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SECTION 1

INTRODUCTION

1.1—SCOPE

These specifications offer a description of the properties of glass fiber-reinforced polymer (GFRP) reinforcing bars as well as provisions for the design and construction of structural concrete bridge members reinforced with GFRP bars.

GFRP reinforcement shall be in the form of deformed or sand-coated bars, or both, to provide bond with concrete.

These specifications are not intended to supplant proper training or the exercise of judgment by the Design Professional, and state only the minimum requirements necessary to provide for public safety. The Owner or the Design Professional may require the sophistication of the design or the quality of materials and construction to be higher than the minimum requirements.

The Design Professional shall be familiar with the provisions of the *AASHTO LRFD Bridge Design Specifications* and latest interim specifications, as well as with the design of conventional reinforced concrete structures.

The commentary directs attention to other documents that provide suggestions for carrying out the requirements and intent of these specifications. However, those

C1.1

GFRP materials have emerged as an alternative noncorrosive material for reinforcing bars for concrete structures (Iyer and Sen, 1991; Neale and Labossiere, 1992; White, 1992; Nanni, 1993; Nanni and Dolan, 1993; Dolan et al., 1996; El-Badry, 1996; Humar and Razaqpur, 2000; Burgoyne, 2001; Cosenza et al., 2001; Figueiras et al., 2001; Teng, 2001; Triantafillou, 2007; Bank, 2007; El-Sayed and Benmokrane, 2008; Nanni et al., 2014). GFRP reinforcing bars offer advantages over steel reinforcement due to their noncorrosive nature, which makes them attractive for bridge structures such as beams (Nanni 1993; 2003; Thériault and Benmokrane, 1998; Ashour, 2006; Bentz et al., 2010; Matta et al., 2013), columns (De Luca et al., 2010; Mohamed et al., 2014; Hadhood et al., 2017), decks (Bradberry, 2001; Nanni and Faza, 2002; Benmokrane et al., 2004; 2006; 2007a; 2007b), and traffic railings (Buth et al., 2003; El-Salakawy et al., 2003; Matta and Nanni, 2009; Ahmed et al., 2013; Sennah and Mostafa, 2018). Due to other differences in the physical and mechanical behavior of GFRP materials as opposed to steel, unique guidance on the design and construction of concrete bridges reinforced with GFRP bars is needed.

The term “shall” denotes a requirement for compliance with these specifications. The term “should” indicates a strong preference for a given criterion. The term “may” indicates a criterion that is usable, but other local and suitably documented, verified, and approved criteria may also be used in a manner consistent with the LRFD approach to bridge design.

Together with straight and hooked GFRP bars used as longitudinal reinforcement, GFRP stirrups in the form of C-shaped bars, spirals, etc., may be implemented to ensure that shear resistance meets safety requirements.

documents and this commentary are not intended to be a part of these specifications.

Material specifications and construction specifications consistent with these design specifications are provided in ASTM D7957/D7957M and Section 6, respectively.

1.2—DEFINITIONS

Design Professional—The architect, engineer, architectural firm, or engineering firm responsible for the design of the bridge and issuing Contract Documents or administering the Work under Contract Documents, or both.

Fiber—Any fine thread-like natural or synthetic object of mineral or organic origin. Note: this term is generally used for materials whose length is at least 100 times its diameter.

Fiber, aramid—Highly oriented organic fiber derived from polyamide incorporating into an aromatic ring structure.

Fiber, carbon—Fiber produced by heating organic precursor materials containing a substantial amount of carbon, such as rayon, polyacrylonitrile (PAN), or pitch in an inert environment.

Fiber, glass—Fiber drawn from an inorganic product of fusion that has cooled without crystallizing.

Grating—A two-dimensional (planar) or three-dimensional (spatial) rigid array of interconnected GFRP pultruded shapes other than deformed bars.

Grid—A two-dimensional (planar) or three-dimensional (spatial) rigid array of interconnected GFRP bars that form a contiguous lattice that can be used to reinforce concrete.

Lightweight concrete—Concrete containing lightweight aggregate conforming to AASHTO M 195 and having an equilibrium density not exceeding 0.135 kcf, as determined in conformance with ASTM C567/C567M.

Reinforced concrete—Structural concrete containing no less than the minimum amounts of GFRP reinforcement specified herein.

Structural concrete—All concrete used for structural purposes.

1.3—LIMITATIONS

Prestressed applications; GFRP reinforcement in combination with steel reinforcing bars to resist the same force effect; use of fiber types other than glass, such as carbon, basalt, and aramid fibers; and the use of grid and gratings as well as smooth and hollow-type GFRP reinforcing bars are not covered in these specifications. Assembly of GFRP mats using GFRP bars addressed in these specifications is allowed.

Plain GFRP reinforcing bars used as dowels (that is, devices that transfer shear load across concrete joints), where the intended function requires slip of the dowel, are not covered in these specifications.

The use of lightweight concrete reinforced with GFRP bars is not covered in these specifications and may be specified by the Owner.

The assumed failure mechanism of GFRP-reinforced flexural members shall not be based on the formation of plastic hinges as GFRP materials demonstrate a linear elastic behavior up to failure. Moment redistribution in continuous members shall not be considered for GFRP-reinforced concrete bridge members.

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GFRP reinforcing bars with a smooth external surface are not covered by these specifications, as their use as concrete reinforcement is restricted due to lack of bond development with concrete. Hollow-type GFRP bars are not considered due to unknown performance as reinforcement for concrete.

The design and construction of lightweight concrete members internally reinforced with GFRP bars is not covered because of the lack of research on this subject. The Owner may specify the use of lightweight concrete based on evidence from physical tests approved by the Owner.

The use of GFRP reinforcing bars in members to be designed for seismic loads is not covered in these specifications.

GFRP reinforcing bars have a significantly lower compressive strength than tensile strength. The strength of any GFRP reinforcing bar in compression shall be ignored in design calculations.

The use of GFRP bars as compression reinforcement of flexural members is not recommended. Placing GFRP reinforcing bars in the compression zone of flexural members is permitted provided that they are not taken into account for the determination of the member flexural resistance.

1.4—DESIGN PHILOSOPHY

These specifications are based on limit state design principles where structural components shall be proportioned to satisfy the requirements at all appropriate service, fatigue and creep rupture, strength, and extreme event limit states. In many instances, serviceability or fatigue and creep rupture limits may control the design.

Provisions related to limit states analysis, general design and location features, loads and load factors, and structural analysis and evaluation shall comply with the *AASHTO LRFD Bridge Design Specifications*.

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The limit states specified herein are intended to provide for a buildable, serviceable bridge, capable of safely carrying design loads for a specified lifetime.

