www.aslanfrp.com

MateenBar, Pultron Composites, www.mateenbar.com

ComBAR, Fiberline Composites A/S, fiberline.com/structural-profiles/combar-fiberline

C-BAR, Marshall Composite Technologies LLC, www.marshallcomposite.com

TUF-BAR, BP Composites Ltd., www.bpcomposites.com/products/tuf-bar/

REINFORCING BAR, GFRP

- XX. DESCRIPTION. This work shall consist of furnishing and placing Glass Fiber Reinforced Polymer (GFRP) reinforcement bars in accordance with these provisions, the Plans, and manufacturer recommendations.
- XX. MATERIALS. GFRP reinforcement shall meet the requirements of the *AASHTO LRFD Bridge Design Guide Specifications for GFRP-Reinforced Concrete Bridge Decks and Traffic Railings* (November 2009), except as shown on the Plans and as stated herein. ACI 440.3R material tests noted in Subsection 4.3 of the aforementioned AASHTO guide specifications shall be considered mandatory tests, regardless of language used within the ACI test narrative. All GFRP reinforcement shall be deformed or sand coated.

GFRP bars shall be furnished according to the design values specified on the Plans. The following manufacturers and/or vendors, in no specific order, are considered pre-approved to supply GFRP reinforcement:

- (a) Aslan 100 by Hughes Brothers, Inc. (www.aslanfrp.com)
- (b) V-Rod by Pultrall Inc. (www.vrodcanada.com)
- (c) ComBar by Fiberline Composites, Inc. (fiberline.com/structural-profiles/combar-fiberline)
- (d) Mateen-bar from Sigma Development Group, LLC (www.mateenbar.com)
- (e) TUF-BAR by BP Composites Ltd. (www.bpcomposites.com/products/tuf-bar/)

Requests for alternate GFRP reinforcement products not identified in the above list shall be submitted for approval to the Agency's office of Contract Administration a minimum of 10 days in advance of the bid opening date. Substitutions after award shall be approved by the Engineer.

All GFRP bars within the same structural component shall be supplied by the same manufacturer; there shall be no mixing of products from different manufacturers in a component unless permitted on the Plans.

A Type D Certification shall be furnished in accordance with Subsection 700.02.

- XX. BAR LISTS. Bar lists and bending schedules shall be prepared by the vendor. It is the Contractor's responsibility to verify the vendor's bar lists and schedules for quantity, size, and shape of bar reinforcement for constructing the structural components shown in the Contract Documents or made a part thereof. If a Reinforcing Schedule is provided in the Plans, it is solely for the purpose of arriving at an estimated quantity and any errors shall not be considered cause for an adjustment of the Contract unit price. Upon delivery of the fabricated material, one copy of the shipping schedule and tabulation of lengths for each bar size shall be furnished to the Engineer.
- XX. FABRICATION AND SHIPMENT. Fabrication, forming, and tolerances of GFRP reinforcing bars shall be in conformance with the latest edition of the "Manual of Standard Practice of the Concrete Reinforcing Steel Institute" and the "Detailing Manual of the American Concrete Institute".

Bars loaded for transport shall be loaded and strapped down in a manner that will prevent damage from motion and vibration, to the greatest extent possible. Bundles of bent bars shall be transported strapped to wooden platforms or shall be crated. All individual bundles and layers of bundles shall be separated, and supported by dunnage.

- XX. PROTECTION OF MATERIAL. Delivery, storage, and handling of GFRP bars shall be in accordance with the manufacturer's instructions to prevent damage. The Contractor shall prevent bending, coating the bars with earth, oil, or other material, or otherwise damaging the GFRP reinforcement. When handling GFRP reinforcement, use equipment that avoids damaging or abrading the GFRP bar. Do not drop or drag GFRP reinforcement.

All handling of GFRP reinforcing bars by mechanical means shall be done by equipment having padded contact areas, or by the use of nylon webbing slings. The use of chains or wire rope slings will not be allowed, even when used with padding. All bundles of GFRP bars shall be lifted with a strong back, spreader bar, multiple supports or a platform bridge to prevent bar-to-bar abrasion from sags in the bundles. Support points during lifting or transporting of bundled GFRP reinforcing bars shall be spaced at a maximum of 15 ft, or as required by the manufacturer, whichever is more restrictive. Bundled bars shall be strapped together with non-metallic or padded straps in a manner to prevent bar-to-bar abrasion due to relative movement between bars.

Individual bars shall be handled in a manner that prevents damage to the coating due to abrasion or impact, and at no time shall any bar be moved by dragging over any surface, including other reinforcing bars. Sufficient personnel shall be assigned to assure that there is compliance with the above.

GFRP reinforcement shall be stored on skids or other supports a minimum of 12 inches above the ground surface and protected at all times from damage and surface contamination. The storage supports shall be constructed of wood or other material that will not damage the surface of the reinforcement or sand coating. Bundles of bars shall be stored on supports in a single layer. Each bundle shall be placed on the supports out of contact with adjacent bundles. GFRP shall be covered to avoid exposure to ultraviolet light. Prevent exposure of GFRP to temperatures above 120 degrees Fahrenheit during storage.

All damaged bars shall be repaired in accordance with manufacturer recommendations and inspected and accepted by the Engineer prior to placing concrete. All bars with total damage, including previously repaired areas, greater than 2 percent of the bar surface area shall be rejected. The depth of the permissible damage shall not exceed 0.04 inches.

- XX. PLACING AND FASTENING. GFRP reinforcement shall be accurately placed in the positions shown on the Plans and firmly held there during the placing and initial setup of the concrete. The bars must be adequately supported or tied to resist settlement, floating upward, or movement in any direction during concrete placement. Field bending of GFRP will not be allowed.

Field cutting of GFRP will be permitted only with the approval of the Engineer. Field cutting shall be performed with a high speed cutter, fine blade saw, diamond blade, or masonry saw. GFRP bars shall not be shear cut. The ends of all field cut bars shall be treated per the manufacturer's recommendations.

Proper distances from the forms shall be maintained by means of stays, blocks, ties, hangers or other approved means. GFRP reinforcing bars supported on formwork shall rest on non-metallic bar supports or other acceptable materials; wire bar supports will not be allowed. Blocks used for this purpose shall be precast Portland cement mortar blocks of approved shape and dimensions. Chairs used for this purpose must be GFRP or plastic. Layers of bars may be separated by precast Portland cement mortar blocks or other approved devices. Reinforcing bars used as support bars shall be GFRP or Level III Reinforcing Steel in accordance with Section 507. The use of pebbles, pieces of broken stone or brick, metal pipe or wooden blocks will not be allowed. The placing of reinforcement as concrete placement progresses, without definite and secure means of holding the bar in its correct position, will not be allowed.

Bars shall be fastened together at all intersections except where spacing is less than 1 ft in either direction, in which case, fastening at alternate intersections of each bar with other bars will be permitted provided this will hold all the bars securely in position. Tie wire for GFRP reinforcing bars shall be soft annealed wire that has been nylon, epoxy or plastic coated.

Immediately before placing concrete, GFRP reinforcement shall be free from all foreign material, which could decrease the bond between the GFRP and concrete. Such foreign material shall include, but not be limited to dirt, paint, oil, bitumen and dried concrete mortar. Reinforcement shall be inspected and approved by the Engineer before any concrete is placed.

- XX. SPLICING. Reinforcing bars shall be spliced in accordance with the requirements of this section, and in the locations shown on the Plans. No modifications of, or additions to, the splice arrangements shown on the Plans shall be made without the Engineer's prior approval.

Any additional splices authorized shall be staggered as much as possible. All splices shall be made in a manner that will ensure that not less than 75% of the clear concrete cover and not less than 75% of the minimum clear distance to other bars will be maintained, as compared to the cover and clear distance requirements for the unspliced bar.

Lapped splices shall be made by placing the bars in contact and wiring/tying them together. Splice laps shall be made in accordance with the plans.

- XX. METHOD OF MEASUREMENT. The quantity of Special Provision (Reinforcing Bar, GFRP) to be measured for payment will be the number of linear feet of reinforcement authorized for use in the complete and accepted work. Measurement for payment will not be made for any bar supports, wire, or other material that may be used by the Contractor for keeping the reinforcing bars in their correct position.

When the substitution of bars of greater diameter than specified is permitted by written authorization of the Engineer, measurement will be made as if the specified diameter had been used. In case short, spliced bars are used where full length bars are shown on the Plans, the length to be measured will be only the equivalent of the length of the full length bars as if they had been used, with no allowance for laps.

- XX. BASIS OF PAYMENT. The accepted quantity of Special Provision (Reinforcing Bar, GFRP) will be paid for at the Contract unit price per linear foot. Payment will be full compensation for detailing, furnishing, handling, and placing the material specified and for furnishing all labor, fastening devices, tools, equipment, and incidentals necessary to complete the work.

Payment will be made under:

<u>Pay Item</u>		<u>Pay Unit</u>
900.640 Special Provision (Reinforcing Bar, GFRP)	(#5)	Linear Foot
900.640 Special Provision (Reinforcing Bar, GFRP)	(#6)	Linear Foot

PROJECT: Example Project
JOB NO: Project #

DESIGN: X.Xxx
CHECK: Y. Yyy

DATE:
DATE:

GFRP Interior Deck Design

References:

1. AASHTO LRFD Bridge Design Specifications, 7th Edition, 2014, with interims through 2016 [Referenced as "AASHTO"]
2. AASHTO LRFD Bridge Design Guide Specifications for GFRP-Reinforced Concrete Bridge Decks and Traffic Railings [Referenced as "GFRP"]
3. VTrans Structures Design Manual, 2010, with revisions through 2014 [Referenced as "SDM"]
4. VTrans Standard Specifications for Construction, 2011
5. VTrans GFRP Reinforced Deck Design Guidelines, 2016 [Referenced as "Guidelines"]

Assumptions/Notes:

1. Deck is idealized as a 4 span continuous beam, spanning across the 5 beam lines.
2. GRFP properties taken as least values from VTrans GFRP Reinforced Deck Design Guidelines, up to #6 bar.

Inputs are in Yellow

Results are in Green

Geometry:

$t_{\text{slab}} := 8.5\text{in}$

Thickness of Deck Slab, Structural Component

$t_{\text{dw}} := 3\text{in}$

Thickness of the Wearing Surface

$\text{skew} := 15\text{deg}$

Maximum Skew Angle of Reinforcement to Girders

$S_{\text{gird.normal}} := 7.25\text{ft}$

Girder Spacing, Maximum, Normal to Girders

$S_{\text{gird}} := \frac{S_{\text{gird.normal}}}{\cos(\text{skew})} = 7.51 \cdot \text{ft}$

Girder Spacing Along Reinforcement

$t_{\text{w}} := 0.625\text{in}$

Web Thickness of Girder

$b_{\text{f}} := 20\text{in}$

Width of Girder Top Flange

$\text{UW} := 1\text{ft}$

Unit Width for Analysis

$\text{Span}_{1_3} := 30\text{ft}$ $\text{Span}_2 := 160\text{ft}$

Span Lengths, Bearing to Bearing

Reinforcement Geometry:

$$\text{Cover}_{\text{top}} := 1.5\text{in}$$

Top Clear Cover of Concrete Deck (Use same as bottom clear cover since reinforcement is non-corrosive) [Guidelines 2.5.1]

$$\text{Cover}_{\text{bot}} := 1.5\text{in}$$

Bottom Clear Cover of Concrete Deck (SDM Table 5.1.2.6-1)

If the skew angle of the deck does not exceed 25 degrees, the primary reinforcement may be placed in the direction of the skew; otherwise, it shall be placed perpendicular to the main supporting components. Skewed Decks [AASHTO 9.7.1.3]

Main (Transverse) GFRP Reinforcement:

$$D_{\text{trans.top}} := 0.75\text{in}$$

Reinforcing Bar Diameter

$$A_{\text{trans.top}} := \pi D_{\text{trans.top}}^2 \div 4 = 0.44 \cdot \text{in}^2$$

Reinforcing Bar Area

$$S_{\text{trans.top}} := 5.0\text{in}$$

Bar Spacing of Top Transverse Reinforcement (perpendicular to the bar)(Bars placed along skew for bridge skews less than 25 degrees.)

$$D_{\text{trans.bot}} := 0.75\text{in}$$

Reinforcing Bar Diameter

$$A_{\text{trans.bot}} := \pi D_{\text{trans.bot}}^2 \div 4 = 0.44 \cdot \text{in}^2$$

Reinforcing Bar Area

$$S_{\text{trans.bot}} := 5.0\text{in}$$

Bar Spacing of Bottom Transverse Reinforcement (perpendicular to the bar)(Bars placed along skew for bridge skews less than 25 degrees.)

Secondary (Longitudinal) GFRP Reinforcement:

$$D_{\text{long.top}} := 0.625\text{in}$$

Reinforcing Bar Diameter

$$A_{\text{long.top}} := \pi D_{\text{long.top}}^2 \div 4 = 0.31 \cdot \text{in}^2$$

Reinforcing Bar Area

$$S_{\text{long.top}} := 9\text{in}$$

Bar Spacing of Top Longitudinal Reinforcement (perpendicular to the bar)

$$D_{\text{long.bot}} := 0.625\text{in}$$

Reinforcing Bar Diameter

$$A_{\text{long.bot}} := \pi D_{\text{long.bot}}^2 \div 4 = 0.31 \cdot \text{in}^2$$

Reinforcing Bar Area

$$S_{\text{long.bot}} := 5\text{in}$$

Bar Spacing of Bottom Longitudinal Reinforcement (perpendicular to the bar)

$$d' := \text{Cover}_{\text{top}} + D_{\text{trans.top}} \div 2$$

$$d' = 1.87 \cdot \text{in}$$

Distance to Top Layer of Main Reinforcement

$$d := t_{\text{slab}} - \text{Cover}_{\text{bot}} - (D_{\text{trans.bot}} \div 2)$$

$$d = 6.62 \cdot \text{in}$$

Distance to Bottom Layer of Main Reinforcement

Material Properties:

Designer Note: GFRP Reinforced Deck Design Guidelines Section 2.3.2: The designer should investigate current values for Nominal Ultimate Tensile Strength, Modulus of Elasticity, and Nominal Tensile Strain at the beginning of each new design regardless of the suggested design values provided. The nominal strength and strain used in design should be the minimum value reported by at least three manufacturers.

$$f_{fu} := 100\text{ksi}$$

Nominal Ultimate Tensile Strength of GFRP Bars for Product Certification [Guidelines 2.3.2]

$$E_f := 6150\text{ksi}$$

Modulus of Elasticity of GFRP [Guidelines 2.3.3]

$$\epsilon_{fu} := 0.01226$$

Nominal Tensile Strain of GFRP Bars for Product Certification [Guidelines 2.3.2]

$$f'_c := 4\text{ksi}$$

Concrete Strength [SDM Table 5.1.1.1-1]

$$C_E := 0.7$$

Environmental Reduction Factor, Concrete Exposed to Earth and Weather [Guidelines 2.3.2 and GFRP T2.6.1.2-1]

$$f_{fd} := C_E \cdot f_{fu} = 70 \cdot \text{ksi}$$

Design Tensile Strength of GFRP Bar [GFRP 2.6.1.2-1]

$$\epsilon_{fd} := C_E \cdot \epsilon_{fu} = 0.0086$$

Design Tensile Strain of GFRP Bars [GFRP 2.6.1.2-1]

$$k_b := 1.0$$

Bond Dependent Coefficient [Guidelines 2.4.2.3 and GFRP 2.9.3.4]

$$E_c := 2500 \left(\frac{f'_c}{\text{ksi}} \right)^{0.33} \text{ ksi} = 3950 \cdot \text{ksi}$$