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SECTION 2

CONCRETE STRUCTURES

2.1—SCOPE

The provisions in this Section apply to the design of bridge and ancillary structures constructed of normal weight concrete and reinforced with GFRP bars. The provisions are based on normal weight concrete design compressive strengths as specified in Article 5.4.2.1 of the *AASHTO LRFD Bridge Design Specifications*, and with design compressive strength, f'_c , not exceeding 10.0 ksi.

The provisions in this Section characterize regions of concrete structures by their behavior as B- (beam or Bernoulli) Regions or D (disturbed or discontinuity) Regions, as defined in Article 2.2. The characterization of regions into B Regions and D Regions is discussed in Article 5.5.1.2 of the *AASHTO LRFD Bridge Design Specifications*.

2.2—DEFINITIONS

Anchor—Steel element either cast into concrete or post-installed into a hardened concrete member and used to transmit applied loads to the concrete. Cast-in-place anchors include headed bolts, hooked bolts (J- or L-bolt), and headed studs. Post-installed anchors include expansion anchors, undercut anchors, and adhesive anchors. Steel elements for adhesive anchors include threaded rods, deformed reinforcing bars, or internally threaded steel sleeves with external deformations.

Anchorage—For reinforcing bars, a length of reinforcement, or a mechanical anchor or hook, or combination thereof at the end of a bar needed to transfer the force carried by the bar into the concrete.

Beam or Bernoulli Region (B-Region)—Regions of concrete members in which Bernoulli's hypothesis of straight-line strain profiles, linear for bending and uniform for shear, applies (see Article 2.5.1.2 for more detail).

Cast-in-place concrete—Concrete placed in its final location in the structure while still in a plastic state.

Composite construction—Concrete components or concrete and steel components interconnected to respond to force effects as a unit.

Compression-controlled section—A cross-section in which the tensile stress in the extreme tension GFRP at nominal resistance is less than the design tensile strength.

Concrete cover—The specified minimum distance between the surface of the reinforcing bars and the surface of the concrete.

Confinement—A condition where the disintegration of the concrete under compression is prevented by the development of lateral and/or circumferential forces such as may be provided by appropriate reinforcement, steel or composite tubes, or similar devices.

Creep—Time-dependent deformation under permanent load.

Deck slab—A solid concrete slab resisting and distributing wheel loads to the supporting components.

Deep component—Components in which the distance from the point of 0.0 shear to the face of the support is less than $2d$ or components in which a load causing more than one third of the shear at a support is closer than $2d$ from the face of the support.

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These specifications use kips and ksi units. Some other specifications, such as ACI 318 and *AASHTO Standard Specifications for Highway Bridges* (17th Edition), use pound and psi units. For most variables the conversion is obvious, but for those which have the form

$$N\sqrt{f'_c} \text{ the conversion is } \frac{N\sqrt{f'_c}}{\sqrt{1,000}} = 0.0316N\sqrt{f'_c}.$$

Design concrete compressive strength—The nominal compressive strength of concrete specified for the work and assumed for design and analysis of new structures.

Development length—The distance required to develop the design tensile strength of a GFRP reinforcing bar.

Direct loading/supporting—Application of a load or use of a support external to the member, as in the case of point or uniform loads applied directly to the deck surface, simply-supported girder ends, bent (pier) cap supported on pinned columns.

Disturbed or Discontinuity Region (D-Region)—Regions of concrete members encompassing abrupt changes in geometry or concentrated forces in which strain profiles more complex than straight lines exist (see Article 2.5.1.2 for more detail).

Edge distance—The minimum distance between the centerline of reinforcement or other embedded elements and the edge of the concrete.

Effective depth—The depth of a component effective in resisting flexural or shear forces.

Embedment length—The length of GFRP reinforcement provided beyond a critical section over which transfer of force between concrete and GFRP reinforcement may occur.

Indirect loading/supporting—Application of a load or use of a support internally such as girders framing into an integral bent (pier) cap, dapped or spliced girders where load transfer is between the top and bottom face of the member, or utility loads hung from the web of a girder.

Normal weight concrete—Plain concrete having an equilibrium density greater than 0.135 kcf and a density not exceeding 0.155 kcf.

Precast members—Concrete elements cast in a location other than their final position.

Reinforced concrete—Structural concrete containing no less than the minimum amounts of prestressing tendons or non-prestressed reinforcement specified herein.

Reinforcement—Reinforcing bars.

Slab—A component having a width of at least four times its effective depth.

Specified concrete strength—The compressive strength of concrete specified in the contract documents which may be greater than the compressive strength of concrete for use in design, f'_c .

Spiral—Continuously wound bar or wire in the form of a cylindrical helix.

Structural concrete—All concrete used for structural purposes.

Structural mass concrete—Any large volume of concrete where special materials or procedures are required to cope with the generation of heat of hydration and attendant volume change to minimize cracking.

Strut-and-tie method—A procedure used principally in regions of concentrated forces and geometric discontinuities (D-Regions) to determine concrete proportions and reinforcement quantities and patterns based on an analytic model consisting of compression struts in the concrete, tensile ties in the reinforcement, and the geometry of nodes at their points of intersection.

Tension-controlled section—A cross section in which the tensile stress in the extreme tension GFRP at nominal resistance is equal to the design tensile strength.

Transverse reinforcement—Reinforcement used to resist shear, torsion, and lateral forces or to confine concrete in a structural member. The terms “stirrups” and “web reinforcement” are usually applied to transverse reinforcement in flexural members and the terms “ties,” “hoops,” and “spirals” are applied to transverse reinforcement in compression members.

2.3—NOTATION

A_f	=	area of GFRP reinforcement (in. ²) (2.5.3)
A_{fv}	=	area of transverse reinforcement within distance s (in. ²) (2.7.2.4)
A_{cp}	=	area enclosed by outside perimeter of concrete cross section (in. ²) (2.7.2.1)
A_{ct}	=	area of concrete on the flexural tension side of the member taken as the half-depth containing the flexural tension zone unless otherwise specified by the Owner (in. ²) (2.7.3.6.2)
A_g	=	gross area of section (in. ²) (2.6.4.2)
A_t	=	total area of longitudinal torsion reinforcement in a box girder (in. ²) (2.7.3.8.3)
A_o	=	area enclosed by the shear flow path, including any area of holes therein (in. ²) (2.7.2.1)
A_t	=	area of one leg of closed transverse torsion reinforcement in solid members or total area of transverse torsion reinforcement in the exterior web and flange of hollow members (in. ²) (2.7.3.8.2)
a	=	depth of equivalent rectangular stress block (in.) (2.6.3.2.2)
b	=	width of the compression face of the member (in.) (2.5.3)
b_e	=	effective width of the shear flow path taken as the minimum thickness of the exterior webs or flanges comprising the closed box section (in.) (2.7.2.1)
b_o	=	perimeter of critical section computed at $d/2$ away from the concentrated load (in.) (C2.7.3.4). The shape of the critical section shall be the same as the shape of the concentrated load (2.10.5.1.3).
b_v	=	effective web width taken as the minimum web width, measured parallel to the neutral axis, between the resultants of the tensile and compressive forces due to flexure, or for circular sections, the diameter of the section, modified for the presence of ducts where applicable (in.) (2.7.2.4)
C	=	lesser of the cover to the center of the bar or one-half of the center-to-center spacing of the bars being developed (in.) (2.9.7.4.1)
C_b	=	reduction factor that accounts for the degree of bond between GFRP reinforcing bars and surrounding concrete (2.6.7)
C_c	=	creep rupture reduction factor (2.5.3)
C_E	=	environmental reduction factor (2.4.2.1)
C_f	=	fatigue rupture reduction factor (2.5.4)
c	=	distance from extreme compression fiber to neutral axis (in.); cohesion factor (ksi) (2.6.2.2) (2.7.4)
c_b	=	distance from extreme compression fiber to neutral axis at balanced strain condition (in.) (2.6.7)
c_c	=	clear cover (in.) (2.6.7)
D	=	external diameter of the circular member (in.) (C2.7.2.8)
D_r	=	diameter of the circle passing through the centers of the longitudinal reinforcement (in.) (C2.7.2.8)
d	=	distance from extreme compression fiber to centroid of tensile reinforcement (in.) (2.5.1.2.1)
d_b	=	GFRP reinforcing bar diameter (in.) (2.7.2.7.2)
d_c	=	thickness of concrete cover measured from extreme tension fiber to center of flexural GFRP reinforcement located closest thereto (in.) (2.6.7)
d_s	=	distance from extreme compression fiber to the centroid of the tensile reinforcement measured along the centerline of the web (in.) (2.7.3.6.2)
d_v	=	effective shear depth taken as the distance, measured perpendicular to the neutral axis, between the resultants of the tensile and compressive forces due to flexure; it need not be taken to be less than the greater of $0.9d$ or $0.72h$ (in.) (2.7.2.8)
E_c	=	modulus of elasticity of concrete (ksi) (2.4.1)
E_f	=	tensile modulus of elasticity of GFRP reinforcement (ksi) (2.6.7)
f'_c	=	compressive strength of concrete for use in design (ksi) (2.1)
f_f	=	effective strength in GFRP reinforcement at the strength and extreme event limit state (ksi) (2.6.3.1)
$f_{f,c}$	=	calculated tensile stress in GFRP reinforcement at the creep rupture limit state (ksi) (2.5.3)
$f_{f,f}$	=	calculated tensile stress in GFRP reinforcement at the fatigue limit state (ksi) (2.5.4)

f_{fb}	= design tensile strength of bent portion of GFRP bar (ksi) (2.7.3.5)
f_{fd}	= design tensile strength of GFRP reinforcing bars considering reductions for service environment (ksi) (2.4.2.1)
f_{fu}	= tensile strength for product certification as reported by GFRP manufacturers (ksi) (2.4.2.1)
f_{fr}	= required GFRP reinforcing bar stress (ksi) (2.7.3.7)
$f_{f,s}$	= calculated tensile stress in GFRP reinforcement at the service limit state (ksi) (2.5.3) (2.6.7)
f_{fv}	= design tensile strength of transverse reinforcement (ksi) (2.7.2.4)
f_{pc}	= unfactored compressive stress in concrete either at the centroid of the cross section resisting transient loads or at the junction of the web and flange where the centroid lies in the flange (ksi) (2.7.2.1)
f_r	= modulus of rupture of concrete specified in Article 5.4.2.6 of the <i>AASHTO LRFD Bridge Design Specifications</i> (ksi) (2.6.3.3)
h	= depth of cross section (in.) (2.7.2.5)
k	= ratio of depth of neutral axis to depth of flexural reinforcement (2.5.3)
I_{cr}	= moment of inertia of transformed cracked section (in. ⁴) (2.5.3)
I_e	= effective moment of inertia (in. ⁴) (2.6.3.4.2)
I_g	= gross moment of inertia (in. ⁴) (2.6.3.4.2)
L	= span length (ft) (2.10.2)
ℓ_d	= development length (in.) (2.9.7.4.1)
ℓ_{dh}	= development length of hooks in tension, measured from critical section to outside end or edge of the hook (in.) (2.9.7.4.3)
M_a	= maximum moment in a component at the stage for which deformation is computed (kip-in.) (2.6.3.4.2)
M_{cr}	= cracking moment (kip-in.) (2.6.3.4.2)
M_{dnc}	= total unfactored dead load moment acting on the monolithic or noncomposite section (kip-in.) (2.6.3.3)
M_n	= nominal flexural resistance (kip-in.) (2.6.3.2.1)
M_r	= factored flexural resistance (kip-in.) (2.6.3.2.1)
$M_{s,s}$	= moment due to dead loads and sustained portion of live loads included in Service I load combination (kip-in.) (2.5.3)
$M_{s,f}$	= moment due to dead loads included in Service I load combination and factored live loads included in Fatigue I load combination (kip-in.) (2.5.4)
M_u	= factored moment at the section (kip-in.) (2.7.3.6.2)
N_u	= factored axial force, taken as positive if tensile and negative if compressive (kip) (2.7.3.6.2)
n_f	= modular ratio = E_f/E_c (2.5.3)
P_n	= nominal axial resistance, with or without flexure (kip) (2.6.4.2); nominal resistance of a tie (kip) (2.6.6.1)
P_r	= factored axial resistance, with or without flexure (kip) (2.6.4.2)
p_c	= length of outside perimeter of the concrete section (in.) (2.7.2.1)
p_h	= perimeter of the centerline of the closed transverse torsion reinforcement (in.); perimeter of the centerline of the closed transverse torsion reinforcement for solid members, or the perimeter of the centroid of the transverse torsion reinforcement in the exterior webs and flanges for hollow members (in.); (2.7.3.6.2) (2.7.3.8.3)
r_b	= internal radius of bent GFRP reinforcing bar (in.) (2.7.3.5)
S_c	= section modulus for the extreme fiber of the composite section where tensile stress is caused by externally applied loads (in. ³) (2.6.3.3)
S_{nc}	= section modulus for the extreme fiber of the monolithic or noncomposite section where tensile stress is caused by externally applied loads (in. ³) (2.6.3.3)
s	= pitch of spiral or vertical spacing of hoops (in.); average spacing of reinforcement in layer closest to tension face (in.); spacing of transverse reinforcement measured in a direction parallel to the longitudinal reinforcement (in.); spacing of stirrups (in.) (2.7.3.5) (2.6.7) (2.7.2.4)
s_x	= crack spacing parameter (in.) (2.7.3.6.2)
s_{xe}	= crack spacing parameter as influenced by aggregate size (in.) (2.7.3.6.2)