Modulus of Elasticity of Concrete [AASHTO C5.4.2.4-1]

$$f_r := 0.24 \cdot \sqrt{f'_c \cdot \text{ksi}} = 0.48 \cdot \text{ksi}$$

Modulus of Rupture of Concrete [AASHTO 5.4.2.6]

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

Ultimate Strain in Concrete

$$\gamma_c := \begin{cases} 150\text{pcf} & \text{if } (f'_c \leq 5\text{ksi}) \\ \left(145 + \frac{f'_c}{\text{ksi}} \right) \cdot \text{pcf} & \text{otherwise} \end{cases}$$

Unit weight of Concrete and Wearing Surface [SDM Table 3.3.1-1]

$$\gamma_c = 150 \cdot \text{pcf}$$

$$\gamma_{bit} := 150\text{pcf}$$

$$E_s := 29000\text{ksi}$$

Modulus of Elasticity of Steel Reinforcement

$$n := \frac{E_f}{E_c} = 1.56$$

Modular ratio

$$\beta_1 := \begin{cases} 0.85 & \text{if } f_c \leq 4\text{ksi} \\ \max\left[0.85 - 0.05 \cdot \left(\frac{f_c - 4\text{ksi}}{\text{ksi}}\right), 0.65\right] & \text{otherwise} \end{cases} \quad \beta_1 = 0.85$$

Stress Block to N.A. Parameter [GFRP 2.9.3.1]

[AASHTO 5.7.2.2]

$$\rho_{fb} := 0.85 \cdot \beta_1 \cdot \frac{f_c}{f_{fd}} \cdot \left(\frac{E_f \cdot \epsilon_{cu}}{E_f \cdot \epsilon_{cu} + f_{fd}} \right) = 0.0086$$

*Balanced Reinforcement Ratio
[GFRP Eq 2.7.4.2-2]*

Effect of Imposed Deformation:

Curvature at the crown of a bridge will impose stress into the GFRP bar, reducing capacity.

$$D_{\text{trans.top}} = 0.75 \text{ in}$$

$$D_{\text{trans.bot}} = 0.75 \text{ in}$$

Diameter of the transverse reinforcement

$$\theta_{\text{crown}} := 4\%$$

Algebraic difference of slopes on either side of the crown break

$$R_{\text{crown}} := \frac{4 \text{ in}}{3 \cdot \theta_{\text{crown}}^2} = 69.44 \cdot \text{ft}$$

*Approximate radius of the rebar bend at the crown
[Guidelines 2.4.1-3]*

Top Transverse Reinforcement

$$R_{e.\text{top}} := \frac{D_{\text{trans.top}}}{2 \cdot R_{\text{crown}}} = 0.00045$$

*Strain reduction due to field flexing at the crown
[Guidelines 2.4.1-2]*

$$R_{f.\text{top}} := \frac{D_{\text{trans.top}}}{2 \cdot R_{\text{crown}}} \cdot E_f = 2.77 \cdot \text{ksi}$$

*Stress reduction due to field flexing at the crown
[Guidelines 2.4.1-1]*

$$f_{fd.r.\text{top}} := C_E \cdot f_{fu} - R_{f.\text{top}} = 67.23 \cdot \text{ksi}$$

*Reduced Design Tensile Strength of GFRP Bars ,
accounting for field flexing [Guidelines 2.3.2]*

$$\epsilon_{fd.r.\text{top}} := C_E \cdot \epsilon_{fu} - R_{e.\text{top}} = 0.00813$$

*Reduced Design Strain of GFRP Bars, accounting
for field flexing [Guidelines 2.3.2]*

Bottom Transverse Reinforcement

$$R_{e.\text{bot}} := \frac{D_{\text{trans.bot}}}{2 \cdot R_{\text{crown}}} = 0.00045$$

*Strain reduction due to field flexing at the crown
[Guidelines 2.4.1-2]*

$$R_{f.\text{bot}} := \frac{D_{\text{trans.bot}}}{2 \cdot R_{\text{crown}}} \cdot E_f = 2.77 \cdot \text{ksi}$$

*Stress reduction due to field flexing at the crown
[Guidelines 2.4.1-1]*

$$f_{fd.r.\text{bot}} := C_E \cdot f_{fu} - R_{f.\text{bot}} = 67.23 \cdot \text{ksi}$$

*Reduced Design Tensile Strength of GFRP Bars ,
accounting for field flexing [Guidelines 2.3.2]*

$$\epsilon_{fd.r.\text{bot}} := C_E \cdot \epsilon_{fu} - R_{e.\text{bot}} = 0.00813$$

*Reduced Design Strain of GFRP Bars, accounting
for field flexing [Guidelines 2.3.2]*

Loads & Moments:

Dead Loads:

$$DW := (t_{dw}) \cdot \gamma_{bit} \cdot UW$$

$$DW = 37.5 \cdot \text{plf} \quad \text{Uniform Load of Wearing Surface}$$

$$DC := \gamma_c \cdot (t_{slab}) \cdot UW$$

$$DC = 106.25 \cdot \text{plf} \quad \text{Uniform Load due to Deck Slab Self Weight}$$

Base dead load moments on continuous 4 span girder model with uniformly distributed load (ignore presence of overhangs - slightly conservative)

[AISC Manual Table 3-22c]

$$M_{DW.neg} := 0.107DW \cdot S_{gird}^2$$

$$M_{DW.neg} = 0.23 \cdot \text{kip} \cdot \text{ft}$$

Max Negative Moment due to DW

$$M_{DW.pos} := 0.0772DW \cdot S_{gird}^2$$

$$M_{DW.pos} = 0.16 \cdot \text{kip} \cdot \text{ft}$$

Max Positive Moment due to DW

$$M_{DC.neg} := 0.107DC \cdot S_{gird}^2$$

$$M_{DC.neg} = 0.64 \cdot \text{kip} \cdot \text{ft}$$

Max Negative Moment due to DC

$$M_{DC.pos} := 0.0772DC \cdot S_{gird}^2$$

$$M_{DC.pos} = 0.46 \cdot \text{kip} \cdot \text{ft}$$

Max Positive Moment due to DC

Live Loads:

- Utilize AASHTO 4.6.2.1.6 and Table A4-1 (Table data is in the following Area):

AASHTO Table A4-1 Data

Functions to lookup and interpolate the Moments from AASHTO Table A4-1

$$M_{pos.function}(S_{gird.in}) := \text{linterp}(S_{lookup}, M_{pos.table}, S_{gird.in})$$

Simple linear interpolation of the AASHTO Table A4-1 positive moment vector

$$M_{neg.function}(S_{gird.in}, b_{neg.in}) := \begin{cases} \text{for } col \in 0 \dots \text{cols}(M_{neg.table}) - 1 \\ M_{neg.S.gird.col} \leftarrow \text{linterp}(S_{lookup}, M_{neg.table}^{(col)}, S_{gird.in}) \\ \text{return } \text{linterp}(b_{neg.lookup}, M_{neg.S.gird}, b_{neg.in}) \end{cases}$$

Nested linear interpolation to lookup and interpolate in both directions in the AASHTO Table A4-1 negative moment matrix. First each column is evaluated by itself based on the input S_{gird} . Then, the output vector from that is interpolated based on the input b_{neg} .

$$b_{neg} := 0.25 \cdot \frac{b_f}{\cos(\text{skew})} = 5.18 \text{ in}$$

Location of Design Section for Negative Moment in Deck [AASHTO 4.6.2.1.6]

- Interpolate in Table A4-1 for Moments

$$M_{LL.pos} := M_{pos.function}(S_{gird}) = 5.44 \cdot \text{kip} \cdot \text{ft}$$

Max Positive LL moment with Impact and Multiple Presence Factors Included

$$M_{LL.neg} := M_{neg.function}(S_{gird}, b_{neg}) = 4.84 \cdot \text{kip} \cdot \text{ft}$$

Max Negative LL moment with Impact and Multiple Presence Factors Included

Strength I Design Moment:

$$\gamma_{DC} := 1.25 \quad \gamma_{DW} := 1.50 \quad \gamma_{LL} := 1.75$$

*Load Factors for Dead Load (DC), Dead Loads (DW), and Live Load (LL)
[AASHTO Table 3.4.1-1, Table 3.4.1-2]*

$$M_{u,pos} := \gamma_{DC} \cdot M_{DC,pos} + \gamma_{DW} \cdot M_{DW,pos} + \gamma_{LL} \cdot M_{LL,pos} = 10.35 \cdot \text{kip} \cdot \text{ft} \quad \textit{Ultimate Positive Moment, Strength I}$$

$$M_{u,neg} := \gamma_{DC} \cdot M_{DC,neg} + \gamma_{DW} \cdot M_{DW,neg} + \gamma_{LL} \cdot M_{LL,neg} = 9.61 \cdot \text{kip} \cdot \text{ft} \quad \textit{Ultimate Negative Moment, Strength I}$$

Service I Design Moment:

$$\gamma_{DC,s} := 1.0 \quad \gamma_{DW,s} := 1.0 \quad \gamma_{LL,s} := 1.0$$

*Load Factors for Dead Load (DC), Dead Loads (DW), and Live Load (LL)
[AASHTO Table 3.4.1-1, Table 3.4.1-2]*

$$M_{s,pos} := \gamma_{DC,s} \cdot M_{DC,pos} + \gamma_{DW,s} \cdot M_{DW,pos} + \gamma_{LL,s} \cdot M_{LL,pos} = 6.07 \cdot \text{kip} \cdot \text{ft} \quad \textit{Service I Positive Moment}$$

$$M_{s,neg} := \gamma_{DC,s} \cdot M_{DC,neg} + \gamma_{DW,s} \cdot M_{DW,neg} + \gamma_{LL,s} \cdot M_{LL,neg} = 5.7 \cdot \text{kip} \cdot \text{ft} \quad \textit{Service I Negative Moment}$$

$$M_{s,pos,DL} := \gamma_{DC,s} \cdot M_{DC,pos} + \gamma_{DW,s} \cdot M_{DW,pos} = 0.63 \cdot \text{kip} \cdot \text{ft} \quad \textit{Service I Positive Dead Load Moment}$$

$$M_{s,neg,DL} := \gamma_{DC,s} \cdot M_{DC,neg} + \gamma_{DW,s} \cdot M_{DW,neg} = 0.87 \cdot \text{kip} \cdot \text{ft} \quad \textit{Service I Negative Dead Load Moment}$$

Check Main Reinforcement (Positive Moment):

$$A_{f,\text{pos}} := A_{\text{trans.bot}} \cdot \frac{UW}{S_{\text{trans.bot}}} = 1.06 \cdot \text{in}^2$$

Tensile Reinforcement per unit width

$$d_{T,\text{pos}} := d = 6.62 \text{ in} \quad S_{T,\text{pos}} := S_{\text{trans.bot}} = 5 \cdot \text{in}$$

Depth to Tension GFRP Reinforcement from Top of Section and Spacing of Tension GFRP Reinforcement

$$M_{u,\text{pos}} = 10.35 \cdot \text{kip} \cdot \text{ft}$$

Ultimate Moment

$$M_{s,\text{pos}} = 6.07 \cdot \text{kip} \cdot \text{ft}$$

Service Moment

$$\rho_{f,\text{pos}} := \frac{A_{f,\text{pos}}}{UW \cdot d_{T,\text{pos}}} = 0.01334 \quad \rho_{fb} = 0.00861$$

Reinforcement Ratio vs. Balanced Ratio

$$\phi_{\text{pos}} := \begin{cases} 0.55 & \text{if } \rho_{f,\text{pos}} \leq \rho_{fb} \\ 0.3 + 0.25 \cdot \frac{\rho_{f,\text{pos}}}{\rho_{fb}} & \text{if } \rho_{fb} < \rho_{f,\text{pos}} \leq 1.4 \cdot \rho_{fb} \\ 0.65 & \text{otherwise} \end{cases} = 0.65 \quad \text{Resistance Factor for Concrete in Flexure [GFRP 2.7.4.2]}$$

If $\rho_{f,\text{pos}} < \rho_{fb}$ is true the failure is initiated by rupture of

GFRP failure is initiated by crushing of the concrete, take the minimum between the equation and the design tensile strength as the effective strength of reinforcement

$$f_{f,\text{pos}} := \begin{cases} f_{fd,r,\text{bot}} & \text{if } \rho_{f,\text{pos}} < \rho_{fb} \\ \min \left[\sqrt{\frac{(E_f \cdot \epsilon_{cu})^2}{4} + \frac{0.85 \cdot \beta_1 \cdot f'_c}{\rho_{f,\text{pos}}} \cdot E_f \cdot \epsilon_{cu} - 0.5 E_f \cdot \epsilon_{cu}}, f_{fd,r,\text{bot}} \right] & \text{otherwise} \end{cases} = 54.67 \cdot \text{ksi} \quad \text{Stress in GFRP Bars [GFRP 2.9.3.1]}$$

$$M_{n,\text{pos}} := a \leftarrow \frac{A_{f,\text{pos}} \cdot f_{f,\text{pos}}}{0.85 \cdot f'_c \cdot UW} = 28.57 \cdot \text{kip} \cdot \text{ft} \quad \text{Nominal Flexural Resistance [GFRP 2.9.3.2.2]}$$

$$c_b \leftarrow \left(\frac{\epsilon_{cu}}{\epsilon_{cu} + \epsilon_{fd,r,\text{bot}}} \cdot d_{T,\text{pos}} \right) \quad \text{[GFRP EQ. 2.9.3.2.2-2] [GFRP EQ. 2.9.3.2.2-4]}$$

$$\left[A_{f,\text{pos}} \cdot f_{f,\text{pos}} \cdot \left(d_{T,\text{pos}} - \frac{a}{2} \right) \right] \quad \text{if } \rho_{f,\text{pos}} > \rho_{fb} \quad \text{If true the failure is initiated by concrete crushing [GFRP EQ. 2.9.3.2.2-1]}$$

$$\left[A_{f,\text{pos}} \cdot f_{fd,r,\text{bot}} \cdot \left(d_{T,\text{pos}} - \frac{\beta_1 \cdot c_b}{2} \right) \right] \quad \text{otherwise} \quad \text{If true the failure is initiated by rupture of GFRP [GFRP EQ. 2.9.3.2.2-3]}$$

$$M_{r,\text{pos}} := \phi_{\text{pos}} \cdot M_{n,\text{pos}} = 18.57 \cdot \text{kip} \cdot \text{ft} \quad \text{Factored Flexural Resistance [GFRP 2.9.3.2.1-1]}$$