T_{cr}	=	torsional cracking moment (kip-in.) (2.7.2.1)
T_n	=	nominal torsional resistance (kip-in.) (2.7.2.1)
T_r	=	factored torsional resistance (kip-in.) (2.7.2.1)
T_u	=	applied factored torsional moment (kip-in.); applied factored torsional moment on the girder (2.7.2.1) (2.7.3.6.2)
V_c	=	nominal shear resistance of the concrete (kip) (2.7.2.2) (2.10.5.1.3)
V_f	=	nominal shear resistance provided by transverse reinforcement (kip) (2.7.3.3)
V_r	=	factored shear resistance (kip) (2.7.2.1)
V_n	=	nominal shear resistance (kip) (2.7.2.1)
V_u	=	factored shear force (kip); factored shear force for the girder or for the web under consideration (kip) (2.7.2.2) (2.7.3.6.2)
v_u	=	shear stress on concrete (ksi) (2.7.2.8)
y_t	=	distance from the neutral axis to the extreme tension fiber (in.) (2.6.3.4.2)
w	=	maximum crack width limit in a concrete component (in.) (2.6.7)
α	=	angle of inclination of transverse reinforcement to longitudinal axis (degrees); bar location modification factor (2.7.3.5) (2.9.7.4.1)
β	=	factor indicating the ability of diagonally cracked concrete to transmit tension and shear (2.7.3.4)
β_1	=	factor taken as 0.85 for concrete compressive strengths not exceeding 4 ksi. For concrete strengths exceeding 4 ksi, β_1 shall be reduced at a rate of 0.05 for each 1 ksi of strength in excess of 4 ksi, except that β_1 shall not be taken to be less than 0.65 (2.6.3.1)
γ_d	=	parameter to account for the variation in stiffness along the length of the member (2.6.3.4.2)
ϵ_{cu}	=	ultimate strain in concrete (2.6.2.1)
ϵ_f	=	longitudinal tensile strain in the section at the centroid of the GFRP tension reinforcement (in./in.) (2.7.3.6.2)
ϵ_{fd}	=	design tensile strain at rupture of GFRP reinforcing bars considering reductions for service environment (2.4.2.1)
ϵ_{ft}	=	tensile strain in extreme tension GFRP at nominal resistance (2.5.5.2)
ϵ_{fu}	=	tensile strain at rupture for product certification as reported by GFRP manufacturers (2.4.2.1)
θ	=	angle of inclination of diagonal compressive stresses (degrees) (2.7.3.5)
μ	=	friction factor (2.7.4)
ξ	=	ratio of distance from neutral axis to extreme tension fiber, $(h - kd)$, to distance from neutral axis to center of tensile reinforcement, $(d - kd)$ (2.6.7)
ρ_f	=	GFRP reinforcement ratio ($\rho_f = A_f/bd$) (2.5.3)
$\rho_{f,st}$	=	ratio of GFRP shrinkage and temperature reinforcement area to gross concrete area ($\rho_f = A_f/bh$) (2.9.6)
ϕ	=	resistance factor (2.5.5.2)

2.4—MATERIAL PROPERTIES

2.4.1—General

Designs should be based on the concrete material properties cited in the *AASHTO LRFD Bridge Design Specifications* and on the use of materials that conform to the standards for the grades or properties of construction materials as specified in the *AASHTO LRFD Bridge Construction Specifications*, and these specifications.

When other grades or types of materials are used, their properties, including statistical variability, shall be established prior to design. The minimum acceptable

properties and test procedures for such materials shall be specified in the contract documents.

The contract documents shall define the grades or properties of all materials to be used.

GFRP reinforcing bars shall conform to the material specifications in ASTM D7957/D7957M, and comply with the applicable requirements of Article 1.3.

2.4.2—GFRP Reinforcing Bars

2.4.2.1—Tensile Strength and Strain

The tensile strength and strain for product certification of GFRP reinforcing bars as reported by manufacturers shall comply with the provisions of ASTM D7957/D7957M.

The guaranteed tensile strength for product certification as reported by GFRP manufacturers, f_{fu} , shall be defined as the mean tensile strength calculated for a frequency and number of specimens as specified in ASTM D7957/D7957M minus three times the standard deviation of the samples tested according to a specified method.

The design tensile strength and strain considering reductions for service environment, f_{fd} and ϵ_{fd} , respectively, to be used in all design equations shall be determined as follows:

$$\begin{aligned} f_{fd} &= C_E f_{fu} \\ \epsilon_{fd} &= C_E \epsilon_{fu} \end{aligned} \quad (2.4.2.1-1)$$

where:

C_E = environmental reduction factor as specified in Table 2.4.2.1-1

f_{fu} = tensile strength for product certification as reported by GFRP manufacturers (ksi)

ϵ_{fu} = tensile strain at rupture for product certification as reported by GFRP manufacturers

Table 2.4.2.1-1—Environmental Reduction Factors

Exposure Condition	Environmental Reduction Factor, C_E
Concrete not exposed to earth or weather	0.80
Concrete exposed to earth or weather	0.70

2.4.2.2—Modulus of Elasticity

The modulus of elasticity for product certification of GFRP reinforcing bars as reported by manufacturers shall

C2.4.2.1

The tensile strength values as reported by the manufacturer provide a 99.87 percent probability that such values are exceeded by GFRP reinforcing bars produced using the same constituent material formulations and manufacturing processes, provided that the number of tested specimens complies with the provisions of ASTM D7957/D7957M. The manufacturer should provide a description of the method used to obtain the reported tensile strength.

The material properties provided by the manufacturers are considered as initial properties that do not include the effects of long-term exposure to the environment. Concrete bridge members reinforced with GFRP bars are typically subject to exterior (i.e., earth or weather) exposure. Because long-term exposure to various environments may reduce the tensile strength, creep rupture stress, and fatigue endurance of GFRP reinforcing bars (D'Antino et al. 2018), the material properties used in all design equations are reduced based on type and level of environmental exposure.

comply with the provisions of ASTM D7957/D7957M and shall be determined as the average modulus of elasticity calculated from experimental tests for a frequency and number of specimens as specified in ASTM D7957/D7957M.

2.5—LIMIT STATES AND DESIGN METHODOLOGIES

2.5.1—General

2.5.1.1—Limit-State Applicability

Structural components shall be proportioned to satisfy the requirements at all appropriate service, fatigue, creep rupture, strength, and extreme event limit states at all stages during the life of the structure. Unless otherwise specified by the Owner, the load combinations and load factors specified in Section 3 of the *AASHTO LRFD Bridge Design Specifications* and elsewhere in this section shall be used.

Stress concentrations due to loads and restraints or imposed deformations shall be considered.

The effects of imposed deformations due to shrinkage, temperature change, creep, and movements of supports shall be investigated.

2.5.1.2—Design Methodologies

2.5.1.2.1—General

Conventional beam theory based on Bernoulli's plane section hypothesis shall be considered applicable for the service, fatigue, and creep rupture limit states. At the strength and extreme event limit states, regions of a concrete structure shall be characterized by their behavior as B-Regions (beam or Bernoulli) or D-Regions (disturbed or discontinuity). Bernoulli's hypothesis of straight-line strain profile, and therefore conventional beam theory, may be assumed to apply in B-Regions. A more complex variation in stress and strain exists in D-Regions as shown in Figure 2.5.1.2.1-1, where the effective depth of the member, d , is defined as the distance from extreme compression fiber to centroid of tensile reinforcement.

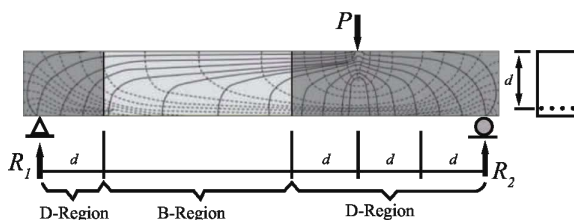


Figure 2.5.1.2.1-1—Stress Trajectories within B- and D-Regions of a Flexural Member (adapted from Birrcher et al., 2009)

C2.5.1.1

For determinate structures, experience may show that evaluating the redistribution of force effects as a result of creep and shrinkage is unnecessary.

C2.5.1.2.1

D-Regions occur in the vicinity of load or geometric discontinuities. In Figure 2.5.1.2.1-1, the applied load and support reactions are discontinuities that “disturb” the regions of the member near the locations at which they act. Frame corners, dapped ends, openings, and corbels are examples of geometric discontinuities which correspond to the existence of D-Regions.

The distribution of strains through the member depth in D-Regions is nonlinear, and the assumptions that underlie the sectional design procedure are therefore invalid. According to St. Venant's principle, an elastic stress analysis indicates that a linear distribution of stress can be assumed at approximately one member depth from a load or geometric discontinuity. In other words, a nonlinear stress distribution exists within one member depth from the location where the discontinuity is introduced (Schlaich et al., 1987). D-Regions are therefore assumed to extend approximately a distance d from the applied load and support reactions in Figure 2.5.1.2.1-1. In the case of the reaction at an interior support, the disturbed region extends a distance d on each side of the reaction.

B-Regions occur between D-Regions, as shown in Figure 2.5.1.2.1-1. Plane sections are assumed to remain plane within B-Regions according to the primary tenets of beam theory, implying that a linear distribution of strains occurs through the member depth. The beam is therefore

D-Regions shall be taken to encompass locations with abrupt changes in geometry or concentrated forces. Based upon St. Venant's principle, D-Regions may be assumed to span one member depth on either side of the discontinuity in geometry or force.

Where the effective depth changes along the component, the length of the D-Regions should be varied accordingly.

2.5.1.2.2—B-Regions

Design practices for B-Regions shall be based on a sectional model for behavior. Design for flexure in B-Regions shall be based upon the conventional beam theory of Article 2.6. Design for shear in B-Regions shall be based on conventional beam theory in conjunction with the truss analogy of Article 2.7.

Conventional beam theory is applicable to all limit states.

2.5.1.2.3—D-Regions

For the strength and extreme event limit states, the strut-and-tie method or other methods from applicable provisions in these specifications and Article 5.8 of the *AASHTO LRFD Bridge Design Specifications* may be used for the design of all types of D-Regions in structural concrete.

The most familiar types of D-Regions, such as beam ends, diaphragms, deep beams, brackets, corbels and beam ledges, may be designed by the empirical approaches or the legacy detailing practices such as those found in Article 5.8 of the *AASHTO LRFD Bridge Design Specifications*, in addition to applicable provisions in these specifications.

2.5.2—Service Limit State

Actions to be considered at the service limit state shall be cracking, deformations, and concrete stresses, as specified in Articles 2.6.7, 2.6.3, 2.6.3.5, and 2.6.4.5, respectively.

The cracking stress shall be taken as the modulus of rupture specified in Article 5.4.2.6 of the *AASHTO LRFD Bridge Design Specifications*.

2.5.3—Creep Rupture Limit State

The maximum sustained tensile stress in the GFRP reinforcement, $f_{f,s}$, shall be calculated using the dead loads and live loads included in the Service I load combination specified in Table 3.4.1-1 of the *AASHTO LRFD Bridge Design Specifications*, where the live load factor shall be reduced from 1.0 to 0.2 unless otherwise specified by the Owner. The resulting stress shall satisfy Eq. 2.5.3-1.

dominated by sectional behavior, and design can proceed on a section-by-section basis (i.e., sectional design). For the flexural design of a B-Region, the compressive stresses (represented by solid lines in Figure 2.5.1.2.1-1) are conventionally assumed to act over a rectangular stress block, while the tensile stresses (represented by dashed lines) are assumed to be carried by the longitudinal GFRP reinforcing bars.

C2.5.1.2.2

Sectional models are appropriate for the design of typical bridge girders, slabs, and other regions of components where the assumptions of traditional engineering beam theory are valid. This theory assumes that the response at a particular section depends only on the calculated values of the sectional force effects, i.e., moment, shear, axial load, and torsion, and does not consider the specific details of how the force effects were introduced into the member.

C2.5.1.2.3

D-Regions of concrete members reinforced with GFRP bars may be designed following the methods specified in Article 5.8 of the *AASHTO LRFD Bridge Design Specifications*, together with applicable provisions from these specifications for the relevant mechanical properties and design and detailing requirements for GFRP reinforcing bars. More research is needed to develop specific analysis and design methods for D-Regions of concrete members reinforced with GFRP bars.

C2.5.3

Creep rupture occurs when the sustained stress exceeds the creep rupture factor times the design tensile strength of GFRP bars. To avoid creep rupture of GFRP reinforcing bars the sustained stress level shall be limited by the creep rupture stress. Because these stress levels will be within the elastic range of the member, the stresses can be computed through an elastic analysis.

$$f_{f,s} \leq C_c f_{fd} \quad (2.5.3-1)$$

Using the full live load in determining the applied stress is prohibitively conservative. Therefore, the load factor applied to live load is reduced from 1.0 to 0.2 as a means to account for the sustained portion of the live load (Nanni et al., 2014).

where:

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

and:

C_c = creep rupture reduction factor

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

n_f = modular ratio = E_f/E_c

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

k = ratio of depth of neutral axis to
reinforcement depth

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

$M_{s,s}$ = moment due to dead loads and sustained
portion of live loads included in
Service I load combination (kip-in.)

The creep rupture reduction factor, C_c , shall be equal to 0.30 unless the GFRP reinforcing bar manufacturer can provide substantiating evidence following ASTM D7337/D7337M asserting that higher values can be safely utilized.