Check₁ := if (M_{r,pos} ≥ M_{u,pos}, "Deck is OK for Positive Moment Strength", "Deck is No Good for Positive Moment Strength")

Check₁ = "Deck is OK for Positive Moment Strength"

$$DC_Ratio_1 := \frac{M_{u,\text{pos}}}{M_{r,\text{pos}}} = 0.56$$

Failure_{pos} := if (ρ_{f,pos} > ρ_{fb}, "Concrete Crushing", "GFRP Rupture") = "Concrete Crushing"

Minimum Reinforcement:

$$A_{f,\text{min},\text{pos}} := \max \left(0.16 \cdot \sqrt{\frac{f'_c}{\text{ksi}}}, 0.33 \right) \cdot \frac{UW \cdot d_{T,\text{pos}}}{\left(\frac{f_{fd,r,\text{bot}}}{\text{ksi}} \right)} = 0.39 \text{ in}^2 \quad \text{Minimum Flexural Tension Reinforcement [GFRP 2.9.3.3]}$$

Check₂ := if (A_{f,pos} ≥ A_{f,min,pos}, "Min. Reinforcement is OK", "Min. Reinforcement is No Good")

Check₂ = "Min. Reinforcement is OK"

$$DC_Ratio_2 := \frac{A_{f,\text{min},\text{pos}}}{A_{f,\text{pos}}} = 0.37$$

Strain Compatibility at Service Stress:

The following is an elastic, straight-line theory approach found in many concrete design text books.

$$M_{s, \text{pos}} = 6.07 \cdot \text{kip} \cdot \text{ft}$$

Service Moment

$$A_{f, \text{pos}} = 1.06 \text{ in}^2$$

Area of GFRP per UW

$$k_{\text{pos}} := \sqrt{2\rho_{f, \text{pos}} \cdot n + (\rho_{f, \text{pos}} \cdot n)^2} - \rho_{f, \text{pos}} \cdot n = 0.18$$

Ratio of Depth of Neural Axis to Reinforcement Depth [GFRP 2.7.3-4]

$$j_{\text{pos}} := \left(1 - \frac{k_{\text{pos}}}{3}\right) = 0.94$$

Ratio of Lever Arm to Depth to Tension Reinforcement

$$f_{fs, \text{pos}} := \frac{M_{s, \text{pos}}}{A_{f, \text{pos}} \cdot j_{\text{pos}} \cdot d_{T, \text{pos}}} = 11.04 \cdot \text{ksi}$$

Stress in GFRP at service loads, in addition to any stresses induced by field flexing

$$f_{cs, \text{pos}} := \frac{2 \cdot M_{s, \text{pos}}}{j_{\text{pos}} \cdot k_{\text{pos}} \cdot UW \cdot d_{T, \text{pos}}^2} = 1.6 \cdot \text{ksi}$$

Stress in Concrete at service loads

$$na_{s, \text{pos}} := k_{\text{pos}} \cdot d_{T, \text{pos}} = 1.22 \text{ in}$$

Neutral axis of transformed cracked section

Crack Control Check:

$$w_{\text{max}} := 0.02 \text{ in}$$

Limiting Crack Width [GFRP 2.9.3.4]

$$d_{c, \text{pos}} := t_{\text{slab}} - d_{T, \text{pos}} = 1.88 \text{ in}$$

Distance to CG GFRP Reinforcement from tension face of member

$$\beta_{\text{pos}} := \frac{t_{\text{slab}} - na_{s, \text{pos}}}{t_{\text{slab}} - na_{s, \text{pos}} - d_{c, \text{pos}}} = 1.35$$

Crack Parameter (Ratio of dist between NA and T face to dist between NA and T reinf. [GFRP 2.9.3.4])

$$w_{\text{pos}} := 2 \cdot \frac{f_{fs, \text{pos}}}{E_f} \cdot \beta_{\text{pos}} \cdot k_b \cdot \sqrt{d_{c, \text{pos}}^2 + \frac{S_{T, \text{pos}}^2}{4}} = 0.015 \text{ in}$$

Crack Width at Service Limit State [GFRP 2.9.3.4-1]

$$\text{Check}_3 := \text{if}(w_{\text{pos}} < w_{\text{max}}, \text{"Crack Control is OK"}, \text{"Crack Control is No Good"})$$

[GFRP 2.9.3.4]

Check₃ = "Crack Control is OK"

$$\text{DC_Ratio}_3 := \frac{w_{\text{pos}}}{w_{\text{max}}} = 0.76$$

Deflection Limit Check:

$$I_g := \frac{UW \cdot t_{slab}^3}{12} = 614.13 \text{ in}^4$$

Gross Moment of Inertia

$$I_{cr,pos} := \frac{UW \cdot d_{T,pos}^3}{3} \cdot k_{pos}^3 + n \cdot A_{f,pos} \cdot d_{T,pos}^2 (1 - k_{pos})^2 = 55.49 \text{ in}^4$$

Cracked Moment of Inertia (GFRP 2.7.3-3)

$$M_{cr} := f_r \cdot \frac{I_g}{t_{slab} \div 2} = 5.78 \cdot \text{kip} \cdot \text{ft}$$

Cracking Moment (GFRP 2.9.3.5-2)

$$\text{Strip}_{width} := 26 \text{ in} + 6.6 \cdot \frac{S_{gird}}{\text{ft}} \cdot \text{in} = 75.54 \text{ in}$$

Equivalent strip width for use in calculating live load service moments [AASHTO Table 4.6.2.1.3-1]

$$P_{LL} := 16 \text{ kip} \cdot 1.2 \cdot 1.33 = 25.54 \cdot \text{kip}$$

Point live load for calculating deflection, including multiple presence factor. AASHTO design truck, no lane, with impact [AASHTO 3.6.1.3.2]

$$M_a := M_{s,pos} = 6.07 \cdot \text{kip} \cdot \text{ft}$$

Maximum Moment Concurrent with Deflection being Investigated [GFRP 2.9.3.5]

$$\beta_d := \min\left(\frac{1}{5} \cdot \frac{\rho_{f,pos}}{\rho_{fb}}, 1\right) = 0.31$$

Reduction Factor used for Effective Moment of Inertia Determination [GFRP 2.9.3.5-2]

$$I_{e,pos} := \text{if}\left[M_{cr} \geq M_a, I_g, \min\left[I_g, \left[\left(\frac{M_{cr}}{M_a}\right)^3 \cdot \beta_d \cdot I_g + \left[1 - \left(\frac{M_{cr}}{M_a}\right)^3\right] \cdot I_{cr,pos}\right]\right]\right] = 171.93 \cdot \text{in}^4$$

Effective Moment of Inertia in Cracked Section (GFRP 2.9.3.5-1)

$$\Delta_{max} := \frac{S_{gird}}{800} = 0.113 \text{ in}$$

Deflection Limit [GFRP 2.9.3.5, AASHTO 9.5.2]

$$\Delta_{s,LL} := \frac{0.015 P_{LL} \cdot S_{gird}^3}{E_c \cdot I_{e,pos} \cdot UW} = 0.0655 \text{ in}$$

LL Deflection at Service Limit (Ref: 2 span single point loaded beam table)

AISC Table 3-23-30: Two equal spans - Concentrated load at center of one

$$\Delta_{s,DL} := \frac{0.0065(DC + DW) \cdot S_{gird}^4}{E_c \cdot I_{e,pos}} = 0.0075 \text{ in}$$

DL Deflection at Service Limit (Ref: 4 span uniformly loaded beam table)

AISC Table 3-23-42: Continuous beam - four equal spans - all spans loaded

$$\Delta_{LT} := \text{if}(I_{e,pos} = I_g, 4, 3) \cdot (\Delta_{s,DL}) = 0.023 \cdot \text{in}$$

Long Term deflection, accounting for creep from sustained loads [Guidelines 2.4.2.4][GFRP 2.9.3.5]

$$\text{Check}_4 := \text{if}(\Delta_{LT} + \Delta_{s,LL} < \Delta_{max}, \text{"Slab Deflection is OK"}, \text{"Slab Deflection is No Good"})$$

[GFRP 2.9.3.5]

Check₄ = "Slab Deflection is OK"

$$\text{DC_Ratio}_4 := \max\left(\frac{\Delta_{LT} + \Delta_{s,LL}}{\Delta_{max}}\right) = 0.78$$

Stress Limit for Concrete Check:

$$f_{cs,pos} = 1.6 \cdot \text{ksi}$$

Max Concrete Stress at Service Limit

$$\text{Check}_5 := \text{if}(f_{cs,pos} < 0.45 \cdot f_c, \text{"Concrete Service Stress is OK"}, \text{"Concrete Service Stress is No Good"}) \quad [\text{GFRP 2.9.3.6}]$$

Check₅ = "Concrete Service Stress is OK"

$$\text{DC_Ratio}_5 := \frac{f_{cs,pos}}{0.45 \cdot f_c} = 0.89$$

Creep Rupture Check:

$$I_{cr,pos} = 55.49 \text{ in}^4$$

Cracked Moment of Inertia [GFRP 2.7.3-3]

$$f_{creep,pos} := \frac{n \cdot d_{T,pos} \cdot (1 - k_{pos})}{I_{cr,pos}} \cdot M_{s,pos} + R_{f,bot} = 13.81 \cdot \text{ksi}$$

Sustained force in rebar at Service I limit state [GFRP 2.7.3-2]

$$f_{creep,limit} := 0.2 \cdot f_{fd} = 14 \cdot \text{ksi}$$

Allowable sustained stress in rebar from dead loads, creep rupture limit [GFRP 2.7.3-1]

$$\text{Check}_6 := \text{if}(f_{creep,pos} < f_{creep,limit}, \text{"Creep Rupture Stress is OK"}, \text{"Creep Rupture Stress is No Good"}) \quad [\text{GFRP 2.9.3.6}]$$

Check₆ = "Creep Rupture Stress is OK"

$$\text{DC_Ratio}_6 := \frac{f_{creep,pos}}{f_{creep,limit}} = 0.99$$

Check Main Reinforcement (Negative Moment):

$$A_{f.neg} := A_{trans.top} \cdot \frac{UW}{S_{trans.top}} = 1.06 \text{ in}^2 \quad \text{Tensile Reinforcement per unit width}$$

$$d_{T.neg} := t_{slab} - d' = 6.62 \text{ in} \quad S_{T.neg} := S_{trans.top} = 5 \text{ in} \quad \text{Depth to Tension GFRP Reinforcement from Bottom of Section and Spacing of Tension GFRP Reinforcement}$$

$$M_{u.neg} = 9.61 \cdot \text{kip} \cdot \text{ft} \quad \text{Ultimate Moment}$$

$$M_{s.neg} = 5.7 \cdot \text{kip} \cdot \text{ft} \quad \text{Service Moment}$$

$$\rho_{f.neg} := \frac{A_{f.neg}}{UW \cdot d_{T.neg}} = 0.01334 \quad \rho_{fb} = 0.00861 \quad \text{Reinforcement Ratio vs. Balanced Ratio}$$

$$\phi_{neg} := \begin{cases} 0.55 & \text{if } \rho_{f.neg} \leq \rho_{fb} \\ 0.3 + 0.25 \cdot \frac{\rho_{f.neg}}{\rho_{fb}} & \text{if } \rho_{fb} < \rho_{f.neg} \leq 1.4 \cdot \rho_{fb} \\ 0.65 & \text{otherwise} \end{cases} = 0.65 \quad \text{Resistance Factor for Concrete in Flexure [GFRP 2.7.4.2]}$$

$$f_{f.neg} := \begin{cases} f_{fd_r.top} & \text{if } \rho_{f.neg} < \rho_{fb} \\ \min \left[\sqrt{\frac{(E_f \cdot \epsilon_{cu})^2}{4} + \frac{0.85 \cdot \beta_1 \cdot f'_c}{\rho_{f.neg}} \cdot E_f \cdot \epsilon_{cu}} - 0.5 E_f \cdot \epsilon_{cu}, f_{fd_r.top} \right] & \text{otherwise} \end{cases} = 54.67 \cdot \text{ksi} \quad \text{Stress in GFRP Bars [GFRP 2.9.3.1]}$$

$$M_{n.neg} := \begin{cases} a \leftarrow \frac{A_{f.neg} \cdot f_{f.neg}}{0.85 \cdot f'_c \cdot UW} = 28.57 \cdot \text{kip} \cdot \text{ft} & \text{Nominal Flexural Resistance [GFRP 2.9.3.2.2]} \\ c_b \leftarrow \left(\frac{\epsilon_{cu}}{\epsilon_{cu} + \epsilon_{fd_r.top}} \cdot d_{T.neg} \right) & \text{[GFRP EQ. 2.9.3.2.2-2]} \\ \left[A_{f.neg} \cdot f_{f.neg} \cdot \left(d_{T.neg} - \frac{a}{2} \right) \right] & \text{if } \rho_{f.neg} > \rho_{fb} \quad \text{If true the failure is initiated by concrete crushing [GFRP EQ. 2.9.3.2.2-1]} \\ \left[A_{f.neg} \cdot f_{fd_r.top} \cdot \left(d_{T.neg} - \frac{\beta_1 \cdot c_b}{2} \right) \right] & \text{otherwise} \quad \text{If true the failure is initiated by rupture of GFRP [GFRP EQ. 2.9.3.2.2-3]} \end{cases}$$

$$M_{r.neg} := \phi_{neg} \cdot M_{n.neg} = 18.57 \cdot \text{kip} \cdot \text{ft} \quad \text{Factored Flexural Resistance [GFRP 2.9.3.2.1-1]}$$

$$\text{Check}_7 := \text{if} (M_{r.neg} \geq M_{u.neg}, \text{"Deck is OK for Negative Moment Strength"}, \text{"Deck is No Good for Negative Moment Strength"})$$

Check₇ = "Deck is OK for Negative Moment Strength"

$$DC_Ratio_7 := \frac{M_{u.neg}}{M_{r.neg}} = 0.52$$

$$\text{Failure}_{neg} := \text{if} (\rho_{f.neg} > \rho_{fb}, \text{"Concrete Crushing"}, \text{"GFRP Rupture"}) = \text{"Concrete Crushing"}$$

Minimum Reinforcement:

$$A_{fmin.neg} := \max\left(0.16 \cdot \sqrt{\frac{f'_c}{\text{ksi}}}, 0.33\right) \cdot \frac{UW \cdot d_{T.neg}}{\left(\frac{f_{fd.r.top}}{\text{ksi}}\right)} = 0.39 \text{ in}^2$$

Minimum Flexural Tension Reinforcement
[GFRP 2.9.3.3]

$$\text{Check}_8 := \text{if}\left[A_{f.neg} \geq A_{fmin.neg}, \text{"Min. Reinforcement is OK"}, \text{"Min. Reinforcement is No Good"}\right]$$

Check₈ = "Min. Reinforcement is OK"

$$\text{DC_Ratio}_8 := \frac{A_{fmin.neg}}{A_{f.neg}} = 0.37$$

Strain Compatibility at Service Stress:

The following is an elastic, straight-line theory approach found in many concrete design text books.

$$M_{s.neg} = 5.7 \cdot \text{kip} \cdot \text{ft}$$

Service Moment

$$A_{f.neg} = 1.06 \text{ in}^2$$

Area of GFRP per UW

$$k_{neg} := \sqrt{2\rho_{f.neg} \cdot n + (\rho_{f.neg} \cdot n)^2} - \rho_{f.neg} \cdot n = 0.18$$

Ratio of Depth of Neutral Axis to Reinforcement
Depth [GFRP 2.7.3-4]

$$j_{neg} := \left(1 - \frac{k_{neg}}{3}\right) = 0.94$$

Ratio of Lever Arm to Depth to Tension
Reinforcement

$$f_{fs.neg} := \frac{M_{s.neg}}{A_{f.neg} \cdot j_{neg} \cdot d_{T.neg}} = 10.38 \cdot \text{ksi}$$

Stress in GFRP at service loads, in addition to any
stresses induced by field flexing

$$f_{cs.neg} := \frac{2 \cdot M_{s.neg}}{j_{neg} \cdot k_{neg} \cdot UW \cdot d_{T.neg}^2} = 1.5 \cdot \text{ksi}$$

Stress in Concrete at service loads

$$na_{s.neg} := k_{neg} \cdot d_{T.neg} = 1.22 \text{ in}$$

Neutral axis of transformed cracked section

Crack Control Check:

$$w_{max} = 0.02 \text{ in}$$

Limiting Crack Width [GFRP 2.9.3.4]

$$d_{c.neg} := t_{slab} - d_{T.neg} = 1.88 \text{ in}$$

Distance to CG GFRP Reinforcement from
tension face of member

$$\beta_{neg} := \frac{t_{slab} - na_{s.neg}}{t_{slab} - na_{s.neg} - d_{c.neg}} = 1.35$$

Crack Parameter (Ratio of dist between NA
and T face to dist between NA and T reinf.
[GFRP 2.9.3.4])

$$w_{neg} := 2 \cdot \frac{f_{fs.neg}}{E_f} \cdot \beta_{neg} \cdot k_b \cdot \sqrt{d_{c.neg}^2 + \frac{S_{T.neg}^2}{4}} = 0.014 \text{ in}$$

Crack Width at Service Limit State [GFRP
2.9.3.4-1]

$$\text{Check}_9 := \text{if}\left(w_{neg} < w_{max}, \text{"Crack Control is OK"}, \text{"Crack Control is No Good"}\right)$$

[GFRP 2.9.3.4]

Check₉ = "Crack Control is OK"

$$\text{DC_Ratio}_9 := \frac{w_{neg}}{w_{max}} = 0.71$$

Stress Limit for Concrete Check:

$$f_{cs.neg} = 1.5 \cdot \text{ksi}$$

Max Concrete Stress at Service Limit

$$\text{Check}_{10} := \text{if}(f_{cs.neg} < 0.45 \cdot f_c, \text{"Concrete Service Stress is OK"}, \text{"Concrete Service Stress is No Good"}) \quad [\text{GFRP 2.9.3.6}]$$

Check₁₀ = "Concrete Service Stress is OK"

$$\text{DC_Ratio}_{10} := \frac{f_{cs.neg}}{0.45 \cdot f_c} = 0.84$$

Creep Rupture Check:

$$I_{cr.neg} := \frac{UW \cdot d_{T.neg}^3}{3} \cdot k_{neg}^3 + n \cdot A_{f.neg} \cdot d_{T.neg}^2 (1 - k_{neg})^2 = 55.49 \text{ in}^4 \quad \text{Cracked Moment of Inertia} \quad [\text{GFRP 2.7.3-3}]$$

$$f_{creep.neg} := \frac{n \cdot d_{T.neg} \cdot (1 - k_{neg})}{I_{cr.neg}} \cdot M_{s.neg} + R_{f.top} = 13.15 \cdot \text{ksi}$$

Sustained force in rebar at Service I limit state
[GFRP 2.7.3-2]

$$f_{creep.limit} = 14 \cdot \text{ksi}$$

Allowable sustained stress in rebar from dead loads, creep rupture limit [GFRP 2.7.3-1]

$$\text{Check}_{11} := \text{if}(f_{creep.neg} < f_{creep.limit}, \text{"Creep Rupture Stress is OK"}, \text{"Creep Rupture Stress is No Good"}) \quad [\text{GFRP 2.9.3.6}]$$

Check₁₁ = "Creep Rupture Stress is OK"

$$\text{DC_Ratio}_{11} := \frac{f_{creep.neg}}{f_{creep.limit}} = 0.94$$

GFRP Distribution Reinforcement:

$$A_{f.bot} := A_{long.bot} \cdot \left(\frac{UW}{S_{long.bot}} \right) \quad A_{f.bot} = 0.736 \cdot \text{in}^2 \quad \text{GFRP Reinforcement per Unit Width, Longitudinal, bottom layer}$$

$$S_{\text{eff}} := S_{\text{gird}} - \frac{b_f}{2} - \frac{t_w}{2} = 6.65 \cdot \text{ft} \quad \text{Effective Span Length [AASHTO 9.7.2.3]}$$

The distance between flange tips, plus the flange overhang, taken as the distance from the extreme flange tip to the face of the web.

$$A_{min.bot} := \min \left(220\% \div \sqrt{\frac{S}{\text{ft}}}, 0.67 \right) \cdot A_{trans.bot} \cdot \frac{UW}{S_{trans.bot}} = 0.71 \cdot \text{in}^2 \quad \text{Minimum required longitudinal reinforcement [GFRP 2.11.4.2]}$$

Check₁₂ := if (A_{f.bot} > A_{min.bot}, "Secondary Bottom Reinforcement OK", "Secondary Bottom Reinforcement No Good")

Check₁₂ = "Secondary Bottom Reinforcement OK"

$$DC_Ratio_{12} := \frac{A_{min.bot}}{A_{f.bot}} = 0.96$$

Temperature And Shrinkage GFRP Reinforcement:

$$\rho_{fst} := \min \left(\max \left(0.0014, 0.0018 \cdot \frac{60\text{ksi}}{f_{fd}} \cdot \frac{E_s}{E_f} \right), 0.0036 \right) = 0.0036 \quad \text{Temperature and Shrinkage GFRP Reinforcement Minimum [GFRP 2.11.5]}$$

$$A_{fst.min} := t_{slab} \cdot \rho_{fst} = 0.37 \cdot \frac{\text{in}^2}{\text{ft}} \quad \text{Minimum GFRP Reinforcement in any face, any direction}$$

$$A_{f.min} := \min \left(\frac{A_{long.top}}{S_{long.top}}, \frac{A_{long.bot}}{S_{long.bot}}, \frac{A_{trans.top}}{S_{trans.top}}, \frac{A_{trans.bot}}{S_{trans.bot}} \right) = 0.41 \cdot \frac{\text{in}^2}{\text{ft}} \quad \text{Minimum Reinforcement Provided in Deck}$$

Check₁₃ := if (A_{f.min} > A_{fst.min}, "Temperature and Shrinkage GFRP OK", "Temperature and Shrinkage GFRP No Good")

Check₁₃ = "Temperature and Shrinkage GFRP OK"

$$DC_Ratio_{13} := \frac{A_{fst.min}}{A_{f.min}} = 0.90$$

$$S_{ts.max} := \min(12\text{in}, 3 \cdot t_{slab}) \quad S_{ts.max} = 12\text{in} \quad \text{Maximum Spacing of T&S Reinforcement [GFRP 2.11.5]}$$

$$S_{max} := \max(S_{long.top}, S_{long.bot}, S_{trans.top}, S_{trans.bot}) = 9\text{in} \quad \text{Maximum Spacing Provided in Deck}$$

Check₁₄ := if (S_{max} ≤ S_{ts.max}, "Reinforcement Spacing OK for T&S", "Reinforcement Spacing No Good for T&S")

Check₁₄ = "Reinforcement Spacing OK for T&S"

$$DC_Ratio_{14} := \frac{S_{max}}{S_{ts.max}} = 0.75$$

Negative Moment Region: Additional GFRP Reinforcement Requirements

- Refer to AASHTO 6.10.1.7 for Guidelines

Negative_Moment := "Yes"

Is there a negative moment region on this bridge? "Yes" or "No"

$$A_{f,neg,min} := 1.0\% \cdot t_{slab} \cdot UW = 1.02 \text{ in}^2$$

Minimum Long. Reinforcement in Negative Moment Regions [AASHTO 6.10.1.7]

$$A_{f,neg,top,min} := \frac{2}{3} \cdot A_{f,neg,min} = 0.68 \text{ in}^2$$

Required GFRP Reinforcement in the Top Layer, per Unit Width

$$A_{f,long,top} := A_{long,top} \cdot \frac{UW}{S_{long,top}} = 0.41 \text{ in}^2$$

Currently Specified Longitudinal GFRP Reinforcement, Top Layer

$$A_{f,long,bot} := A_{long,bot} \cdot \frac{UW}{S_{long,bot}} = 0.74 \text{ in}^2$$

Currently Specified Longitudinal GFRP Reinforcement, Bottom Layer

$$\text{Addl_Reinf_Need} := \text{if} \left[\left(\min \left(\frac{A_{f,long,top}}{A_{f,neg,top,min}}, \frac{A_{f,long,top} + A_{f,long,bot}}{A_{f,neg,min}} \right) \geq 1 \right) \vee \text{Negative_Moment} = \text{"No"}, \text{"No"}, \text{"Yes"} \right]$$

Addl_Reinf_Need = "Yes"

Check if additional negative moment reinforcement is needed.

Specify Additional Longitudinal GFRP Reinforcement in Negative Moment Regions:

$D_{long,top,addl} := 0.625 \text{ in}$

Reinforcing Bar Diameter

$$A_{long,top,addl} := \pi D_{long,top,addl}^2 \div 4 = 0.31 \cdot \text{in}^2$$

Reinforcing Bar Area

$S_{long,top,addl} := S_{long,top} = 9 \text{ in}$

Bar Spacing on Top of Slab, Longitudinal (staggered with continuous longitudinal GFRP Reinforcement)

$$A_{f,long,top,addl} := A_{long,top,addl} \cdot \frac{UW}{S_{long,top,addl}} = 0.41 \text{ in}^2$$

Additional Longitudinal GFRP Reinforcement Area

$$A_{f,long,top,tot} := A_{f,long,top} + A_{f,long,top,addl} = 0.82 \text{ in}^2$$

Total Longitudinal GFRP Reinforcement, top

$$A_{f,long,tot} := A_{f,long,top,tot} + A_{f,long,bot} = 1.55 \text{ in}^2$$

Total Longitudinal GFRP Reinforcement, both layers

$$M_{neg_Ratio} := \min \left(\frac{A_{f,long,top,tot}}{A_{f,neg,top,min}}, \frac{A_{f,long,tot}}{A_{f,neg,min}} \right) = 1.2$$

Ratio of Provided Longitudinal Reinforcement to that Required for Negative Moment Regions

$$\text{Check}_{15} := \text{if} \left(M_{neg_Ratio} \geq 1.0 \vee \text{Addl_Reinf_Need} = \text{"No"}, \text{"Neg Moment Reinf OK"}, \text{"Neg Moment Reinf No Good"} \right)$$

Check₁₅ = "Neg Moment Reinf OK"

$$\text{DC_Ratio}_{15} := \max \left(\frac{A_{f,neg,top,min}}{A_{f,long,top,tot}}, \frac{A_{f,neg,min}}{A_{f,long,tot}} \right) = 0.83$$

Extent of Negative Moment Region Longitudinal Reinforcement is determined separately from this worksheet

Summary of Deck Checks:

Positive Moment Design Checks

Check ₁ = "Deck is OK for Positive Moment Strength"	DC_Ratio ₁ = 0.56
Check ₂ = "Min. Reinforcement is OK"	DC_Ratio ₂ = 0.37
Check ₃ = "Crack Control is OK"	DC_Ratio ₃ = 0.76
Check ₄ = "Slab Deflection is OK"	DC_Ratio ₄ = 0.78
Check ₅ = "Concrete Service Stress is OK"	DC_Ratio ₅ = 0.89
Check ₆ = "Creep Rupture Stress is OK"	DC_Ratio ₆ = 0.99

Negative Moment Design Checks

Check ₇ = "Deck is OK for Negative Moment Strength"	DC_Ratio ₇ = 0.52
Check ₈ = "Min. Reinforcement is OK"	DC_Ratio ₈ = 0.37
Check ₉ = "Crack Control is OK"	DC_Ratio ₉ = 0.71
Check ₁₀ = "Concrete Service Stress is OK"	DC_Ratio ₁₀ = 0.84
Check ₁₁ = "Creep Rupture Stress is OK"	DC_Ratio ₁₁ = 0.94

Detailing Checks

Check ₁₂ = "Secondary Bottom Reinforcement OK"	DC_Ratio ₁₂ = 0.96
Check ₁₃ = "Temperature and Shrinkage GFRP OK"	DC_Ratio ₁₃ = 0.9
Check ₁₄ = "Reinforcement Spacing OK for T&S"	DC_Ratio ₁₄ = 0.75
Check ₁₅ = "Neg Moment Reinf OK"	DC_Ratio ₁₅ = 0.83

Development for Reinforcement:

GFRP 2.12.4 allows the use of the actual stress in the reinforcement at the ultimate limit state (f_f) instead of the design tensile strength of the bar (f_{fd}) for calculating development length. However, for the longitudinal reinforcement in typical decks no ultimate stress is directly calculated from imposed loads. The following example provides an approximation of stress in the GFRP reinforcement and based on the following assumptions:

- The beams supporting the deck are steel I-girders;
- Strain in the GFRP is determined from curvature within the beam at a pier location (negative moment region)
- Curvature within the beam ignores stiffness contributions from the deck and reinforcement
- Stress/Strain within the girder does not discount locked in stresses from self weight of the girder and deck

Approximate Stress in Longitudinal GFRP Bars

$$F_y := 50\text{ksi}$$

Yield Stress of Steel Girders

$$\epsilon_y := \frac{F_y}{E_s} = 0.00172$$

Yield Strain of Steel Girder

$$t_{f_top} := 1.125\text{in}$$

Girder Top Flange Thickness

$$t_{f_bot} := 1.125\text{in}$$

Girder Bottom Flange Thickness

$$D_w := 78\text{in}$$

Girder Web Depth

$$H_{gird} := t_{f_top} + t_{f_bot} + D_w = 80.25\text{in}$$

Overall Girder Depth

$$\text{haunch} := 3.5\text{in}$$

Assumed Maximum Haunch Between Top Flange and Deck

$$y_{NA} := \frac{t_{f_bot} \cdot b_f \cdot \frac{t_{f_bot}}{2} + D_w \cdot t_w \cdot \left(t_{f_bot} + \frac{D_w}{2} \right) + t_{f_top} \cdot b_f \cdot \left(t_{f_bot} + D_w + \frac{t_{f_top}}{2} \right)}{t_{f_bot} \cdot b_f + D_w \cdot t_w + t_{f_top} \cdot b_f} = 40.12\text{in} \quad \text{Neutral Axis of Girder Section}$$

$$y_{tf} := t_{f_bot} + D_w - y_{NA} = 39\text{in}$$

Distance from Neutral Axis to Bottom of Top Flange

$$\rho := \frac{y_{tf}}{\epsilon_y} = 1885 \cdot \text{ft}$$

Curvature in Section Corresponding to Moment when Top Flange Becomes Fully Plastic

$$y_{gfrp} := H_{gird} + \text{haunch} + t_{slab} - \text{Cover}_{top} - y_{NA} = 50.63\text{in}$$