The cracked moment of inertia and ratio of neutral axis depth to reinforcement depth for singly reinforced, rectangular cross sections without compression reinforcement bent in uniaxial bending may be computed as follows:

$$I_{cr} = \frac{bd^3}{3} k^3 + n_f A_f (d - kd)^2 \quad (2.5.3-3)$$

in which:

$$k = \sqrt{2\rho_f n_f + (\rho_f n_f)^2} - \rho_f n_f \quad (2.5.3-4)$$

where:

A_f = area of GFRP reinforcement (in.²)

b = width of the compression face of the
member (in.)

k = ratio of depth of neutral axis to depth of flexural reinforcement

n_f = modular ratio = E_f/E_c

ρ_f = GFRP reinforcement ratio = A_f/bd

2.5.4—Fatigue Limit State

Fatigue need not be investigated for concrete deck slabs in multigirder applications.

The maximum tensile stress in the GFRP reinforcement, $f_{f,f}$, resulting from the dead loads included in the Service I load combination and the factored live loads included in the Fatigue I load combination, as specified in Table 3.4.1-1 and Article 3.6.1.4 of the *AASHTO LRFD Bridge Design Specifications*, shall satisfy Eq. 2.5.4-1.

$$f_{f,f} \leq C_f f_{fd} \quad (2.5.4-1)$$

where:

$$f_{f,f} = \frac{n_f d (1-k)}{I_{cr}} M_{s,f} \quad (2.5.4-2)$$

and:

C_f = fatigue rupture reduction factor

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

n_f = modular ratio = E_f/E_c

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

k = ratio of depth of neutral axis to depth of flexural reinforcement

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

$M_{s,f}$ = moment due to dead loads included in Service I load combination and factored live loads included in Fatigue I load combination (kip-in.)

The fatigue rupture reduction factor, C_f , shall be equal to 0.25 unless the GFRP reinforcing bar manufacturer can provide substantiating evidence following ASTM D3479/D3479M asserting that higher values can be safely utilized.

C2.5.4

Stresses measured in concrete deck slabs of bridges in service are far below the fatigue stress limit, $C_f f_{fd}$, calculated using Eq. 2.5.4-1 (Benmokrane et al., 2004, 2006, 2007), most probably due to internal arching action.

To avoid failure of GFRP reinforcing bars due combined sustained and cyclic (fatigue) stress, the stress levels in the reinforcement under these stress conditions shall be limited. Because these stress levels will be within the elastic range of the member, the stresses can be computed through an elastic analysis.

The cracked moment of inertia and ratio of neutral axis depth to reinforcement depth for singly reinforced, rectangular cross sections without compression reinforcement bent in uniaxial bending may be computed using Eq. 2.5.3-3.

2.5.5—Strength Limit State

2.5.5.1—General

The strength limit state issues to be considered shall be those of strength and stability.

Factored resistance shall be the product of nominal resistance as determined in accordance with the applicable provisions of Articles 2.6, 2.7, 2.8, 2.9, 2.10, and 2.11, unless another limit state is specifically identified, and the resistance factor as specified in Article 2.5.5.2.

2.5.5.2—Resistance Factors

The resistance factor, ϕ , shall be taken as:

- For compression-controlled and tension-controlled reinforced concrete sections as specified in Article 2.6.3:

$$\phi = \begin{cases} 0.55 & \text{for } \epsilon_{ft} = \epsilon_{fd} \\ 1.55 - \frac{\epsilon_{ft}}{\epsilon_{fd}} & \text{for } 0.80\epsilon_{fd} < \epsilon_{ft} < \epsilon_{fd} \\ 0.75 & \text{for } \epsilon_{ft} \leq 0.80\epsilon_{fd} \end{cases} \quad (2.5.5.2-1)$$

where:

ϵ_{fd} = design tensile strain of GFRP reinforcing bars (Eq. 2.4.2.1-1)

ϵ_{ft} = tensile strain in extreme tension GFRP at nominal resistance

- For shear and torsion 0.75
- For compression-controlled sections with spirals or ties, as specified in Article 2.6.4 0.75
- For bearing on concrete 0.70
- For compression in strut-and-tie models 0.70
- For tension in strut-and-tie models 0.55

C2.5.5.2

A reliability analysis on FRP reinforced concrete beams subject to flexure using load combination 2 from ACI 318-05 (for live to dead load ratios between 1 and 3) resulted in reliability indexes between 3.5 and 4.0 when ϕ is set to 0.65 and 0.55 for compression-controlled and tension-controlled sections, respectively (Gulbrandsen, 2005). More recently, a comparative reliability analysis of steel- and GFRP-reinforced concrete beams resulted in ϕ with a lower limit of 0.70 instead of 0.55 for tension-controlled sections, and an upper limit of 0.75 instead of 0.65 for compression-controlled sections (Jawaheri Zadeh and Nanni, 2013).

Because tension-controlled concrete members reinforced with GFRP bars do not exhibit ductile behavior, a conservative resistance factor of 0.55 has been retained.

While a concrete crushing failure mode can be predicted based on calculations, the member, as constructed, may not fail accordingly. For example, if the concrete strength is higher than specified, the member can fail due to GFRP rupture. For this reason and to establish a transition between the two values of ϕ , 0.55 and 0.75, a section controlled by concrete crushing is defined as a section in which $\epsilon_{ft} \leq 0.80\epsilon_{fd}$, and a section controlled by GFRP rupture is defined as one in which $\epsilon_{ft} = \epsilon_{fd}$. The resulting relation between resistance factor for flexure and tensile strain at failure in the GFRP reinforcement is illustrated in Figure C2.5.5.2-1.

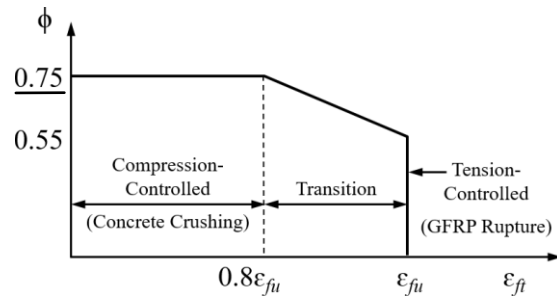


Figure C2.5.5.2-1—Variation of ϕ with Tensile Strain at Failure, ϵ_{ft} , in GFRP Reinforcement

2.5.5.3—Stability

The structure as a whole and its components shall be designed to resist sliding, overturning, uplift and buckling. Effects of eccentricity of loads shall be considered in the analysis and design.

Buckling of precast members during handling, transportation, and erection shall be investigated.

2.5.6—Extreme Event Limit State

The structure as a whole and its components shall be proportioned to resist collapse due to extreme events, specified in Table 3.4.1-1 of the *AASHTO LRFD Bridge Design Specifications*, as may be appropriate to its site and use.