Distance from Neutral Axis to Top Fiber of GFRP Reinforcement

$$\epsilon_{gfrp} := \frac{y_{gfrp}}{\rho} = 0.00224$$

Strain in GFRP Reinforcement

$$f_{f_long} := \epsilon_{gfrp} \cdot E_f = 13.76 \cdot \text{ksi}$$

Stress in GFRP Reinforcement

Longitudinal Top Bars

$$\alpha_{top} := 1.0$$

Bar location modification factor [GFRP 2.12.2.1]

$$d_{b.top} := D_{long.top} = 0.625 \text{ in}$$

$$Spacing_{top} := \left(\frac{1}{\frac{1}{S_{long.top}} + \frac{1}{S_{long.top.addl}}} \right) = 4.5 \text{ in}$$

$$C_{top} := \min \left(Cover_{top} + \frac{D_{long.top}}{2} + D_{trans.top}, \frac{Spacing_{top}}{2} \right) = 2.25 \text{ in} \quad \text{Spacing coefficient [GFRP 2.12.2.1]}$$

$$l_{d.top} := \max \left(\frac{31.6\alpha_{top} \cdot \frac{f_{f, long} \div \text{ksi}}{\sqrt{f_c \div \text{ksi}}} - 340}{13.6 + \min \left(3.5, \frac{C_{top}}{d_{b.top}} \right)} \cdot d_{b.top}, 20 \cdot d_{b.top} \right) = 12.5 \text{ in} \quad \text{Development Length of Bars in Tension [GFRP 2.12.4]}$$

$$l_{s.top} := \text{Ceil}(\max(1.3 \cdot l_{d.top}, 12 \text{ in}), 1 \text{ in}) = 17 \text{ in} \quad \text{Splice Length of Bars in Tension [GFRP 2.12.4]}$$

Longitudinal Bottom Bars

$$\alpha_{bot} := 1.0$$

Bar location modification factor [GFRP 2.12.2.1]

$$d_{b.bot} := D_{long.bot} = 0.625 \text{ in}$$

$$Spacing_{bot} := S_{long.bot} = 5 \text{ in}$$

$$C_{bot} := \min \left(Cover_{bot} + \frac{D_{long.bot}}{2} + D_{trans.bot}, \frac{Spacing_{bot}}{2} \right) = 2.5 \text{ in} \quad \text{Spacing coefficient [GFRP 2.12.2.1]}$$

$$l_{d.bot} := \max \left(\frac{31.6\alpha_{bot} \cdot \frac{f_{f, long} \div \text{ksi}}{\sqrt{f_c \div \text{ksi}}} - 340}{13.6 + \min \left(3.5, \frac{C_{bot}}{d_{b.bot}} \right)} \cdot d_{b.bot}, 20 \cdot d_{b.bot} \right) = 12.5 \text{ in} \quad \text{Development Length of Bars in Tension [GFRP 2.12.4]}$$

$$l_{s.bot} := \text{Ceil}(\max(1.3 \cdot l_{d.bot}, 12 \text{ in}), 1 \text{ in}) = 17 \text{ in} \quad \text{Splice Length of Bars in Tension [GFRP 2.12.4]}$$

PROJECT: Example Project
JOB NO: Project #

DESIGN: X.Xxx
CHECK: Y. Yyy

DATE:
DATE:

GFRP Interior Deck Design

References:

1. AASHTO LRFD Bridge Design Specifications, 7th Edition, 2014, with interims through 2016 [Referenced as "AASHTO"]
2. AASHTO LRFD Bridge Design Guide Specifications for GFRP-Reinforced Concrete Bridge Decks and Traffic Railings [Referenced as "GFRP"]
3. VTrans Structures Design Manual, 2010, with revisions through 2014 [Referenced as "SDM"]
4. VTrans Standard Specifications for Construction, 2011
5. VTrans GFRP Reinforced Deck Design Guidelines, 2016 [Referenced as "Guidelines"]

Assumptions/Notes:

1. Deck is idealized as a 4 span continuous beam, spanning across the 5 beam lines.
2. GRFP properties taken as least values from VTrans GFRP Reinforced Deck Design Guidelines, up to #6 bar.
3. This worksheet is an example design originally developed by T.Y. Lin International, November 2016.

Inputs are in Yellow

Results are in Green

Design Sheet Updates:5/1/2017

Geometry:

$$t_{\text{slab}} := 8.5\text{in}$$

Thickness of Deck Slab, Structural Component

$$t_{\text{dw}} := 3\text{in}$$

Thickness of the Wearing Surface

$$\text{skew} := 15\text{deg}$$

Maximum Skew Angle of Reinforcement to Girders

$$S_{\text{gird.normal}} := 7.25\text{ft}$$

Girder Spacing, Maximum, Normal to Girders

$$S_{\text{gird}} := \frac{S_{\text{gird.normal}}}{\cos(\text{skew})} = 7.51 \cdot \text{ft}$$

Girder Spacing Along Reinforcement

$$t_w := 0.625\text{in}$$

Web Thickness of Girder

$$b_f := 20\text{in}$$

Width of Girder Top Flange

$$UW := 1\text{ft}$$

Unit Width for Analysis

$$\text{Span}_{1_3} := 30\text{ft} \quad \text{Span}_2 := 160\text{ft}$$

Span Lengths, Bearing to Bearing

Reinforcement Geometry:

$$\text{Cover}_{\text{top}} := 1.5\text{in}$$

Top Clear Cover of Concrete Deck (Use same as bottom clear cover since reinforcement is non-corrosive) [Guidelines 2.5.1]

$$\text{Cover}_{\text{bot}} := 1.5\text{in}$$

Bottom Clear Cover of Concrete Deck (SDM Table 5.1.2.6-1)

If the skew angle of the deck does not exceed 25 degrees, the primary reinforcement may be placed in the direction of the skew; otherwise, it shall be placed perpendicular to the main supporting components. Skewed Decks [AASHTO 9.7.1.3]

Main (Transverse) GFRP Reinforcement:

$$D_{\text{trans.top}} := 0.75\text{in}$$

Reinforcing Bar Diameter

$$A_{\text{trans.top}} := \pi D_{\text{trans.top}}^2 \div 4 = 0.44 \cdot \text{in}^2$$

Reinforcing Bar Area

$$S_{\text{trans.top}} := 5.0\text{in}$$

Bar Spacing of Top Transverse Reinforcement (perpendicular to the bar)(Bars placed along skew for bridge skews less than 25 degrees.)

$$D_{\text{trans.bot}} := 0.75\text{in}$$

Reinforcing Bar Diameter

$$A_{\text{trans.bot}} := \pi D_{\text{trans.bot}}^2 \div 4 = 0.44 \cdot \text{in}^2$$

Reinforcing Bar Area

$$S_{\text{trans.bot}} := 5.0\text{in}$$

Bar Spacing of Bottom Transverse Reinforcement (perpendicular to the bar)(Bars placed along skew for bridge skews less than 25 degrees.)

Secondary (Longitudinal) GFRP Reinforcement:

$$D_{\text{long.top}} := 0.625\text{in}$$

Reinforcing Bar Diameter

$$A_{\text{long.top}} := \pi D_{\text{long.top}}^2 \div 4 = 0.31 \cdot \text{in}^2$$

Reinforcing Bar Area

$$S_{\text{long.top}} := 9\text{in}$$

Bar Spacing of Top Longitudinal Reinforcement (perpendicular to the bar)

$$D_{\text{long.bot}} := 0.625\text{in}$$

Reinforcing Bar Diameter

$$A_{\text{long.bot}} := \pi D_{\text{long.bot}}^2 \div 4 = 0.31 \cdot \text{in}^2$$

Reinforcing Bar Area

$$S_{\text{long.bot}} := 5\text{in}$$

Bar Spacing of Bottom Longitudinal Reinforcement (perpendicular to the bar)

$$d' := \text{Cover}_{\text{top}} + D_{\text{trans.top}} \div 2$$

$$d' = 1.87 \cdot \text{in}$$

Distance to Top Layer of Main Reinforcement

$$d := t_{\text{slab}} - \text{Cover}_{\text{bot}} - (D_{\text{trans.bot}} \div 2)$$

$$d = 6.62 \cdot \text{in}$$

Distance to Bottom Layer of Main Reinforcement

Material Properties:

Designer Note: GFRP Reinforced Deck Design Guidelines Section 2.3.2: The designer should investigate current values for Nominal Ultimate Tensile Strength, Modulus of Elasticity, and Nominal Tensile Strain at the beginning of each new design regardless of the suggested design values provided. The nominal strength and strain used in design should be the minimum value reported by at least three manufacturers.

$$f_{fu} := 100\text{ksi}$$

Nominal Ultimate Tensile Strength of GFRP Bars for Product Certification [Guidelines 2.3.2]

$$E_f := 6150\text{ksi}$$

Modulus of Elasticity of GFRP [Guidelines 2.3.3]

$$\epsilon_{fu} := 0.01226$$

Nominal Tensile Strain of GFRP Bars for Product Certification [Guidelines 2.3.2]

$$f'_c := 4\text{ksi}$$

Concrete Strength [SDM Table 5.1.1.1-1]

$$C_E := 0.7$$

Environmental Reduction Factor, Concrete Exposed to Earth and Weather [Guidelines 2.3.2 and GFRP T2.6.1.2-1]

$$f_{fd} := C_E \cdot f_{fu} = 70 \cdot \text{ksi}$$

Design Tensile Strength of GFRP Bar [GFRP 2.6.1.2-1]

$$\epsilon_{fd} := C_E \cdot \epsilon_{fu} = 0.0086$$

Design Tensile Strain of GFRP Bars [GFRP 2.6.1.2-1]

$$k_b := 1.0$$

Bond Dependent Coefficient [Guidelines 2.4.2.3 and GFRP 2.9.3.4]

$$E_c := 2500 \left(\frac{f'_c}{\text{ksi}} \right)^{0.33} \text{ ksi} = 3950 \cdot \text{ksi}$$

Modulus of Elasticity of Concrete [AASHTO C5.4.2.4-1]

$$f_r := 0.24 \cdot \sqrt{f'_c \cdot \text{ksi}} = 0.48 \cdot \text{ksi}$$

Modulus of Rupture of Concrete [AASHTO 5.4.2.6]

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

Ultimate Strain in Concrete

$$\gamma_c := \begin{cases} 150\text{pcf} & \text{if } (f'_c \leq 5\text{ksi}) \\ \left(145 + \frac{f'_c}{\text{ksi}} \right) \cdot \text{pcf} & \text{otherwise} \end{cases}$$

Unit weight of Concrete and Wearing Surface [SDM Table 3.3.1-1]

$$\gamma_c = 150 \cdot \text{pcf}$$

$$\gamma_{bit} := 150\text{pcf}$$

$$E_s := 29000\text{ksi}$$

Modulus of Elasticity of Steel Reinforcement

$$\nu := 0.2$$

Poisson's Ratio for Concrete

$$n := \frac{E_f}{E_c} = 1.56$$

Modular ratio

$$\beta_1 := \begin{cases} 0.85 & \text{if } f_c \leq 4\text{ksi} \\ \max\left[0.85 - 0.05 \cdot \left(\frac{f_c - 4\text{ksi}}{\text{ksi}}\right), 0.65\right] & \text{otherwise} \end{cases} \quad \beta_1 = 0.85$$

Stress Block to N.A. Parameter [GFRP 2.9.3.1]

[AASHTO 5.7.2.2]

$$\rho_{fb} := 0.85 \cdot \beta_1 \cdot \frac{f_c}{f_{fd}} \cdot \left(\frac{E_f \cdot \epsilon_{cu}}{E_f \cdot \epsilon_{cu} + f_{fd}} \right) = 0.0086$$

*Balanced Reinforcement Ratio
[GFRP Eq 2.7.4.2-2]*

Effect of Imposed Deformation:

Curvature at the crown of a bridge will impose stress into the GFRP bar, reducing capacity.

$$D_{\text{trans.top}} = 0.75 \text{ in}$$

$$D_{\text{trans.bot}} = 0.75 \text{ in}$$

Diameter of the transverse reinforcement

$$\theta_{\text{crown}} := 4\%$$

Algebraic difference of slopes on either side of the crown break

$$R_{\text{crown}} := \frac{4 \text{ in}}{3 \cdot \theta_{\text{crown}}^2} = 69.44 \cdot \text{ft}$$

*Approximate radius of the rebar bend at the crown
[Guidelines 2.4.1-3]*

Top Transverse Reinforcement

$$R_{e,\text{top}} := \frac{D_{\text{trans.top}}}{2 \cdot R_{\text{crown}}} = 0.00045$$

*Strain reduction due to field flexing at the crown
[Guidelines 2.4.1-2]*

$$R_{f,\text{top}} := \frac{D_{\text{trans.top}}}{2 \cdot R_{\text{crown}}} \cdot E_f = 2.77 \cdot \text{ksi}$$

*Stress reduction due to field flexing at the crown
[Guidelines 2.4.1-1]*

$$f_{\text{fd}_r,\text{top}} := C_E \cdot f_{\text{fu}} - R_{f,\text{top}} = 67.23 \cdot \text{ksi}$$

*Reduced Design Tensile Strength of GFRP Bars ,
accounting for field flexing [Guidelines 2.3.2]*

$$\epsilon_{\text{fd}_r,\text{top}} := C_E \cdot \epsilon_{\text{fu}} - R_{e,\text{top}} = 0.00813$$

*Reduced Design Strain of GFRP Bars, accounting
for field flexing [Guidelines 2.3.2]*

Bottom Transverse Reinforcement

$$R_{e,\text{bot}} := \frac{D_{\text{trans.bot}}}{2 \cdot R_{\text{crown}}} = 0.00045$$

*Strain reduction due to field flexing at the crown
[Guidelines 2.4.1-2]*

$$R_{f,\text{bot}} := \frac{D_{\text{trans.bot}}}{2 \cdot R_{\text{crown}}} \cdot E_f = 2.77 \cdot \text{ksi}$$

*Stress reduction due to field flexing at the crown
[Guidelines 2.4.1-1]*

$$f_{\text{fd}_r,\text{bot}} := C_E \cdot f_{\text{fu}} - R_{f,\text{bot}} = 67.23 \cdot \text{ksi}$$

*Reduced Design Tensile Strength of GFRP Bars ,
accounting for field flexing [Guidelines 2.3.2]*

$$\epsilon_{\text{fd}_r,\text{bot}} := C_E \cdot \epsilon_{\text{fu}} - R_{e,\text{bot}} = 0.00813$$

*Reduced Design Strain of GFRP Bars, accounting
for field flexing [Guidelines 2.3.2]*

Loads & Moments:

Dead Loads:

$$DW := (t_{dw}) \cdot \gamma_{bit} \cdot UW$$

$$DW = 37.5 \cdot \text{plf} \quad \text{Uniform Load of Wearing Surface}$$

$$DC := \gamma_c \cdot (t_{slab}) \cdot UW$$

$$DC = 106.25 \cdot \text{plf} \quad \text{Uniform Load due to Deck Slab Self Weight}$$

Base dead load moments on continuous 4 span girder model with uniformly distributed load (ignore presence of overhangs - slightly conservative)

[AISC Manual Table 3-22c]

$$M_{DW.neg} := 0.107DW \cdot S_{gird}^2$$

$$M_{DW.neg} = 0.23 \cdot \text{kip} \cdot \text{ft}$$

Max Negative Moment due to DW

$$M_{DW.pos} := 0.0772DW \cdot S_{gird}^2$$

$$M_{DW.pos} = 0.16 \cdot \text{kip} \cdot \text{ft}$$

Max Positive Moment due to DW

$$M_{DC.neg} := 0.107DC \cdot S_{gird}^2$$

$$M_{DC.neg} = 0.64 \cdot \text{kip} \cdot \text{ft}$$

Max Negative Moment due to DC

$$M_{DC.pos} := 0.0772DC \cdot S_{gird}^2$$

$$M_{DC.pos} = 0.46 \cdot \text{kip} \cdot \text{ft}$$

Max Positive Moment due to DC

Live Loads:

- Utilize AASHTO 4.6.2.1.6 and Table A4-1 (Table data is in the following Area):

▶ AASHTO Table A4-1 Data

Functions to lookup and interpolate the Moments from AASHTO Table A4-1

$$M_{pos.function}(S_{gird.in}) := \text{linterp}(S_{lookup}, M_{pos.table}, S_{gird.in})$$

Simple linear interpolation of the AASHTO Table A4-1 positive moment vector

$$M_{neg.function}(S_{gird.in}, b_{neg.in}) := \begin{cases} \text{for } col \in 0 .. \text{cols}(M_{neg.table}) - 1 \\ M_{neg.S.gird.col} \leftarrow \text{linterp}(S_{lookup}, M_{neg.table}^{(col)}, S_{gird.in}) \\ \text{return } \text{linterp}(b_{neg.lookup}, M_{neg.S.gird}, b_{neg.in}) \end{cases}$$

Nested linear interpolation to lookup and interpolate in both directions in the AASHTO Table A4-1 negative moment matrix. First each column is evaluated by itself based on the input S_{gird} . Then, the output vector from that is interpolated based on the input B_{neg} .

$$b_{neg} := 0.25 \cdot \frac{b_f}{\cos(\text{skew})} = 5.18 \text{ in}$$

Location of Design Section for Negative Moment in Deck [AASHTO 4.6.2.1.6]

- Interpolate in Table A4-1 for Moments

$$M_{LL.pos} := M_{pos.function}(S_{gird}) = 5.44 \cdot \text{kip} \cdot \text{ft}$$

Max Positive LL moment with Impact and Multiple Presence Factors Included

$$M_{LL.neg} := M_{neg.function}(S_{gird}, b_{neg}) = 4.84 \cdot \text{kip} \cdot \text{ft}$$

Max Negative LL moment with Impact and Multiple Presence Factors Included

Strength I Design Moment:

$$\gamma_{DC} := 1.25 \quad \gamma_{DW} := 1.50 \quad \gamma_{LL} := 1.75$$

*Load Factors for Dead Load (DC), Dead Loads (DW), and Live Load (LL)
[AASHTO Table 3.4.1-1, Table 3.4.1-2]*

$$M_{u,pos} := \gamma_{DC} \cdot M_{DC,pos} + \gamma_{DW} \cdot M_{DW,pos} + \gamma_{LL} \cdot M_{LL,pos} = 10.35 \cdot \text{kip} \cdot \text{ft} \quad \textit{Ultimate Positive Moment, Strength I}$$

$$M_{u,neg} := \gamma_{DC} \cdot M_{DC,neg} + \gamma_{DW} \cdot M_{DW,neg} + \gamma_{LL} \cdot M_{LL,neg} = 9.61 \cdot \text{kip} \cdot \text{ft} \quad \textit{Ultimate Negative Moment, Strength I}$$

Service I Design Moment:

$$\gamma_{DC,s} := 1.0 \quad \gamma_{DW,s} := 1.0 \quad \gamma_{LL,s} := 1.0$$

*Load Factors for Dead Load (DC), Dead Loads (DW), and Live Load (LL)
[AASHTO Table 3.4.1-1, Table 3.4.1-2]*

$$M_{s,pos} := \gamma_{DC,s} \cdot M_{DC,pos} + \gamma_{DW,s} \cdot M_{DW,pos} + \gamma_{LL,s} \cdot M_{LL,pos} = 6.07 \cdot \text{kip} \cdot \text{ft} \quad \textit{Service I Positive Moment}$$

$$M_{s,neg} := \gamma_{DC,s} \cdot M_{DC,neg} + \gamma_{DW,s} \cdot M_{DW,neg} + \gamma_{LL,s} \cdot M_{LL,neg} = 5.7 \cdot \text{kip} \cdot \text{ft} \quad \textit{Service I Negative Moment}$$

$$M_{s,pos,DL} := \gamma_{DC,s} \cdot M_{DC,pos} + \gamma_{DW,s} \cdot M_{DW,pos} = 0.63 \cdot \text{kip} \cdot \text{ft} \quad \textit{Service I Positive Dead Load Moment}$$

$$M_{s,neg,DL} := \gamma_{DC,s} \cdot M_{DC,neg} + \gamma_{DW,s} \cdot M_{DW,neg} = 0.87 \cdot \text{kip} \cdot \text{ft} \quad \textit{Service I Negative Dead Load Moment}$$

Check Main Reinforcement (Positive Moment):

$$A_{f,pos} := A_{trans.bot} \cdot \frac{UW}{S_{trans.bot}} = 1.06 \cdot \text{in}^2 \quad \text{Tensile Reinforcement per unit width}$$

$$d_{T,pos} := d = 6.62 \text{ in} \quad S_{T,pos} := S_{trans.bot} = 5 \cdot \text{in} \quad \text{Depth to Tension GFRP Reinforcement from Top of Section and Spacing of Tension GFRP Reinforcement}$$

$$M_{u,pos} = 10.35 \cdot \text{kip} \cdot \text{ft} \quad \text{Ultimate Moment}$$

$$M_{s,pos} = 6.07 \cdot \text{kip} \cdot \text{ft} \quad \text{Service Moment}$$

$$\rho_{f,pos} := \frac{A_{f,pos}}{UW \cdot d_{T,pos}} = 0.01334 \quad \rho_{fb} = 0.00861 \quad \text{Reinforcement Ratio vs. Balanced Ratio}$$

$$\phi_{pos} := \begin{cases} 0.55 & \text{if } \rho_{f,pos} \leq \rho_{fb} \\ 0.3 + 0.25 \cdot \frac{\rho_{f,pos}}{\rho_{fb}} & \text{if } \rho_{fb} < \rho_{f,pos} \leq 1.4 \cdot \rho_{fb} \\ 0.65 & \text{otherwise} \end{cases} = 0.65 \quad \text{Resistance Factor for Concrete in Flexure [GFRP 2.7.4.2]}$$

If $\rho_{f,pos} < \rho_{fb}$ is true the failure is initiated by rupture of GFRP

When failure is initiated by crushing of the concrete, take the minimum between the equation and the design tensile strength as the effective strength of reinforcement

$$f_{f,pos} := \begin{cases} f_{fd,r.bot} & \text{if } \rho_{f,pos} < \rho_{fb} \\ \min \left[\sqrt{\frac{(E_f \cdot \epsilon_{cu})^2}{4} + \frac{0.85 \cdot \beta_1 \cdot f'_c}{\rho_{f,pos}} \cdot E_f \cdot \epsilon_{cu}} - 0.5 E_f \cdot \epsilon_{cu}, f_{fd,r.bot} \right] & \text{otherwise} \end{cases} = 54.67 \cdot \text{ksi} \quad \text{Stress in GFRP Bars [GFRP 2.9.3.1]}$$

$$M_{n,pos} := a \leftarrow \frac{A_{f,pos} \cdot f_{f,pos}}{0.85 \cdot f'_c \cdot UW} = 28.57 \cdot \text{kip} \cdot \text{ft} \quad \text{Nominal Flexural Resistance [GFRP 2.9.3.2.2]}$$

$$c_b \leftarrow \left(\frac{\epsilon_{cu}}{\epsilon_{cu} + \epsilon_{fd,r.bot}} \cdot d_{T,pos} \right) \quad \text{[GFRP EQ. 2.9.3.2.2-2]}$$

$$\begin{cases} \left[A_{f,pos} \cdot f_{f,pos} \cdot \left(d_{T,pos} - \frac{a}{2} \right) \right] & \text{if } \rho_{f,pos} > \rho_{fb} \\ \left[A_{f,pos} \cdot f_{fd,r.bot} \cdot \left(d_{T,pos} - \frac{\beta_1 \cdot c_b}{2} \right) \right] & \text{otherwise} \end{cases} \quad \begin{matrix} \text{[GFRP EQ. 2.9.3.2.2-4]} \\ \text{If true the failure is initiated by concrete crushing [GFRP EQ. 2.9.3.2.2-1]} \\ \text{If true the failure is initiated by rupture of GFRP [GFRP EQ. 2.9.3.2.2-3]} \end{matrix}$$

$$M_{r,pos} := \phi_{pos} \cdot M_{n,pos} = 18.57 \cdot \text{kip} \cdot \text{ft} \quad \text{Factored Flexural Resistance [GFRP 2.9.3.2.1-1]}$$

Check₁ := if (M_{r,pos} ≥ M_{u,pos}, "Deck is OK for Positive Moment Strength", "Deck is No Good for Positive Moment Strength")

Check₁ = "Deck is OK for Positive Moment Strength"

$$DC_Ratio_1 := \frac{M_{u,pos}}{M_{r,pos}} = 0.56$$

Failure_{pos} := if (ρ_{f,pos} > ρ_{fb}, "Concrete Crushing", "GFRP Rupture") = "Concrete Crushing"

Minimum Reinforcement:

$$A_{fmin,pos} := \max\left(0.16 \cdot \sqrt{\frac{f'_c}{\text{ksi}}}, 0.33\right) \cdot \frac{UW \cdot d_{T,pos}}{\left(\frac{f_{fd,r,bot}}{\text{ksi}}\right)} = 0.39 \text{ in}^2$$

*Minimum Flexural Tension Reinforcement
[GFRP 2.9.3.3]*

$$\text{Check}_2 := \text{if}\left[A_{f,pos} \geq A_{fmin,pos}, \text{"Min. Reinforcement is OK"}, \text{"Min. Reinforcement is No Good"}\right]$$

Check₂ = "Min. Reinforcement is OK"

$$\text{DC_Ratio}_2 := \frac{A_{fmin,pos}}{A_{f,pos}} = 0.37$$

Strain Compatibility at Service Stress:

The following is an elastic, straight-line theory approach found in many concrete design text books.

$$M_{s,pos} = 6.07 \cdot \text{kip} \cdot \text{ft}$$

Service Moment

$$A_{f,pos} = 1.06 \text{ in}^2$$

Area of GFRP per UW

$$k_{pos} := \sqrt{2\rho_{f,pos} \cdot n + (\rho_{f,pos} \cdot n)^2} - \rho_{f,pos} \cdot n = 0.18$$

*Ratio of Depth of Neutral Axis to Reinforcement
Depth [GFRP 2.7.3-4]*

$$j_{pos} := \left(1 - \frac{k_{pos}}{3}\right) = 0.94$$

*Ratio of Lever Arm to Depth to Tension
Reinforcement*

$$f_{fs,pos} := \frac{M_{s,pos}}{A_{f,pos} \cdot j_{pos} \cdot d_{T,pos}} = 11.04 \cdot \text{ksi}$$

*Stress in GFRP at service loads, in addition to any
stresses induced by field flexing*

$$f_{cs,pos} := \frac{2 \cdot M_{s,pos}}{j_{pos} \cdot k_{pos} \cdot UW \cdot d_{T,pos}^2} = 1.6 \cdot \text{ksi}$$

Stress in Concrete at service loads

$$na_{s,pos} := k_{pos} \cdot d_{T,pos} = 1.22 \text{ in}$$

Neutral axis of transformed cracked section

Crack Control Check:

$$w_{max} := 0.02 \text{ in}$$

Limiting Crack Width [GFRP 2.9.3.4]

$$d_{c,pos} := t_{slab} - d_{T,pos} = 1.88 \text{ in}$$

*Distance to CG GFRP Reinforcement from
tension face of member*

$$\beta_{pos} := \frac{t_{slab} - na_{s,pos}}{t_{slab} - na_{s,pos} - d_{c,pos}} = 1.35$$

*Crack Parameter (Ratio of dist between NA
and T face to dist between NA and T reinf.
[GFRP 2.9.3.4])*

$$w_{pos} := 2 \cdot \frac{f_{fs,pos}}{E_f} \cdot \beta_{pos} \cdot k_b \cdot \sqrt{d_{c,pos}^2 + \frac{S_{T,pos}^2}{4}} = 0.015 \text{ in}$$

*Crack Width at Service Limit State [GFRP
2.9.3.4-1]*

$$\text{Check}_3 := \text{if}\left(w_{pos} < w_{max}, \text{"Crack Control is OK"}, \text{"Crack Control is No Good"}\right)$$

[GFRP 2.9.3.4]

Check₃ = "Crack Control is OK"

$$\text{DC_Ratio}_3 := \frac{w_{pos}}{w_{max}} = 0.76$$

Deflection Limit Check:

$$\Delta_{\max} := \frac{S_{\text{gird}}}{800} = 0.113 \text{ in}$$

Deflection Limit [GFRP 2.9.3.5, AASHTO 9.5.2]

$$I_g := \frac{UW \cdot t_{\text{slab}}^3}{12} = 614.13 \text{ in}^4$$

Gross Moment of Inertia

$$I_{\text{cr,pos}} := \frac{UW \cdot d_{\text{T,pos}}^3}{3} \cdot k_{\text{pos}}^3 + n \cdot A_{\text{f,pos}} \cdot d_{\text{T,pos}}^2 (1 - k_{\text{pos}})^2 = 55.49 \text{ in}^4$$

Cracked Moment of Inertia (GFRP 2.7.3-3)

$$M_{\text{cr}} := f_r \cdot \frac{I_g}{t_{\text{slab}} \div 2} = 5.78 \cdot \text{kip} \cdot \text{ft}$$