C2.5.6

The lack of ductility and limited potential for moment redistribution after cracking should be carefully considered under this limit state where the resistance factor is typically specified as 1.0. In fact, different from steel RC members, neither the formation of plastic hinges nor moment redistribution can be assumed for GFRP RC members. Large deformations and limited rotations may still be possible for tension-controlled GFRP RC members due to concrete cracking and loss of stiffness. However, it is difficult to reliably quantify related beneficial effects.

2.6—DESIGN FOR FLEXURAL AND AXIAL FORCE EFFECTS—B-REGIONS

2.6.1—Assumptions for Service, Fatigue, and Creep Rupture Limit States

The following assumptions may be used in the design of GFRP reinforced concrete components for all compressive strength levels:

- The strains in the concrete vary linearly, except in components or regions of components for which conventional strength of materials is inappropriate.
- Where transformed section analysis is used to assess the time-dependent response to permanent loads, an age adjusted effective modular ratio of $2n_f$ is applicable.

C2.6.1

Examples of components for which the assumption of linearly varying strains may not be suitable include deep components such as deep beams, corbels, and brackets.

Transformed section properties are used in the working stress methods based on elastic and time-dependent analysis, for instantaneous and creep effects, respectively. The methods are applicable for service and fatigue limit states. Approximate analysis using gross section properties may be adequate in some designs provided that volume change effects are recognized.

2.6.2—Assumptions for Strength and Extreme Event Limit States

2.6.2.1—General

Factored resistance of concrete components shall be based on the following assumptions:

- Strain is directly proportional to the distance from the neutral axis, except for disturbed regions.
- If the concrete is unconfined, the maximum usable strain at the extreme concrete compression fiber, ϵ_{cu} , is not greater than 0.003.
- The stress in the reinforcement is based on a stress-strain curve representative of the GFRP or on an approved mathematical representation. GFRP reinforcement shall be taken as linear-elastic up to failure.
- The tensile strength in the concrete is neglected.
- The concrete compressive stress-strain distribution is assumed to be rectangular, parabolic, or any other shape that results in a prediction of strength in substantial agreement with independent test results.
- Perfect bond exists between concrete and GFRP reinforcement.
- Balanced strain conditions exist at a cross section when tension GFRP reinforcement reaches the strain, ϵ_{fd} , corresponding to its design tensile strength, f_{fd} , just as the concrete in compression reaches its assumed ultimate strain, ϵ_{cu} , of 0.003.
- Sections are compression-controlled when $\epsilon_{ft} < \epsilon_{fd}$ and failure of the component is initiated by crushing of the concrete.
- Sections are tension-controlled when $\epsilon_{ft} = \epsilon_{fd}$ and failure of the component is initiated by rupture of the GFRP reinforcing bars.
- The use of compression reinforcement in conjunction with additional tensile reinforcement is not permitted in the analysis to increase the strength of flexural members.

2.6.2.2—Rectangular Stress Distribution

The natural relationship between concrete stress and strain may be considered satisfied by an equivalent rectangular concrete compressive stress block of $0.85 f'_c$ over a zone bounded by the edges of the cross section and a straight line located parallel to the neutral axis at the distance $a = \beta_1 c$ from the extreme compression fiber. The distance c shall be measured perpendicular to the neutral axis. The factor β_1 shall be taken as specified in Article 2.3.

C2.6.2.1

The flexural resistance of a GFRP reinforced member is dependent on whether the failure is controlled by crushing of the concrete or rupture of GFRP bars. The appropriate failure mode may be determined by comparing the GFRP reinforcement ratio to the balanced reinforcement ratio when concrete crushing and GFRP rupture occur simultaneously at a strain equal to ϵ_{cu} and ϵ_{fd} , respectively.

C2.6.2.2

For practical design, the rectangular compressive stress distribution defined in this Article may be used in lieu of a more exact concrete stress distribution. This rectangular stress distribution does not represent the actual stress distribution in the compression zone at ultimate, but in many practical cases it does provide essentially the same results as those obtained in tests. All strength equations presented in Article 2.6.3 are based on the rectangular stress block.

The stress block factor β_1 is fundamentally related to rectangular sections; however, for flanged sections in which the neutral axis is in the web, β_1 has experimentally been found to be an adequate approximation.

2.6.3—Flexural Members

2.6.3.1—Stress in GFRP Reinforcement at Nominal Flexural Resistance

When failure of the concrete component is initiated by rupture of the GFRP reinforcement, the maximum stress in the reinforcement equals its design tensile strength, f_{fd} . When failure is initiated by crushing of the concrete, the effective strength in the reinforcement, f_f , for strength and extreme event limit states may be determined as:

$$f_f = \sqrt{\frac{(E_f \epsilon_{cu})^2}{4} + \frac{0.85\beta_1 f'_c}{\rho_f} E_f \epsilon_{cu} - 0.5 E_f \epsilon_{cu}} \leq f_{fd} \quad (2.6.3.1-1)$$

where:

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

ϵ_{cu} = ultimate strain in concrete (Article 2.6.2.1)

β_1 = factor dependent on concrete strength as specified in Article 2.3

f'_c = specified compressive strength of concrete (ksi)

ρ_f = GFRP reinforcement ratio = A_f/bd

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

2.6.3.2—Flexural Resistance

2.6.3.2.1—Factored Flexural Resistance

The factored flexural resistance, M_r , shall be taken as:

$$M_r = \phi M_n \quad (2.6.3.2.1-1)$$

where:

M_n = nominal flexural resistance (kip-in.)

ϕ = resistance factor as specified in Article 2.5.5.2

2.6.3.2.2—Rectangular Sections

When $\epsilon_{ft} < \epsilon_{fu}$, the failure of the member is initiated by crushing of the concrete. Based on force equilibrium

C2.6.3.2.2

The flexural resistance of a concrete member reinforced with GFRP bars is dependent on whether the

and strain compatibility considerations, the nominal flexural resistance may be calculated as:

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

in which:

$$a = \frac{A_f f_f}{0.85 f'_c b} \quad (2.6.3.2.2-2)$$

where:

- A_f = area of GFRP reinforcement (in.²)
- f_f = effective strength in GFRP tensile reinforcement at the strength and extreme event limit state as specified in Eq. 2.6.3.1-1 (ksi)
- d = distance from extreme compression fiber to centroid of tensile reinforcement (in.)
- a = depth of equivalent rectangular stress block (in.)
- f'_c = specified compressive strength of concrete (ksi)
- b = width of the compression face of the member (in.)