Cracking Moment (GFRP 2.9.3.5-2)

$$P_{\text{LL}} := 16 \text{ kip} \cdot 1.33 = 21.28 \cdot \text{kip}$$

Point live load for calculating deflection - AASHTO design truck, no lane, with impact [AASHTO 3.6.1.3.2]. Note multipl Presence is not included to better correlate with Standard Specifications.

$$w_{\text{tire}} := 20 \text{ in}$$

Width of HL-93 Truck Tire

$$p_o := \frac{P_{\text{LL}}}{w_{\text{tire}}} = 1.06 \cdot \frac{\text{kip}}{\text{in}}$$

Wheel Load, Distributed over Tire Width

$$M_a := M_{\text{s,pos}} = 6.07 \cdot \text{kip} \cdot \text{ft}$$

Maximum Moment Concurrent with Deflection being Investigated [GFRP 2.9.3.5]

$$\beta_d := \min\left(\frac{1}{5} \cdot \frac{\rho_{\text{f,pos}}}{\rho_{\text{fb}}}, 1\right) = 0.31$$

Reduction Factor used for Effective Moment of Inertia Determination [GFRP 2.9.3.5-2]

$$I_{\text{e,pos}} := \text{if} \left[M_{\text{cr}} \geq M_a, I_g, \min \left[I_g, \left[\left(\frac{M_{\text{cr}}}{M_a} \right)^3 \cdot \beta_d \cdot I_g + \left[1 - \left(\frac{M_{\text{cr}}}{M_a} \right)^3 \right] \cdot I_{\text{cr,pos}} \right] \right] \right] = 171.93 \cdot \text{in}^4$$

Effective Moment of Inertia in Cracked Section (GFRP 2.9.3.5-1)

$$t_{\text{e,pos}} := \left(I_{\text{e,pos}} \cdot \frac{12}{UW} \right)^{\left(\frac{1}{3} \right)} = 5.56 \text{ in}$$

Equivalent Slab Thickness Corresponding to Effective Moment of Inertia

$$D := \frac{E_c \cdot t_{\text{e,pos}}^3}{12 \cdot (1 - \nu^2)} = 58955 \cdot \text{kip} \cdot \text{in}$$

Plate Flexural Rigidity Term

Deflection Limit Check (continued):

The following equation for live load deflection is based on a single, simply supported bay and ignores continuity from adjacent bays. Assuming an "end span" and continuity with at least two more bays, assume an effective $0.9 \cdot S$ span length for live load deflection calculations.

$$S_{\text{defl}} := 0.90 \cdot S_{\text{gird}} = 81.06 \text{ in}$$

Equivalent Girder Spacing for Deflection Checks

$$\Delta_{s,LL} := \frac{P_o \cdot S_{\text{defl}}^3}{\pi^4 \cdot D} \cdot \sum_{m=1}^{100} \left[\frac{1}{m^4} \cdot \left(\sin \left(\frac{m \cdot \pi \cdot w_{\text{tire}}}{2 S_{\text{defl}}} \right) \right) \cdot \left(\sin \left(\frac{m \cdot \pi}{2} \right) \right)^2 \right] = 0.0386 \text{ in}$$

LL Deflection at Service Limit (Ref: Single Span, One-Way Plate using Levy Approximate Solution and Transverse Load Distribution Across Tire Width)

$$\Delta_{s,DL} := \frac{0.0069(DC + DW) \cdot S_{\text{gird}}^4}{E_c \cdot I_{e,\text{pos}}} = 0.008 \text{ in}$$

DL Deflection at Service Limit (Ref: 4 span uniformly loaded beam table)

$$\Delta_{LT} := \text{if}(I_{e,\text{pos}} = I_{g,4,2.2}) \cdot (\Delta_{s,DL}) = 0.018 \text{ in}$$

Long Term deflection, accounting for creep from sustained loads (2.2 value is taken from ACI 440.1r-06)

$$\text{Check}_4 := \text{if}(\Delta_{LT} + \Delta_{s,LL} < \Delta_{\text{max}}, \text{"Slab Deflection is OK"}, \text{"Slab Deflection is No Good"})$$

[GFRP 2.9.3.5]

Check₄ = "Slab Deflection is OK"

$$\text{DC_Ratio}_4 := \max \left(\frac{\Delta_{LT} + \Delta_{s,LL}}{\Delta_{\text{max}}} \right) = 0.50$$

Stress Limit for Concrete Check:

$$f_{cs,pos} = 1.6 \cdot \text{ksi}$$

Max Concrete Stress at Service Limit

$$\text{Check}_5 := \text{if}(f_{cs,pos} < 0.45 \cdot f_c, \text{"Concrete Service Stress is OK"}, \text{"Concrete Service Stress is No Good"}) \quad [\text{GFRP 2.9.3.6}]$$

Check₅ = "Concrete Service Stress is OK"

$$\text{DC_Ratio}_5 := \frac{f_{cs,pos}}{0.45 \cdot f_c} = 0.89$$

Creep Rupture Check:

$$I_{cr,pos} = 55.49 \text{ in}^4$$

Cracked Moment of Inertia [GFRP 2.7.3-3]

$$f_{creep,pos} := \frac{n \cdot d_{T,pos} \cdot (1 - k_{pos})}{I_{cr,pos}} \cdot M_{s,pos} + R_{f,bot} = 13.81 \cdot \text{ksi}$$

Sustained force in rebar at Service I limit state [GFRP 2.7.3-2]

$$f_{creep,limit} := 0.2 \cdot f_{fd} = 14 \cdot \text{ksi}$$

Allowable sustained stress in rebar from dead loads, creep rupture limit [GFRP 2.7.3-1]

$$\text{Check}_6 := \text{if}(f_{creep,pos} < f_{creep,limit}, \text{"Creep Rupture Stress is OK"}, \text{"Creep Rupture Stress is No Good"}) \quad [\text{GFRP 2.9.3.6}]$$

Check₆ = "Creep Rupture Stress is OK"

$$\text{DC_Ratio}_6 := \frac{f_{creep,pos}}{f_{creep,limit}} = 0.99$$

Check Main Reinforcement (Negative Moment):

$$A_{f,neg} := A_{trans,top} \cdot \frac{UW}{S_{trans,top}} = 1.06 \text{ in}^2 \quad \text{Tensile Reinforcement per unit width}$$

$$d_{T,neg} := t_{slab} - d' = 6.62 \text{ in} \quad S_{T,neg} := S_{trans,top} = 5 \text{ in} \quad \text{Depth to Tension GFRP Reinforcement from Bottom of Section and Spacing of Tension GFRP Reinforcement}$$

$$M_{u,neg} = 9.61 \cdot \text{kip} \cdot \text{ft} \quad \text{Ultimate Moment}$$

$$M_{s,neg} = 5.7 \cdot \text{kip} \cdot \text{ft} \quad \text{Service Moment}$$

$$\rho_{f,neg} := \frac{A_{f,neg}}{UW \cdot d_{T,neg}} = 0.01334 \quad \rho_{fb} = 0.00861 \quad \text{Reinforcement Ratio vs. Balanced Ratio}$$

$$\phi_{neg} := \begin{cases} 0.55 & \text{if } \rho_{f,neg} \leq \rho_{fb} \\ 0.3 + 0.25 \cdot \frac{\rho_{f,neg}}{\rho_{fb}} & \text{if } \rho_{fb} < \rho_{f,neg} \leq 1.4 \cdot \rho_{fb} \\ 0.65 & \text{otherwise} \end{cases} = 0.65 \quad \text{Resistance Factor for Concrete in Flexure [GFRP 2.7.4.2]}$$

$$f_{f,neg} := \begin{cases} f_{fd,r,top} & \text{if } \rho_{f,neg} < \rho_{fb} \\ \min \left[\sqrt{\frac{(E_f \cdot \epsilon_{cu})^2}{4} + \frac{0.85 \cdot \beta_1 \cdot f'_c}{\rho_{f,neg}} \cdot E_f \cdot \epsilon_{cu}} - 0.5 E_f \cdot \epsilon_{cu}, f_{fd,r,top} \right] & \text{otherwise} \end{cases} = 54.67 \cdot \text{ksi} \quad \text{Stress in GFRP Bars [GFRP 2.9.3.1]}$$

$$M_{n,neg} := \begin{cases} a \leftarrow \frac{A_{f,neg} \cdot f_{f,neg}}{0.85 \cdot f'_c \cdot UW} = 28.57 \cdot \text{kip} \cdot \text{ft} & \text{Nominal Flexural Resistance [GFRP 2.9.3.2.2]} \\ c_b \leftarrow \left(\frac{\epsilon_{cu}}{\epsilon_{cu} + \epsilon_{fd,r,top}} \cdot d_{T,neg} \right) & \text{[GFRP EQ. 2.9.3.2.2-2]} \\ \left[A_{f,neg} \cdot f_{f,neg} \cdot \left(d_{T,neg} - \frac{a}{2} \right) \right] & \text{if } \rho_{f,neg} > \rho_{fb} \quad \text{If true the failure is initiated by concrete crushing [GFRP EQ. 2.9.3.2.2-1]} \\ \left[A_{f,neg} \cdot f_{fd,r,top} \cdot \left(d_{T,neg} - \frac{\beta_1 \cdot c_b}{2} \right) \right] & \text{otherwise} \quad \text{If true the failure is initiated by rupture of GFRP [GFRP EQ. 2.9.3.2.2-3]} \end{cases}$$

$$M_{r,neg} := \phi_{neg} \cdot M_{n,neg} = 18.57 \cdot \text{kip} \cdot \text{ft} \quad \text{Factored Flexural Resistance [GFRP 2.9.3.2.1-1]}$$

$$\text{Check}_7 := \text{if} (M_{r,neg} \geq M_{u,neg}, \text{"Deck is OK for Negative Moment Strength"}, \text{"Deck is No Good for Negative Moment Stre})$$

$$\text{Check}_7 = \text{"Deck is OK for Negative Moment Strength"} \quad DC_Ratio_7 := \frac{M_{u,neg}}{M_{r,neg}} = 0.52$$

$$\text{Failure}_{neg} := \text{if} (\rho_{f,neg} > \rho_{fb}, \text{"Concrete Crushing"}, \text{"GFRP Rupture"}) = \text{"Concrete Crushing"}$$

Minimum Reinforcement:

$$A_{fmin.neg} := \max\left(0.16 \cdot \sqrt{\frac{f'_c}{\text{ksi}}}, 0.33\right) \cdot \frac{UW \cdot d_{T.neg}}{\left(\frac{f_{fd.r.top}}{\text{ksi}}\right)} = 0.39 \text{ in}^2$$

Minimum Flexural Tension Reinforcement
[GFRP 2.9.3.3]

$$\text{Check}_8 := \text{if}\left[A_{f.neg} \geq A_{fmin.neg}, \text{"Min. Reinforcement is OK"}, \text{"Min. Reinforcement is No Good"}\right]$$

Check₈ = "Min. Reinforcement is OK"

$$\text{DC_Ratio}_8 := \frac{A_{fmin.neg}}{A_{f.neg}} = 0.37$$

Strain Compatibility at Service Stress:

The following is an elastic, straight-line theory approach found in many concrete design text books.

$$M_{s.neg} = 5.7 \cdot \text{kip} \cdot \text{ft}$$

Service Moment

$$A_{f.neg} = 1.06 \text{ in}^2$$

Area of GFRP per UW

$$k_{neg} := \sqrt{2\rho_{f.neg} \cdot n + (\rho_{f.neg} \cdot n)^2} - \rho_{f.neg} \cdot n = 0.18$$

Ratio of Depth of Neutral Axis to Reinforcement
Depth [GFRP 2.7.3-4]

$$j_{neg} := \left(1 - \frac{k_{neg}}{3}\right) = 0.94$$

Ratio of Lever Arm to Depth to Tension
Reinforcement

$$f_{fs.neg} := \frac{M_{s.neg}}{A_{f.neg} \cdot j_{neg} \cdot d_{T.neg}} = 10.38 \cdot \text{ksi}$$

Stress in GFRP at service loads, in addition to any
stresses induced by field flexing

$$f_{cs.neg} := \frac{2 \cdot M_{s.neg}}{j_{neg} \cdot k_{neg} \cdot UW \cdot d_{T.neg}^2} = 1.5 \cdot \text{ksi}$$

Stress in Concrete at service loads

$$na_{s.neg} := k_{neg} \cdot d_{T.neg} = 1.22 \text{ in}$$

Neutral axis of transformed cracked section

Crack Control Check:

$$w_{max} = 0.02 \text{ in}$$

Limiting Crack Width [GFRP 2.9.3.4]

$$d_{c.neg} := t_{slab} - d_{T.neg} = 1.88 \text{ in}$$

Distance to CG GFRP Reinforcement from
tension face of member

$$\beta_{neg} := \frac{t_{slab} - na_{s.neg}}{t_{slab} - na_{s.neg} - d_{c.neg}} = 1.35$$

Crack Parameter (Ratio of dist between NA
and T face to dist between NA and T reinf.
[GFRP 2.9.3.4])

$$w_{neg} := 2 \cdot \frac{f_{fs.neg}}{E_f} \cdot \beta_{neg} \cdot k_b \cdot \sqrt{d_{c.neg}^2 + \frac{S_{T.neg}^2}{4}} = 0.014 \text{ in}$$

Crack Width at Service Limit State [GFRP
2.9.3.4-1]

$$\text{Check}_9 := \text{if}\left(w_{neg} < w_{max}, \text{"Crack Control is OK"}, \text{"Crack Control is No Good"}\right)$$

[GFRP 2.9.3.4]

Check₉ = "Crack Control is OK"

$$\text{DC_Ratio}_9 := \frac{w_{neg}}{w_{max}} = 0.71$$

Stress Limit for Concrete Check:

$$f_{cs.neg} = 1.5 \cdot \text{ksi}$$

Max Concrete Stress at Service Limit

$$\text{Check}_{10} := \text{if}(f_{cs.neg} < 0.45 \cdot f_c, \text{"Concrete Service Stress is OK"}, \text{"Concrete Service Stress is No Good"}) \quad [\text{GFRP 2.9.3.6}]$$

Check₁₀ = "Concrete Service Stress is OK"

$$\text{DC_Ratio}_{10} := \frac{f_{cs.neg}}{0.45 \cdot f_c} = 0.84$$

Creep Rupture Check:

$$I_{cr.neg} := \frac{UW \cdot d_{T.neg}^3}{3} \cdot k_{neg}^3 + n \cdot A_{f.neg} \cdot d_{T.neg}^2 (1 - k_{neg})^2 = 55.49 \text{ in}^4 \quad \text{Cracked Moment of Inertia [GFRP 2.7.3-3]}$$

$$f_{creep.neg} := \frac{n \cdot d_{T.neg} \cdot (1 - k_{neg})}{I_{cr.neg}} \cdot M_{s.neg} + R_{f.top} = 13.15 \cdot \text{ksi}$$

*Sustained force in rebar at Service I limit state
[GFRP 2.7.3-2]*

$$f_{creep.limit} = 14 \cdot \text{ksi}$$

Allowable sustained stress in rebar from dead loads, creep rupture limit [GFRP 2.7.3-1]

$$\text{Check}_{11} := \text{if}(f_{creep.neg} < f_{creep.limit}, \text{"Creep Rupture Stress is OK"}, \text{"Creep Rupture Stress is No Good"}) \quad [\text{GFRP 2.9.3.6}]$$

Check₁₁ = "Creep Rupture Stress is OK"

$$\text{DC_Ratio}_{11} := \frac{f_{creep.neg}}{f_{creep.limit}} = 0.94$$

GFRP Distribution Reinforcement:

$$A_{f.bot} := A_{long.bot} \cdot \left(\frac{UW}{S_{long.bot}} \right) \quad A_{f.bot} = 0.736 \cdot \text{in}^2 \quad \text{GFRP Reinforcement per Unit Width, Longitudinal, bottom layer}$$

$$S_{\text{eff}} := S_{\text{gird}} - \frac{b_f}{2} - \frac{t_w}{2} = 6.65 \cdot \text{ft} \quad \text{Effective Span Length [AASHTO 9.7.2.3]}$$

The distance between flange tips, plus the flange overhang, taken as the distance from the extreme flange tip to the face of the web.

$$A_{min.bot} := \min \left(220\% \div \sqrt{\frac{S}{\text{ft}}}, 0.67 \right) \cdot A_{trans.bot} \cdot \frac{UW}{S_{trans.bot}} = 0.71 \cdot \text{in}^2 \quad \text{Minimum required longitudinal reinforcement [GFRP 2.11.4.2]}$$

Check₁₂ := if (A_{f.bot} > A_{min.bot}, "Secondary Bottom Reinforcement OK", "Secondary Bottom Reinforcement No Good")

Check₁₂ = "Secondary Bottom Reinforcement OK"

$$DC_Ratio_{12} := \frac{A_{min.bot}}{A_{f.bot}} = 0.96$$

Temperature And Shrinkage GFRP Reinforcement:

$$\rho_{fst} := \min \left(\max \left(0.0014, 0.0018 \cdot \frac{60\text{ksi}}{f_{fd}} \cdot \frac{E_s}{E_f} \right), 0.0036 \right) = 0.0036 \quad \text{Temperature and Shrinkage GFRP Reinforcement Minimum [GFRP 2.11.5]}$$

$$A_{fst.min} := t_{slab} \cdot \rho_{fst} = 0.37 \cdot \frac{\text{in}^2}{\text{ft}} \quad \text{Minimum GFRP Reinforcement in any face, any direction}$$

$$A_{f.min} := \min \left(\frac{A_{long.top}}{S_{long.top}}, \frac{A_{long.bot}}{S_{long.bot}}, \frac{A_{trans.top}}{S_{trans.top}}, \frac{A_{trans.bot}}{S_{trans.bot}} \right) = 0.41 \cdot \frac{\text{in}^2}{\text{ft}} \quad \text{Minimum Reinforcement Provided in Deck}$$

Check₁₃ := if (A_{f.min} > A_{fst.min}, "Temperature and Shrinkage GFRP OK", "Temperature and Shrinkage GFRP No Good")

Check₁₃ = "Temperature and Shrinkage GFRP OK"

$$DC_Ratio_{13} := \frac{A_{fst.min}}{A_{f.min}} = 0.90$$

$$S_{ts.max} := \min(12\text{in}, 3 \cdot t_{slab}) \quad S_{ts.max} = 12\text{in} \quad \text{Maximum Spacing of T&S Reinforcement [GFRP 2.11.5]}$$

$$S_{max} := \max(S_{long.top}, S_{long.bot}, S_{trans.top}, S_{trans.bot}) = 9\text{in} \quad \text{Maximum Spacing Provided in Deck}$$

Check₁₄ := if (S_{max} ≤ S_{ts.max}, "Reinforcement Spacing OK for T&S", "Reinforcement Spacing No Good for T&S")

Check₁₄ = "Reinforcement Spacing OK for T&S"

$$DC_Ratio_{14} := \frac{S_{max}}{S_{ts.max}} = 0.75$$

Negative Moment Region: Additional GFRP Reinforcement Requirements

- Refer to AASHTO 6.10.1.7 for Guidelines

Negative_Moment := "Yes"

Is there a negative moment region on this bridge? "Yes" or "No"

$$A_{f,neg,min} := 1.0\% \cdot t_{slab} \cdot UW = 1.02 \text{ in}^2$$

Minimum Long. Reinforcement in Negative Moment Regions [AASHTO 6.10.1.7]

$$A_{f,neg,top,min} := \frac{2}{3} \cdot A_{f,neg,min} = 0.68 \text{ in}^2$$

Required GFRP Reinforcement in the Top Layer, per Unit Width

$$A_{f,long,top} := A_{long,top} \cdot \frac{UW}{S_{long,top}} = 0.41 \text{ in}^2$$

Currently Specified Longitudinal GFRP Reinforcement, Top Layer

$$A_{f,long,bot} := A_{long,bot} \cdot \frac{UW}{S_{long,bot}} = 0.74 \text{ in}^2$$

Currently Specified Longitudinal GFRP Reinforcement, Bottom Layer

$$\text{Addl_Reinf_Need} := \text{if} \left[\left(\min \left(\frac{A_{f,long,top}}{A_{f,neg,top,min}}, \frac{A_{f,long,top} + A_{f,long,bot}}{A_{f,neg,min}} \right) \geq 1 \right) \vee \text{Negative_Moment} = \text{"No"}, \text{"No"}, \text{"Yes"} \right]$$

Addl_Reinf_Need = "Yes"

Check if additional negative moment reinforcement is needed.

Specify Additional Longitudinal GFRP Reinforcement in Negative Moment Regions:

$D_{long,top,addl} := 0.625 \text{ in}$

Reinforcing Bar Diameter

$$A_{long,top,addl} := \pi D_{long,top,addl}^2 \div 4 = 0.31 \cdot \text{in}^2$$

Reinforcing Bar Area

$S_{long,top,addl} := S_{long,top} = 9 \text{ in}$

Bar Spacing on Top of Slab, Longitudinal (staggered with continuous longitudinal GFRP Reinforcement)

$$A_{f,long,top,addl} := A_{long,top,addl} \cdot \frac{UW}{S_{long,top,addl}} = 0.41 \text{ in}^2$$

Additional Longitudinal GFRP Reinforcement Area

$$A_{f,long,top,tot} := A_{f,long,top} + A_{f,long,top,addl} = 0.82 \text{ in}^2$$

Total Longitudinal GFRP Reinforcement, top

$$A_{f,long,tot} := A_{f,long,top,tot} + A_{f,long,bot} = 1.55 \text{ in}^2$$

Total Longitudinal GFRP Reinforcement, both layers

$$M_{neg_Ratio} := \min \left(\frac{A_{f,long,top,tot}}{A_{f,neg,top,min}}, \frac{A_{f,long,tot}}{A_{f,neg,min}} \right) = 1.2$$

Ratio of Provided Longitudinal Reinforcement to that Required for Negative Moment Regions

$$\text{Check}_{15} := \text{if} \left(M_{neg_Ratio} \geq 1.0 \vee \text{Addl_Reinf_Need} = \text{"No"}, \text{"Neg Moment Reinf OK"}, \text{"Neg Moment Reinf No Good"} \right)$$

Check₁₅ = "Neg Moment Reinf OK"

$$\text{DC_Ratio}_{15} := \max \left(\frac{A_{f,neg,top,min}}{A_{f,long,top,tot}}, \frac{A_{f,neg,min}}{A_{f,long,tot}} \right) = 0.83$$

Extent of Negative Moment Region Longitudinal Reinforcement is determined separately from this worksheet

Summary of Deck Checks:

Positive Moment Design Checks

Check ₁ = "Deck is OK for Positive Moment Strength"	DC_Ratio ₁ = 0.56
Check ₂ = "Min. Reinforcement is OK"	DC_Ratio ₂ = 0.37
Check ₃ = "Crack Control is OK"	DC_Ratio ₃ = 0.76
Check ₄ = "Slab Deflection is OK"	DC_Ratio ₄ = 0.5
Check ₅ = "Concrete Service Stress is OK"	DC_Ratio ₅ = 0.89
Check ₆ = "Creep Rupture Stress is OK"	DC_Ratio ₆ = 0.99

Negative Moment Design Checks

Check ₇ = "Deck is OK for Negative Moment Strength"	DC_Ratio ₇ = 0.52
Check ₈ = "Min. Reinforcement is OK"	DC_Ratio ₈ = 0.37
Check ₉ = "Crack Control is OK"	DC_Ratio ₉ = 0.71
Check ₁₀ = "Concrete Service Stress is OK"	DC_Ratio ₁₀ = 0.84
Check ₁₁ = "Creep Rupture Stress is OK"	DC_Ratio ₁₁ = 0.94

Detailing Checks

Check ₁₂ = "Secondary Bottom Reinforcement OK"	DC_Ratio ₁₂ = 0.96
Check ₁₃ = "Temperature and Shrinkage GFRP OK"	DC_Ratio ₁₃ = 0.9
Check ₁₄ = "Reinforcement Spacing OK for T&S"	DC_Ratio ₁₄ = 0.75
Check ₁₅ = "Neg Moment Reinf OK"	DC_Ratio ₁₅ = 0.83

Development for Reinforcement:

GFRP 2.12.4 allows the use of the actual stress in the reinforcement at the ultimate limit state (f_f) instead of the design tensile strength of the bar (f_{fd}) for calculating development length. However, for the longitudinal reinforcement in typical decks no ultimate stress is directly calculated from imposed loads. The following example provides an approximation of stress in the GFRP reinforcement and based on the following assumptions:

- The beams supporting the deck are steel I-girders;
- Strain in the GFRP is determined from curvature within the beam at a pier location (negative moment region)
- Curvature within the beam ignores stiffness contributions from the deck and reinforcement
- Stress/Strain within the girder does not discount locked in stresses from self weight of the girder and deck

Approximate Stress in Longitudinal GFRP Bars

$$F_y := 50\text{ksi}$$

Yield Stress of Steel Girders

$$\epsilon_y := \frac{F_y}{E_s} = 0.00172$$

Yield Strain of Steel Girder

$$t_{f_top} := 1.125\text{in}$$

Girder Top Flange Thickness

$$t_{f_bot} := 1.125\text{in}$$

Girder Bottom Flange Thickness

$$D_w := 78\text{in}$$

Girder Web Depth

$$H_{gird} := t_{f_top} + t_{f_bot} + D_w = 80.25\text{in}$$

Overall Girder Depth

$$\text{haunch} := 3.5\text{in}$$

Assumed Maximum Haunch Between Top Flange and Deck

$$y_{NA} := \frac{t_{f_bot} \cdot b_f \cdot \frac{t_{f_bot}}{2} + D_w \cdot t_w \cdot \left(t_{f_bot} + \frac{D_w}{2} \right) + t_{f_top} \cdot b_f \cdot \left(t_{f_bot} + D_w + \frac{t_{f_top}}{2} \right)}{t_{f_bot} \cdot b_f + D_w \cdot t_w + t_{f_top} \cdot b_f} = 40.12\text{in} \quad \text{Neutral Axis of Girder Section}$$

$$y_{tf} := t_{f_bot} + D_w - y_{NA} = 39\text{in}$$

Distance from Neutral Axis to Bottom of Top Flange

$$\rho := \frac{y_{tf}}{\epsilon_y} = 1885\text{ft}$$

Curvature in Section Corresponding to Moment when Top Flange Becomes Fully Plastic

$$y_{gfrp} := H_{gird} + \text{haunch} + t_{slab} - \text{Cover}_{top} - y_{NA} = 50.63\text{in}$$

Distance from Neutral Axis to Top Fiber of GFRP Reinforcement

$$\epsilon_{gfrp} := \frac{y_{gfrp}}{\rho} = 0.00224$$

Strain in GFRP Reinforcement

$$f_{f_long} := \epsilon_{gfrp} \cdot E_f = 13.76\text{ksi}$$

Stress in GFRP Reinforcement

Longitudinal Top Bars

$$\alpha_{\text{top}} := 1.0$$

Bar location modification factor [GFRP 2.12.2.1]

$$d_{\text{b.top}} := D_{\text{long.top}} = 0.625 \text{ in}$$

$$\text{Spacing}_{\text{top}} := \left(\frac{1}{\frac{1}{S_{\text{long.top}}} + \frac{1}{S_{\text{long.top.addl}}}} \right) = 4.5 \text{ in}$$

$$C_{\text{top}} := \min \left(\text{Cover}_{\text{top}} + \frac{D_{\text{long.top}}}{2} + D_{\text{trans.top}}, \frac{\text{Spacing}_{\text{top}}}{2} \right) = 2.25 \text{ in} \quad \text{Spacing coefficient [GFRP 2.12.2.1]}$$

$$l_{\text{d.top}} := \max \left(\frac{31.6\alpha_{\text{top}} \cdot \frac{f_{\text{f.long}} \div \text{ksi}}{\sqrt{f_{\text{c}} \div \text{ksi}}} - 340}{13.6 + \min \left(3.5, \frac{C_{\text{top}}}{d_{\text{b.top}}} \right)} \cdot d_{\text{b.top}}, 20 \cdot d_{\text{b.top}} \right) = 12.5 \text{ in} \quad \text{Development Length of Bars in Tension [GFRP 2.12.4]}$$

$$l_{\text{s.top}} := \text{Ceil}(\max(1.3 \cdot l_{\text{d.top}}, 12\text{in}), 1\text{in}) = 17 \text{ in} \quad \text{Splice Length of Bars in Tension [GFRP 2.12.4]}$$

Longitudinal Bottom Bars

$$\alpha_{\text{bot}} := 1.0$$

Bar location modification factor [GFRP 2.12.2.1]

$$d_{\text{b.bot}} := D_{\text{long.bot}} = 0.625 \text{ in}$$

$$\text{Spacing}_{\text{bot}} := S_{\text{long.bot}} = 5 \text{ in}$$

$$C_{\text{bot}} := \min \left(\text{Cover}_{\text{bot}} + \frac{D_{\text{long.bot}}}{2} + D_{\text{trans.bot}}, \frac{\text{Spacing}_{\text{bot}}}{2} \right) = 2.5 \text{ in} \quad \text{Spacing coefficient [GFRP 2.12.2.1]}$$

$$l_{\text{d.bot}} := \max \left(\frac{31.6\alpha_{\text{bot}} \cdot \frac{f_{\text{f.long}} \div \text{ksi}}{\sqrt{f_{\text{c}} \div \text{ksi}}} - 340}{13.6 + \min \left(3.5, \frac{C_{\text{bot}}}{d_{\text{b.bot}}} \right)} \cdot d_{\text{b.bot}}, 20 \cdot d_{\text{b.bot}} \right) = 12.5 \text{ in} \quad \text{Development Length of Bars in Tension [GFRP 2.12.4]}$$

$$l_{\text{s.bot}} := \text{Ceil}(\max(1.3 \cdot l_{\text{d.bot}}, 12\text{in}), 1\text{in}) = 17 \text{ in} \quad \text{Splice Length of Bars in Tension [GFRP 2.12.4]}$$