When $\epsilon_{ft} = \epsilon_{fus}$, the failure of the member is initiated by rupture of the GFRP reinforcing bars. The rectangular stress block approach is not applicable because the ultimate concrete strain, $\epsilon_{cu} = 0.003$, may not be attained. In lieu of a more detailed analysis, a simplified and conservative determination of the nominal flexural resistance of the member may be based on Eq. 2.6.3.2.2-3:

$$M_n = A_f f_{fd} \left(d - \frac{\beta_1 c_b}{2} \right) \quad (2.6.3.2.2-3)$$

in which:

$$c_b = \left(\frac{\epsilon_{cu}}{\epsilon_{cu} + \epsilon_{fd}} \right) d \quad (2.6.3.2.2-4)$$

where:

- f_{fd} = design tensile strength of GFRP reinforcing bars (Eq. 2.4.2.1-1) (ksi)
- β_1 = factor dependent on concrete strength as specified in Article 2.3.
- c_b = distance from extreme compression fiber to neutral axis at balanced strain condition (in.)

failure is governed by concrete crushing or GFRP rupture (Nanni, 1993; Benmokrane et al., 1996). Both failure modes are acceptable in governing the design of flexural members provided that strength and serviceability requirements are satisfied.

When the failure of the member is initiated by crushing of the concrete, the stress distribution in the concrete may be approximated with the rectangular stress block approach.

When the failure of the member is initiated by rupture of the GFRP reinforcement, the rectangular stress block approach is not applicable. In this case, an equivalent stress block would need to be used to approximate the stress distribution in the concrete at the particular strain level attained.

ϵ_{cu} = ultimate strain in concrete (Article 2.6.2.1)

ϵ_{fd} = design tensile strain at rupture of GFRP reinforcing bars (Eq. 2.4.2.1-1)

2.6.3.2.3—Other Cross Sections

For cross sections other than rectangular sections with vertical axis of symmetry or for sections subjected to biaxial flexure without axial load, the nominal flexural resistance, M_n , shall be determined by an analysis based on the assumptions specified in Article 2.6.2. The requirements of Article 2.6.3.3 shall apply.

2.6.3.2.4—Strain Compatibility Approach

Alternatively, the strain compatibility approach may be used if more precise calculations are required. The appropriate provisions of Article 2.6.2.1 shall apply.

The stress and corresponding strain in any given layer of reinforcement may be taken from any representative stress-strain formula or graph for GFRP reinforcement approved by the Owner or established through independent physical tests approved by the Owner.

2.6.3.3—Limits for Reinforcement

There is no maximum reinforcement limit.

Unless otherwise specified by the Owner, at any section of a noncompression-controlled flexural component, the amount of tensile reinforcement shall be adequate to develop a factored flexural resistance, M_r , greater than or equal to the lesser of the following:

- 1.33 times the factored moment required by the applicable strength load combination specified in Table 3.4.1-1 of the *AASHTO LRFD Bridge Design Specifications*.

$$1.6f_r S_c - M_{dnc} \left(\frac{S_c}{S_{nc}} - 1 \right) \quad (2.6.3.3-1)$$

C2.6.3.2.4

In the case of steel-reinforced concrete sections, all steel tension reinforcement is assumed to yield at ultimate when using the strength design method to calculate the resistance of members with reinforcement arranged in multiple layers. Therefore, the tension force is assumed to act at the centroid of the reinforcement with a magnitude equal to the area of tension reinforcement times the yield strength of steel. Because GFRP bars exhibit a linear elastic behavior until failure, the stress in each reinforcement layer will vary depending on its distance from the neutral axis. Similarly, if different types of GFRP bars are used to reinforce the same member, the variation in the stress level in each bar type should be considered when calculating the flexural resistance. In these cases, failure of the outermost layer is most likely to control overall reinforcement failure, and the flexural resistance should be based on a strain-compatibility approach.

C2.6.3.3

Minimum reinforcement provisions are intended to reduce the probability of brittle failure upon concrete cracking.

where:

- f_r = modulus of rupture of concrete specified in Article 5.4.2.6 of the *AASHTO LRFD Bridge Design Specifications* (ksi)
- M_{dnc} = total unfactored dead load moment acting on the monolithic or noncomposite section (kip-in.)
- S_c = section modulus for the extreme fiber of the composite section where tensile stress is caused by externally applied loads (in.³)
- S_{nc} = section modulus for the extreme fiber of the monolithic or noncomposite section where tensile stress is caused by externally applied loads (in.³)

Appropriate values for M_{dnc} and S_{nc} shall be used for any intermediate composite sections. Where the beams are designed for the monolithic or noncomposite section to resist all loads, S_{nc} shall be substituted for S_c in Eq. 2.6.3.3-1.

2.6.3.4—Deformations

2.6.3.4.1—General

The provisions of Article 2.5.2.6 of the *AASHTO LRFD Bridge Design Specifications* shall be considered.

Deck joints shall accommodate the dimensional changes caused by loads, creep, shrinkage, thermal changes, and settlement.

2.6.3.4.2—Deflection and Camber

Deflection and camber calculations shall consider dead load, live load, erection loads, and concrete creep and shrinkage.

In the absence of a more comprehensive analysis, instantaneous deflections may be computed using the modulus of elasticity for concrete as specified in Article 5.4.2.4 of the *AASHTO LRFD Bridge Design Specifications* and taking the moment of inertia as appropriate, using either the gross moment of inertia, I_g , or an effective moment of inertia, I_e , given by Eq. 2.6.3.4.2-1:

$$I_e = \frac{I_{cr}}{1 - \gamma_d \left(\frac{M_{cr}}{M_a} \right)^2 \left(1 - \frac{I_{cr}}{I_g} \right)} \leq I_g \quad (2.6.3.4.2-1)$$

in which:

In this instance, the resulting requirement is that the factored flexural resistance, M_r , be at least 60 percent greater than the cracking moment, M_{cr} , similar to the *AASHTO LRFD Bridge Design Specification* for low-ductility reinforcement.

C2.6.3.4.1

For more precise determinations of long-term deflections, the creep and shrinkage coefficients cited in Article 5.4.2.3 of the *AASHTO LRFD Bridge Design Specifications* should be utilized. These coefficients include the effects of aggregate characteristics, humidity at the structure site, relative thickness of member, maturity at time of loading, and length of time under loads.

C2.6.3.4.2

Camber is the deflection that is built into a member, either intentionally or by accident, other than prestressing, ideally to achieve the desired roadway geometry with minimal buildup of the top flange.

GFRP-reinforced concrete members have a relatively small stiffness after cracking. Therefore, permissible deflections under service loads can control the design. In general, designing using compression-controlled sections satisfies serviceability criteria for deflection and crack width (Nanni, 1993; Thériault and Benmokrane, 1998; Nanni et al., 2014). The section-based expression to calculate the effective moment of inertia, I_e , proposed by Bischoff (2005) is modified in Eq. 2.6.3.4.2-1 to include the factor γ_d . This factor is intended to account for the variation in stiffness along the length of the member. This approach provides reasonable estimates of deflection for beams and one-way slabs (Bischoff et al., 2009). The factor γ_d may be calculated using Eq. 2.6.3.4.2-3 in lieu of a more comprehensive analysis (Bischoff and Gross, 2011).